FIGURES

APPENDIX 2A FIGURES

APPENDIX 2B FIGURES

FIGURES

<u>Appendix 3B Figures</u>

1. Support load combination is seismic plus deadweight.
2. Support load combination is seismic plus deadweight.

FIGURES

5 REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS

5 REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS

5 REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS

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9.3.4.4.6.1 Suction Lines 136

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1 INTRODUCTION AND GENERAL DESCRIPTION OF THE PLANT

1.1 INTRODUCTION

Ginna Station is located in Wayne County, near Rochester, New York. The Ginna reactor is a pressurized light water moderated and cooled system designed by Westinghouse. A renewed operating license was issued to R.E. Ginna Nuclear Power Plant by NRC letter dated May 19, 2004. The renewed operating license is effective from the date of issuance through September 18, 2029.

Technical Specification Amendment 115 was issued on April 1, 2014 which approved the transfer of the license for R. E. Ginna Nuclear Power Plant (Ginna) held by R. E. Ginna Nuclear Power Plant, LLC, (Ginna LLC) to Exelon Generation Company, LLC, as approved by Order dated March 24, 2014. The joint venture held between Constellation Energy Nuclear Group, LLC, (CENG) and Électricité de France, S.A., was not modified as part of Amendment 115. The joint venture consists of a 50.01% ownership interest of an ultimate domestic parent Exelon Generation Company, LLC, and a 49.99% ownership interest of an ultimate foreign parent, Électricité de France, S.A., a French corporation (*Reference 1*).

Rochester Gas and Electric filed the application for a construction permit and operating license in October 1965. The construction permit was issued on April 25, 1966. The initial submittal of the Final Facility Description and Safety Analysis Report was filed in March 1969, and the initial provisional operating license was issued on September 19, 1969.

Ginna Station began commercial operation in July 1970, at a licensed output of 1300 MWt and at 420 MW net electrical power. On March l, 1972, the licensed output was increased to 1520 MWt and the net electrical output was increased to 490 MW. In August 1972 RG&E applied for a full-term operating license. The Safety Evaluation Report related to the full-term operating license for the R. E. Ginna Nuclear Power Plant (NUREG 0944) was published in October 1983; Supplement 1 was published in October 1984. The full-term operating license was issued on December 10, 1984. The license was to expire on April 25, 2006. On August 8, 1991, the license was amended to change the expiration date to September 18, 2009, which is 40 years after the date of issuance of the provisional operating license.

During the October 2006 refueling outage, Ginna Station completed the Extended Power Uprate (EPU) Project. The NRC approved the EPU under Technical Specification Amendment No. 97 on July 11, 2006. This license change authorized an approximate 16.8% increase in the steady-state thermal power level from 1520 megawatts thermal to 1775 megawatts thermal. The EPU changed the design electrical rating from 470 MW to 585 MW. Changes to the plant as a result of EPU have been incorporated in the UFSAR.

The R. E. Ginna Nuclear Power Plant was reviewed under Phase II of the Systematic Evaluation Program (SEP). The review began in 1978 and the Integrated Plant Safety Assessment, Final Report, NUREG 082l, was issued by the NRC in December 1982. Supplement 1 to NUREG 0821 was issued in August 1983.

The Ginna Station primary coolant system configuration consists of two hot legs, two U-tube steam generators, a pressurizer, and two cold legs with a reactor coolant pump in each cold leg. The secondary system consists basically of the turbine generator, the condenser, and the

feedwater and condensate systems. Auxiliary equipment includes a radioactive waste disposal system, fuel handling system, main transformer, circulating water system, engineered safety features systems, and all auxiliaries, structures, and onsite facilities required to provide for a complete and operable nuclear power plant. A more detailed list of structures, systems, and components is provided in Section 3.2. The turbine and condenser system as well as the nuclear steam supply system were designed and supplied by Westinghouse. The remainder of the plant was designed by either RG&E or Gilbert Associates, Incorporated. The replacement steam generators were designed and supplied by Babcock and Wilcox International (BWI).

The reactor containment structure was designed by Gilbert Associates. It is a reinforced-concrete, vertical right cylinder with a flat base and a hemispherical dome. A welded steel liner is attached to the inside face of the concrete shell to provide for leaktightness. The containment cylinder is founded on rock by post-tensioned rock anchors. The cylinder wall is prestressed vertically by tendons coupled to the rock anchors.

Ginna Station is located on the south shore of Lake Ontario, which is the source of circulating water and the ultimate heat sink. The site initially contained 338 acres. In 1973 the site, including the switchyard, was increased to 488 acres. As a result of the purchase of Ginna Station by Constellation Energy in 2004, the site was reduced to approximately 426 acres.

REFERENCES FOR SECTION 1.1

1. Letter from Nadiyah S. Morgen, NRC, to Mary G. Korsnick and Bryan P. Wright, Constellation Energy Group: R.E. Ginna Nuclear Power Plant – Issuance of Amendment to Conform the Renewed Facility Operating License to Reflect the Direct Transfer of Operating Authority (TAC No. MF2588), dated April 1, 2014.

1.2 GENERAL PLANT DESCRIPTION

1.2.1 SITE AND ENVIRONMENT

The site is on the south shore of Lake Ontario l6 miles east of Rochester, New York, an urban area of about 700,000. The area immediately around the site is sparsely populated and is utilized primarily for farming. The site, in open, rolling terrain, is well ventilated and is not generally subject to severe flooding. Liquids released to the lake from the site will move predominately eastward and diffuse slowly. Hurricanes have not seriously affected the site region and tornadoes and severe ice storms are rare. Onsite measurements indicate that ground water within the site will flow to the lake and will not affect offsite wells.

The site has sound bedrock on which major structures are founded and is in a seismologically quiet region. It is within l50 miles of the St. Lawrence Valley area, where earthquakes of Richter magnitude 7 have been experienced, and 35 miles from the area around Batavia-Attica which has experienced moderate seismological activity of smaller magnitudes.

1.2.2 SUMMARY PLANT DESCRIPTION

The inherent design of the pressurized water reactor ensures that the probability of release of significant quantities of fission products to the atmosphere is low. Four barriers exist between the fission product accumulation and the environment. These are the uranium dioxide fuel matrix, the fuel cladding, the reactor vessel and coolant loops, and the reactor containment. The consequences of a breach of the fuel cladding are greatly reduced by the ability of the uranium dioxide lattice to retain fission products. Escape of fission products through a fuel cladding defect would be contained within the pressure vessel, loops, and auxiliary systems. A breach of these systems or equipment would release the fission products to the reactor containment where they would be retained. The reactor containment is designed to adequately retain these fission products under the most severe accident conditions, the design-basis loss-of-coolant accident. This accident and its consequences are analyzed in Section 15.6.

Several engineered safety features have been incorporated into the plant design to reduce the consequences of a loss-of-coolant accident. These safety features include a safety injection system (Emergency Core Cooling System (ECCS)). This system automatically delivers borated water to the reactor vessel for cooling under high and low reactor coolant pressure conditions. The safety injection system also serves to insert negative reactivity into the core in the form of borated water during an uncontrolled plant cooldown following a steam line break or an accidental steam release. Other safety features which have been included in the reactor containment design are a containment air recirculation, cooling, and filtration system, which would effect a depressurization of the containment following a loss of coolant and provide for iodine filtration if fission products are released from the core; and a containment spray system which would depressurize the containment and remove elemental iodine from the atmosphere by a washing action. The containment spray system and containment air recirculation, cooling, and filtration system are redundant containment heat removal systems. Additional engineered safety features are listed in Section 3.2.

1.2.3 STRUCTURES

1.2.3.1 General

The major structures are a reactor containment, auxiliary building, intermediate building, control building, turbine building, screen house, all volatile-treatment or condensate demineralizer building, standby auxiliary feedwater (SAFW) building, diesel generator buildings, and the service building containing offices, shops, and laboratories. A general plan of the building arrangement is shown in Figure 1.2-1. Several drawings in the 33013-2100 series show the general internal layout of the buildings. Structures containing equipment that is associated with, or required for, operating the plant are part of the power block. Additionally, the old steam generator storage facility is located northwest of the plant outside the security fence.

The reactor containment is a vertical, cylindrical reinforced-concrete type with prestressed tendons in the vertical wall, reinforced-concrete ring anchored to the bedrock and a reinforced hemispherical dome. The containment is designed to withstand the internal pressure accompanying a loss-of-coolant accident or main steam line break and to provide adequate radiation shielding for both MODES 1 and 2 and accident conditions.

1.2.3.2 Containment

The reactor containment structure is a reinforced-concrete vertical right cylinder with a flat base and a hemispherical dome. A welded steel liner is attached to the inside face of the concrete shell to ensure a high degree of leaktightness. The thickness of the liner in the cylinder and dome is 3/8-in. and in the base it is 1/4 in. The cylindrical reinforced-concrete walls are 3 ft 6 in. thick, and the concrete hemispherical dome is 2 ft 6 in. thick. These thicknesses are nominal values. The true relevant engineering values are dependent on the specific location in the structure and the loading condition that is present. The concrete base slab is 2 ft thick with an additional thickness of concrete fill of 2 ft over the bottom liner plate. The containment structure is 99 ft high to the spring line of the dome and has an inside diameter of 105 ft. The containment vessel provides a minimum free volume of approximately 1,000,000 ft^3. Access is provided during operation by means of two personnel airlocks designed with an interlocked single-door-opening feature that is leak testable at containment design pressure between doors. The open and closed status of each door is indicated in the control room.

The major components of the reactor coolant system are located within the containment structure. The containment structure provides a physical barrier to protect the equipment from natural disasters and shielding to protect personnel from radiation emitted from the reactor core while at power.

The reactor vessel is located in the center of the containment structure below ground level. Extending around the reactor vessel is a stainless-steel-lined refueling cavity. During MODE 6 (Refueling) operations, the refueling cavity is flooded with borated water to provide shielding of the irradiated fuel being removed from the reactor vessel.

Thick reinforced-concrete walls are located around the major reactor coolant system components to serve as shielding for plant personnel. These walls also serve as a missile barrier to

prevent damage to the containment wall and to components of the safety injection system should a failure occur to one of the reactor coolant system components located inside the walls.

The containment houses the following major equipment (see Drawings 33013-2101, 33013-2102, 33013-2105, 33013-2106, 33013-2107, 33013-2113, 33013-2114, 33013-2115, 33013-2131, and 33013-2132):

1. Reactor coolant loop piping, reactor coolant pumps, and steam generators.
2. Pressurizer.
3. Pressurizer relief tank.
4. Reactor coolant drain tank and pumps.
5. Containment recirculation filtering and cooling units (four).
6. Safety injection system accumulators.
7. Refueling cavity and equipment.

1.2.3.3 Auxiliary Building

The auxiliary building is located just south of the containment. The auxiliary building houses the major support and engineered safety features equipment for plant operation. The auxiliary building is a restricted area and normal exit is from the intermediate building (hot side), as shown in Drawing 33013-2116.

The auxiliary building has three major levels and a subbasement level pit which contains the residual heat removal pumps. The refueling water storage tank (RWST) extends through all three levels. The following is a list of major equipment on each level of the auxiliary building.

Auxiliary Building Basement (See Drawing 33013-2103)

1. Chemical and volume control system holdup tanks.
2. Residual heat removal pumps (subbasement).
3. Residual heat removal heat exchangers.
4. Spent fuel pool pump.
5. Residual heat pump cooling.
6. Boric acid evaporator.
7. Gas stripper.
8. Waste holdup tank.
9. Various operations panels.
10. Waste evaporator (system physically removed in 1999).
11. Blender room.
12. Spent resin tanks.

13. Safety injection filters.
14. Seal injection filters.
15. Containment spray pumps.
16. Nonregenerative heat exchanger.
17. Seal return filter and cooler.
18. Charging pump rooms and accumulator.
19. Sodium hydroxide tank and leakoff tank.
20. Safety injection pumps (three).
21. Safety injection accumulator makeup pump.

Auxiliary Building - Intermediate Level (See Drawing 33013-2108)

1. Spent fuel pool filter and heat exchanger.
2. Chemical and volume control system holdup tanks.
3. Residual heat removal heat exchangers.
4. Waste gas compressors and gas stripper.
5. Gas decay tanks (four).
6. Reactor coolant filter.
7. Volume control tank.
8. Concentrates holding tank and transfer pump.
9. Demineralizer vault.
10. High efficiency particulate air filters.
11. Nonregenerative heat exchanger.
12. 480-V bus 16 (vital bus).
13. Charcoal filter unit.
14. Motor control center 1D.
15. Motor control center 1M.

Auxiliary - Building Operating Floor (See Drawing 33013-2116)

1. Decontamination pit.
2. Spent fuel storage pool, crane, and transfer canal.
3. New fuel unloading area.
4. New fuel storage racks.
5. Auxiliary building maintenance shop.
6. Crane bay.
7. Refueling water storage tank (RWST) (all levels).

8. Component cooling pumps.
9. Component cooling water heat exchangers and surge tank.
10. Boric acid demineralizers.
11. Monitor tanks and pumps.
12. Waste condensate tanks.
13. Reactor makeup water tank and pumps.
14. Drumming station and drum storage area.
15. 480-V bus 14 (vital bus).
16. Auxiliary building supply fan and filter.
17. Boric acid batching tank.
18. Boric acid storage tank and boric acid transfer pumps.
19. Waste condenser demineralizer.
20. Motor control center 1C.
21. Motor control center 1L.
22. Motor control center 1E.
23. Vendor supplied demineralization system.

1.2.3.4 Intermediate Building (See Drawings 33013-2101, 33013-2102, 33013-2105, 33013-2106, 33013-2107, 33013-2113, 33013-2114, 33013-2115, and 33013-2121)

The intermediate building surrounds the containment building to the west and north and joins the service building and turbine building. It is divided into two sections called the hot side (restricted area access) and the cold side.

<u>Hot Side (Restricted Area Access)</u>

The hot side is west of the containment building and joins the service building, intermediate building cold side, and auxiliary building. Personnel enter and exit the intermediate building hot side, at the access control area.

The intermediate building hot side extends from the access control area to the personnel door to the auxiliary building, spent fuel pool (SFP) area. The intermediate building hot side has four levels, plus a subbasement for access to the containment tendons. In addition, there is a mezzanine level for access to the containment personnel hatch. The following equipment is among that located in the intermediate building cold side:

1. Primary sample room.
2. Post-accident sample panel.
3. Hydrogen recombiner panel.
4. Auxiliary building exhaust fans A, B, and C.
5. Auxiliary building HEPA filter bank.

6. Intermediate building exhaust fans A and B.
7. Access control area exhaust fans A and B.
8. Access control area HEPA and charcoal filter banks.

Cold Side

The intermediate building cold side is a radiologically unrestricted area. The intermediate building cold side provides access to the cable tunnel area. The building is constructed to partially surround the containment structure to the north and west and house its support equipment.

Access to the intermediate building cold side is normally made from the turbine building. Doors from the cold side to the hot side are available but not normally used. The following equipment is among that located in the intermediate building cold side:

1. Turbine-driven auxiliary feedwater pump (TDAFW).
2. Motor-driven auxiliary feed pumps (MDAFW) (two).
3. Rod control power panels.
4. Rod control logic cabinets.
5. Rod drive motor-generator sets and power panels.
6. Reactor trip and bypass breakers.
7. Auxiliary building and containment ventilation units.
8. Safety and relief valves (main steam).
9. Purge exhaust fans.
10. Radiation monitors (e.g., R-11, R-12).
11. Main steam and feedwater lines.

1.2.3.5 Turbine Building

The turbine building is located north of the intermediate building. The turbine building houses the major secondary system equipment and systems, including the main turbine, generator, and condenser (see Drawing 33013-2140 and Drawing 33013-2141). The following equipment is located on each level of the turbine building:

Basement level (See Drawing 33013-2104)

1. Main feedwater pumps (2).
2. Fire service water storage tank.
3. Turbine oil reservoir and purifier.
4. Turbine oil pumps (on top of reservoir).
5. Steam dump valves.
6. Circulating water inlet and outlet headers.

7. Seal-oil unit.
8. Blowdown recovery system.
9. Bus duct cooling fans.
10. Condensate coolers.
11. Condensate pumps (three).
12. Condensate booster pumps (three).
13. Heater drain tank.
14. Heater drain tank pumps.
15. Motor control center 1A.

Intermediate Level-Mezzanine (See Drawing 33013-2112)

1. Low-pressure heaters (inside of condenser).
2. Moisture separator reheater units (four).
3. Main feedwater regulating valves.
4. Hydrazine and NH_4OH addition tanks.
5. Feedwater heaters 1A, 1B, 2A, 2B, 3A, 3B, 4A, 4B, 5A, and 5B.
6. Air ejector and condenser.
7. Gland exhaust condenser.
8. Generator bus ducts.
9. Main power panels and motor control centers: 4l60-V buses 11A, 11B, 12A, 12B; 480-V bus 13, 15; and motor control center 1B.
10. Secondary sampling station.
11. Electro-hydraulic oil system.

Operating Floor (See Drawing 33013-2120)

1. Main turbine and generator.
2. Intercept and low pressure stop valves.
3. Entrance to main control room.

1.2.3.6 Control Building

The control building is adjacent to the turbine building and consists of three floors (see Drawings 33013-2123, 33013-2124, 33013-2125 and 33013-2136). The main control room is on the upper floor. The relay room is directly below the control room and houses relay racks and the multiplexer (MUX) room. The battery rooms and the air handling room are on the lowest level of the control building.

1.2.3.7 All-Volatile-Treatment Building

The all-volatile-treatment building houses demineralizers and other equipment necessary for the condensate polishing system to allow all-volatile-treatment of secondary water (see Drawing 33013-2111).

The technical support center is located on the second floor of the all-volatile-treatment building and houses the computers and equipment, including emergency power supplies (diesel generator and batteries), necessary to provide the staff technical support during an emergency event (see Drawing 33013-2119).

1.2.3.8 Standby Auxiliary Feedwater Pump Building

The standby auxiliary feedwater pump (SAFW) building is located on the southeast corner of the auxiliary building and houses the two standby auxiliary feedwater pumps (SAFW). The building is a Seismic Category I concrete structure supported by caissons (see Drawing D-024-017)

1.2.3.9 Screen House

The screen house is located north of the turbine building on Lake Ontario and houses the main circulating water inlet lines and pumps; the service water (SW) pumps (four); 480-V switchgear buses 17 and 18, the diesel fire pump, the motor-driven fire pump, and motor control center G (MCCG) (see Drawing 33013-2143).

1.2.3.10 Service Building

The service building is located at the west end of the auxiliary building. This building provides the office spaces for the administrative staff at Ginna Station (see Drawings 33013-2109, 33013-2110, 33013-2117, and 33013-2118).

The service building has two levels. The basement level is comprised of storerooms, machine shops, maintenance areas, etc. The basement level also contains a water treatment area, Material and Test Equipment area, and maintenance management offices.

The ground level consists primarily of offices for groups such as Operations, Maintenance Support, Radiation Protection, and Chemistry. The ground level also contains the cafeteria, fire brigade response room, locker rooms, plant management offices, and Instrument and Control office/shop.

1.2.3.11 Diesel Generator Building

The diesel generator building adjoins the turbine building on the east end of the north wall opposite the control building. The building is a one-story reinforced-concrete structure that houses the emergency diesel generators.

1.2.3.12 Old Steam Generator Storage Facility

The old steam generator storage facility (OSGSF) is a reinforced concrete building which will provide long-term storage of the two old steam generators and the attached insulation material. Also stored in the OSGSF are the control rod drive mechanisms (CRDM) and related equipment removed from the plant during the 2003 refueling outage.

The OSGSF is a stand-alone facility located outside the existing security perimeter fence and will have no interface with permanent plant structures.

1.2.3.13 Canister Preparation Building

A Canister Preparation Building (CPB) located south of the Auxiliary Building was constructed at the Ginna Nuclear Generating Station for the general purpose of performing spent fuel Dry Shielded Canister (DSC) and Transfer Cask handling and preparation activities.

The CPB superstructure is designed to meet the applicable requirements of 10 CFR 50, the Ginna UFSAR, and the Building Code of New York State. The CPB superstructure is a seismic II/I structure such that the building cannot adversely impact the transfer cask, Auxiliary Building, or DSC when fuel is present.

The CPB and large overhead door opening through the south wall of the Auxiliary Building are considered functionally an extension of the Auxiliary Building. The CPB is part of the Auxiliary Building and for NEIL insurance purposes will be considered to act as an Auxiliary Building Truck Bay.

A 30-ton building crane was installed in the CPB. The 30-ton crane is supported on a crane structure mounted to the building columns, which permits operation of the crane in the north-south direction. The 30 ton trolley operates in the east-west direction.

The new 125-ton single failure proof cantilevered gantry crane has a stationary runway mounted to an embedded steel support system. A rolling bridge is mounted on top of the stationary runway. A main trolley is mounted on the rolling bridge with a flying trolley mounted to the main trolley.

1.2.3.14 ISFSI Transfer Path and Storage Pad

The Independent Spent Fuel Storage Installation (ISFSI) pad site is located north and west of the station power block and south of the Meteorological (MET) Tower. The ISFSI pad site was constructed to provide storage capacity for 30 loaded Transnuclear (TN) Dry Shielded Canisters (DSC's). Storage capacity is intended to satisfy spent fuel storage requirements through the end of the extended plant life with the reactor defueled and the SFP full. The ISFSI site was comprised of a reinforced concrete foundation slab (pad) surrounded by a reinforced concrete approach slab (apron) and a concrete haul path to facilitate the transfer of the fuel.

The ISFSI pad was placed on top of the soil mixed elements that stabilized the soil. These soil mixed elements extend approximately 20 feet outside of the ISFSI pad on all sides.

The construction of the ISFSI pad and aprons was performed while the area was outside the Protected Area boundary. Upon completion of the construction, the Protected Area boundary was extended to include the ISFSI pad and aprons.

1.2.3.15 Administration Building

A new Administration Building was constructed in 2005 to house more personnel onsite. The Administration Building is a two-story structure located on the west side of the Service Building. This building contains conference rooms, an auditorium, and offices for groups such as Scheduling, Planning, Information Technology, Procurement, and Finance. The Administration Building also contains the first aid/Fitness for Duty office. Records Management, which includes a protected vault, is located on the first floor of the building.

The Outage Control Center (OCC) is also located in the Administration Building. The OCC is used at the Operational Support Center (OSC) during plant emergencies.

1.2.4 NUCLEAR STEAM SUPPLY SYSTEM

The nuclear steam supply system consists of a pressurized water reactor, reactor coolant system, and associated auxiliary fluid systems. The reactor coolant system is arranged as two closed reactor coolant loops connected in parallel to the reactor vessel, each containing a reactor coolant pump and a steam generator. An electrically heated pressurizer is connected to one of the loops (loop B).

The reactor core is composed of uranium dioxide pellets enclosed in zircaloy or ZIRLO™ tubes with welded end plugs. The tubes are supported in assemblies by a grid structure. The mechanical control rods consist of clusters of stainless steel clad absorber rods inserted into guide tubes located within the fuel assembly. The core fuel is divided into several regions.

The replacement steam generators are vertical U-tube units containing Inconel tubes. Integral separating equipment reduces the moisture content of the steam at the steam generator outlet nozzle to 0.1% or less.

The reactor coolant pumps are vertical, single-stage, centrifugal pumps equipped with controlled leakage shaft seals.

Auxiliary systems are provided to add makeup water to the reactor coolant system, purify reactor coolant water, provide chemicals for corrosion inhibition and reactor control, cool system components, remove residual heat when the reactor is shut down, cool the spent fuel storage pool, sample reactor coolant water, provide for emergency safety injection, vent and drain the reactor coolant system, and for other purposes.

1.2.5 REACTOR AND PLANT CONTROL

The reactor is controlled by a coordinated combination of chemical shim and mechanical control rods. The control system allows the plant to accept step load increases of 10% and ramp load increases of 5% per minute over the load range of 12.8% to 100%. Similar step and ramp load reductions are possible over the range of 100% to 12.8%.

Complete supervision of both the nuclear and turbine generator plants is accomplished from the central control room. This supervision includes the capability to test periodically the operability of the Reactor Trip System (RTS).

1.2.6 WASTE DISPOSAL SYSTEM

The waste disposal system provides all equipment necessary to collect, process, and prepare for disposal all potentially radioactive liquid, gaseous, and solid wastes produced as a result of reactor operation.

Liquid wastes requiring cleanup before release are collected and processed by a vendor supplied demineralization system. After appropriate cleaning and filtering, the liquid is collected in the chemical and volume control system monitor tank A or B for ultimate release to the circulating water discharge canal at a concentration below 10 CFR 20 limits. The spent demineralizer resin is packaged and shipped from the site for ultimate disposal in an authorized location. Liquid wastes were also processed by the waste evaporator system until 1990 when use of the evaporator was discontinued. The waste evaporator package was physically removed in 1999.

Gaseous wastes are collected and stored until their radioactivity level is low enough so that discharge to the environment does not create radioactivity concentrations above 10 CFR 20 limits.

Solid wastes including evaporator concentrates are packaged and shipped from the site for ultimate disposal in an authorized location. Wet solid wastes are solidified. Dry solid wastes are shipped in bulk form to a vendor for volume reduction and packaging for delivery to a disposal site.

Operating procedures generally limit normal effluents to within 10 CFR 50, Appendix I, limits. Sanitary waste from Ginna Station is piped into the Town of Ontario, New York, sewer system.

1.2.7 FUEL HANDLING SYSTEM

The reactor is refueled with equipment designed to handle spent fuel under water from the time it leaves the reactor vessel until it is placed in a cask for shipment from the site. Underwater transfer of spent fuel provides an optically transparent radiation shield, as

well as a reliable source of coolant for removal of decay heat.

1.2.8 TURBINE AND AUXILIARIES

The turbine is a tandem-compound, three-cylinder, 1800-rpm unit having 40-in. exhaust blading in the low-pressure elements. Four combination moisture separator reheater units are employed to dry and superheat the steam between the high- and low-pressure turbine cylinders.

A single-pass deaerating, radial flow surface condenser, steam jet air ejectors, three half-capacity condensate pumps, three half-capacity condensate booster pumps, two half-capacity main feedwater pumps, and five stages of feedwater heaters are provided. One preferred auxiliary turbine-driven (TDAFW), two preferred auxiliary motor-driven (MDAFW), and two standby auxiliary motor-driven feedwater pumps (SAFW) are available in case of a complete loss of offsite power.

1.2.9 ELECTRICAL SYSTEM

The main generator is a 1800 rpm, three-phase, 60-cycle, hydrogen inner cooled unit. The main step-up transformer is a conventional two-winding forced oil air cooled unit.

The station service system consists of auxiliary transformers, 4160-V and 480-V switchgear, 480-V motor control centers, and 125-V dc equipment.

Emergency power supplied by one of two diesel-engine-driven generators is capable of operating postaccident safeguards equipment or safe shutdown equipment to ensure an acceptable plant response.

1.2.10 ENGINEERED SAFETY FEATURES PROTECTION SYSTEMS

The engineered safety features protection systems provided for the station have sufficient redundancy of component and power sources such that under the conditions of a design-basis loss-of-coolant accident, the system can, even in the event of a single failure, maintain emergency core cooling, maintain the integrity of the containment, and perform other safeguards functions to ensure that postaccident exposures are maintained below the guidelines of 10 CFR 100.

The systems provided are:

A. The containment system, which provides an essentially leaktight barrier against the escape of fission products. The containment penetrations and liner weld seams are provided with a leak test system, which can be utilized to check the integrity of these two locations that are the most likely sources of containment leakage. Very low leakage requirements are also imposed on the containment isolation valves.

B. The safety injection system, which provides borated water to cool the core by injection into the cold legs of the reactor coolant loops and by injection over the top of the core through nozzles that penetrate the reactor vessel.

C. The containment recirculation fan cooler (CRFC) and filtration system, which provides a dynamic heat sink to cool the containment atmosphere and filtration of the containment atmosphere to remove airborne particulate and halogen fission products

that form the source for potential public exposure. The system utilizes the normal containment ventilation and cooling equipment in addition to the charcoal filters.

D. The containment spray system, which provides a spray of cool, chemically treated borated water to the containment atmosphere to provide additional heat sink and iodine removal capability together with the containment air recirculation cooling and filtration system.

E. The hydrogen recombiners, which limit the concentration of hydrogen in containment following a loss-of-coolant accident.

F. Auxiliary systems, which serve to ensure the operability of the above systems.

1.2.11 DESIGN HIGHLIGHTS

The design of Ginna Station was based upon proven concepts which have been developed and successfully applied in the construction of pressurized water reactor systems. In subsequent sections, a few of the design features of Ginna Station are listed that represent slight variations or extrapolations from units such as San Onofre and Connecticut-Yankee, which were licensed to operate before Ginna Station.

1.2.11.1 Power Level

The power level is 1520 MWt. This is greater than the capability of the San Onofre plant, but smaller than the capability of the Connecticut Yankee plant (1825 MWt). Therefore, this power level does not represent any significant variation from the power levels of other pressurized water reactors in operation at the time Ginna Station was licensed. In 2006 the licensed core power level for Ginna Station was increased to 1775 MWt.

1.2.11.2 Reactor Coolant Loops

The reactor coolant system for Ginna Station consists of two loops, as compared with three loops for San Onofre and four loops for Connecticut Yankee, and required an attendant increase in the size and capacity of the reactor coolant system components such as the reactor coolant pumps, piping, and steam generators. These increases represented reasonable engineering extrapolations of existing and proven designs at the time and, as such, the components of the reactor coolant system were designed for conditions exceeding operation at 1520 MWt.

1.2.11.3 Peak Specific Power

Based on the design hot channel factors, operation at 1520 MWt produces a peak specific power of 13.5 kW/ft for a 12 month fuel cycle (with F_Q of 2.32) and 14.2 kW/ft for an 18 month fuel cycle (with F_Q of 2.45). For an 18 month cycle at 1775 MWt core power with a design hot channel F_Q=2.60, the resulting peak specific power is 18.25 kW/ft.

1.2.11.4 Fuel Clad

The initial fuel rod design for Ginna Station utilized zircaloy as a clad material, which has proven successful in other operations. ZIRLO™ is also being used as a clad material, commencing in 1999.

1.2.11.5 Fuel Assembly Design

The fuel assembly is a canless type with the basic assembly consisting of the rod cluster control guide thimbles fastened to the grids and the top and bottom nozzles. The fuel rods are held by the grids and grid springs, which provide lateral and axial support.

Ginna Station was initially fueled with Westinghouse fuel. Starting with cycle 8 (1978), Exxon fuel was used. Starting with cycle 14 (1984), Westinghouse (optimized fuel assemblies) fuel is being used. Commencing with cycle 28 (1999), Westinghouse (VANTAGE +) fuel is being used. Commencing with cycle 33 in 2006 as part of the plant uprate to 1775 MWt, Ginna started using Westinghouse 422V+ fuel assemblies.

1.2.11.6 Engineered Safety Features

The engineered safety features provided are of the same types provided for the Connecticut Yankee plant augmented by borated water injection accumulators. There is a safety injection system of the Connecticut Yankee type which can be operated in part (any two of three high-head pumps and any one of two low-head pumps) from emergency onsite diesel power. The system design is such that it can be tested while the plant is at power. There is containment recirculation fan cooler (CRFC) and filtration for post-loss-of-coolant conditions inside the containment that utilize the normal ventilation system flow path so that deterioration is not expected. Provisions are made for periodic testing to determine the condition of the filter material. A containment spray system provides cool, borated water sprayed into the containment atmosphere for additional cooling and iodine removal capacity.

1.2.11.7 Emergency Power

In addition to the multiple ties to outside sources for emergency power, two diesel generator units are provided as backup power supplies in case of a loss of all outside power. Each generator is capable of operating sufficient safeguards equipment to ensure an acceptable post loss-of-coolant containment pressure transient.

1.2.12 STATION WATER USE

The total nominal flow of circulating water through the turbine condenser and service water (SW) systems is about 400,000 gpm. Approximately 340,000 gpm is used in the turbine condenser system and the rest is available for the service cooling supply and fire protection systems. In addition, domestic quality water at a flow of about 100,000 gal/day is purchased from the Ontario Water District, Town of Ontario, for drinking, sanitary purposes, auxiliary boiler feed, and condensate makeup and polishing.

Lake Ontario is the source of the circulating water, which is taken through the eight 17.3 ft. wide by 10 ft. high ports of the submerged octagonal intake structure that lies about 3100 ft. offshore in about 33 ft. of water at mean lake level, 244.7 ft. [International Great Lakes Datum, 1955 (IGLD 1955)]. Six of the eight ports are screened for large debris with heater bars spaced 14 in. apart; the screens can be heated electrically to minimize accumulation of frazil ice. Two of the eight ports are non-heated and have open space of approximately 68 in. x 112 in. to prevent accumulation of frazil ice. Refer to Section 10.6.2.1 for a current description of the configuration of the intake structure screens. The water flows by gravity through a 10 ft. diameter concrete lined tunnel into the screen house, where it passes through a fine mesh traveling screen before being pumped through

the turbine condenser or service water (SW) system. The water from these two systems is combined and is released to the discharge canal, which opens into Lake Ontario at the shoreline. The discharge canal is protected from large debris by a submarine net placed inside the canal near the shoreline.

1.2.13 FACILITY SAFETY CONCLUSIONS

The safety of the public and station operating personnel and reliability of plant equipment and systems were primary considerations in the plant design. The approach taken in fulfilling the safety consideration was three fold. First, careful attention was given to the design so as to prevent the release of radioactivity to the environment under conditions which could be hazardous to the health and safety of the public. Second, the plant was designed so as to provide adequate protection for plant personnel wherever a potential radiation hazard exists. Third, reactor systems and controls were designed with a great degree of redundancy and fail-safe characteristics.

Based on the overall design of the plant and its engineered safety features and the analysis of the possible incidents and of design basis accidents, it was concluded that Ginna Station can be operated with no undue hazard to the public health and safety.

1.3 COMPARISON TABLES

The information presented in Section 1.3.1 provides a comparison of the R. E. Ginna Nuclear Power Plant as originally licensed at 1300 MWt output and as originally uprated to 1520 MWt output to Point Beach Units 1 and 2 as originally licensed. It also compares Ginna as originally licensed at 1300 MWt to San Onofre Unit 1 and Connecticut Yankee. The information presented in Section 1.3.2 identifies the significant changes made in the Ginna Nuclear Power Plant design between submittal of the PSAR and submittal of the original FSAR. In general, neither of these Sections have been updated. The information contained in them may or may not represent the current design of the Ginna Nuclear Power Plant.

1.3.1 COMPARISONS WITH SIMILAR FACILITY DESIGNS

The design parameters of the Ginna Nuclear Power Plant for 1520 MWt are presented in Table 1.3-1 along with the comparisons of the major parameters from the initial design rating of 1300 MWt for Ginna Nuclear Power Plant and the original Point Beach, Units 1 and 2 design rating. Table 1.3-2 presents a comparison of the Ginna Station steam and power conversion design parameters to those of San Onofre Unit 1 and Connecticut Yankee as presented in the original FSARs of the three plants.

In 2006, Ginna uprated the licensed power level from 1520 MWt to 1775 MWt. Section 1.3.3 compares the Ginna uprated parameter to comparable parameters for another Westinghouse 2 loop plant.

1.3.2 COMPARISON OF FINAL AND PRELIMINARY SAFETY ANALYSIS REPORT INFORMATION (HISTORICAL)

1.3.2.1 Partial Length Rod Cluster Control Assemblies

Previously abandoned in place, partial length rod assemblies were removed and not replaced by PCR 2001-0042, Reactor Vessel Closure Head Replacement.

1.3.2.2 Burnable Shim Rods

Burnable shim rods were added to ensure a zero or negative moderator temperature coefficient of reactivity at all times. (These are no longer used.)

1.3.2.3 Safety Injection System Trip Signal

The actuating signal for the safety injection system was revised to increase the initiation reliability and to increase protection in the case of a steam line rupture.

1.3.2.4 Containment Spray System Signal

The actuating signal for the containment spray system was revised to operate from two sets of two-out-of-three containment high-pressure signal channels.

1.3.2.5 Safety Injection System Accumulators

Two accumulators were added to the safety injection system to provide short-term core cooling before the injection pumps become effective for postulated large area primary system rupture.

1.3.2.6 Spray Additive

The containment spray additive for increasing inorganic iodine removal rate in case of a primary system rupture was changed to sodium hydroxide. (See Chapter 6).

1.3.2.7 Rod Stop and Reactor Trip on Startup

The automatic rod stop signal is actuated by an intermediate range flux level setting, and the reactor trip signal on startup is supplied by a high flux level setting.

1.3.2.8 Miniature Neutron Flux Detectors

Four miniature neutron flux detectors capable of traversing 36 thimbles replace the original three detectors in 25 thimbles to provide more detailed flux mapping during core physics tests.

1.3.2.9 Core Thermocouples

Fewer core thermocouples are provided (39 in place of 45).

1.3.2.10 Initial Leak Rate Test Method

The initial leak rate testing of the containment makes use of the absolute method instead of the reference volume method to provide higher sensitivity at low leak rates.

1.3.2.11 Auxiliary Building Ventilation Filters

Absolute and charcoal filters are added to the auxiliary building ventilation system (ABVS) to reduce air activity levels in case of recirculation system components leakage following a loss-of-coolant accident.

1.3.2.12 Control Center Buses

The 480-V system buses are increased from four to six to provide greater operating flexibility under single component failure or emergency power conditions.

1.3.2.13 Condenser Circulating Water Flow

The condenser circulating water flow was increased to 334,000 gpm.

1.3.2.14 Ramp Loading Range

The ramp loading range is increased from 15% to 95% up to 15% to 100% of full load.

1.3.2.15 Condensate Storage Tanks Capacity

The two condensate storage tanks total capacity is 60,000 gal (decreased from 72,000 gal or 6.5 hr versus 8 hr capacity). A third tank with a 100,000 gal. capacity has been added. It is located outdoors next to the all-volatile-treatment building. See Section 9.2.4.

1.3.2.16 Fuel Transfer System Drive

An air-motor drive replaces the cable drive for the fuel transfer conveyor car. The air-motor was removed by PCR 2005-0033. See Section 9.1.4.3.4.

1.3.2.17 Steam Line Flow Nozzles

Steam line flow nozzles were incorporated to limit the consequences of a steam line rupture.

1.3.3 COMPARISON OF UPRATE PARAMETERS

In 2006 Ginna implemented a power uprate to increase core licensed power from 1520 MWt to 1775 MWt. Prior to the Ginna uprate, Kewaunee which is a Westinghouse 2 loop plant similar to Ginna, had also implemented a power uprate. A comparison of the key NSSS parameters at the uprated power level for both plants is presented in Table 1.3-3.

Table 1.3-1

COMPARISON OF DESIGN PARAMETERS WITH POINT BEACH
[Represents original design parameters for plants listed and may not represent current design of the plants]

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
HYDRAULIC AND THERMAL DESIGN PARAMETERS			
Total heat output, MWt	1518.5	1300	1520
Total heat output, Btu/hr	5181×10^6	4437×10^6	5188×10^6
Heat generated in fuel,%	97.4	97.4	97.4
Peak specific power, kW/ft	16	16.5	16.0
System pressure, nominal, psia	2250	2250	2250
System pressure, minimum steady-state, psia	2220	2220	2220
Hot-channel factors			
Heat flux, F_Q	2.80	3.38	2.80
Enthalpy rise, $F_{\Delta H}$	1.60	1.77	1.66
DNBR at nominal conditions	2.11	2.15	2.06
Minimum DNBR for design transients	1.30	1.30	1.30
Coolant flow			
Total flow rate, lb/hr	66.7×10^6	67.3×10^6	68.0×10^6
Effective flow rate for heat transfer, lb/hr	63.6×10^6	64.3×10^6	64.9×10^6
Effective flow area for heat transfer, ft^2	27.0	27.0	27.0
Average velocity along fuel rods, ft/sec	15.0	14.7	14.8
Average mass velocity, lb/hr-ft^2	2.37×10^6	2.38×10^6	2.41×10^6

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Coolant temperature, °F			
Nominal inlet	552.5	551.9	544.5
Maximum inlet due to instrumentation, error, and deadband	556.5	555.9	548.5
Average rise in vessel	57.6	49.5	58.0
Average rise in core	60.0	52	60.5
Average in core	582.5	578.0	575.8
Average in vessel	581.3	577.0	573.5
Nominal outlet of hot channel	642.9	634.0	637.8
Average film coefficient, Btu/hr-ft ^{2}-°F	5600	5590	5690
Average film temperature difference, °F	31.0	26.9	30.9
Heat transfer at 100% power			
Active heat transfer surface area, ft^2	28,715	28,715	28,715
Average heat flux, Btu/hr-ft^2	175,800	150,500	176,700 (Region 4)[a] 176,000 (Region 3)[a]
Maximum heat flux, Btu/hr-ft^2	491,000	508,700	494,800 (Region 4) 492,700 (Region 3)
Average thermal output, kW/ft	5.7	4.88	5.7
Maximum thermal output, kW/ft	16.0	16.5	16.0
Maximum clad surface temperature at nominal pressure, °F	657	657	657

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Fuel central temperature, °F			
Maximum at 100% power	≈3750	3880	3900 (Region 4) 3850 (Region 3)
Maximum at overpower	≈4000	4100	4500 (Region 4) 4500 (Region 3)
Thermal output, kW/ft at maximum overpower	17.9	18.5	21.1
CORE MECHANICAL DESIGN PARAMETERS			
Fuel assemblies			
Design	RCC canless 14 x 14	RCC canless 14 x 14	RCC canless 14 x 14
Rod pitch, in.	0.556	0.556	0.556
Overall dimensions, in.	7.763 x7.763	7.763 x7.763	7.763 x 7.763
Fuel weight (as UO_2), lb	118,729	118,729	118,246[b]
Total weight, lb	154,519	150,750	150,267[b]
Number of grids per assembly	7	9	9
Fuel rods			
Number	21,659	21,659	21,659
Outside diameter, in.	0.422	0.422	0.422
Diametral gap, in.	0.0065	0.0065	0.0085 (Region 4) 0.0065 (Region 3)
Clad thickness, in.	0.0243	0.0243	0.0243
Clad material	Zircaloy	Zircaloy-4	Zircaloy-4

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Fuel pellets			
Material	UO_2 Sintered	UO Sintered	UO Sintered
Density (% of theoretical)	Unit 1 94-92-91 Unit 2 94-93-92	92-90	92 (Region 4) 90 (Region 3)
Diameter, in	0.3699	0.3699	0.3649 (Region 4) 0.3669 (Region 3)
Length, in.	0.6000	0.6000	0.6000
Rod cluster control assemblies			
Neutron absorber	5% Cd, 15% In, 80% Ag	5% Cd, 15% In, 80% Ag	5% Cd, 15% In, 80% Ag
Cladding material	Type 304 SS-Cold Worked	Type 304 SS-Cold Worked	Type 304 SS-Cold Worked
Clad thickness, in	0.019	0.019	0.019
Number of clusters, full/part-length	37	29/4	29/4
Number of control rods per cluster	16	16	16
Core structure			
Core barrel I.D./O.D., in.	109.0/112.5	109.0/112.5	109.0/112.5
Thermal shield I.D./O.D., in.	115.3/122.5	115.3/122.5	115.3/112.5

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
NUCLEAR DESIGN DATA			
Structural characteristics			
Fuel weight (as UO_2), lb	118,729	118,727	118,727
Clad weight, lb	24,260	22,440	22,440
Core diameter, in. (equivalent)	96.5	96.5	96.5
Reflector thickness and composition	144	144	143.4 (Region 4) 144 (Region 3)
Top-water plus steel, in.	10	≈10	≈10
Bottom-water plus steel, in.	10	≈10	≈10
Side-water plus steel, in.	15	≈15	≈15
H_2O/U, unit cell (cold volume ratio)	3.35	3.35	3.35
Number of fuel assemblies	121	121	121
UO_2 rods per assembly	179	179	179
Performance characteristics			
Loading technique	3 region, nonuniform	3 region, nonuniform	3 region, nonuniform
Fuel discharge burnup, MWd/MTU			
Average first cycle	15,100	≈14,126	≈8,000
First core average	33,000	24,400	24,400
Feed enrichments, wt %			
Region 1	2.27	2.44	2.44
Region 2 (first core with burnable poison)	3.03	2.78	2.78
Region 3	3.04	3.48	3.48
Equilibrium	3.40	3.00	3.00

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Control characteristics (beginning-of-life) Effective multiplication (with burnable poison)			
Cold, no power, clean	1.211	1.188	1.188
Hot, no power, clean (T_{mod} = 573 °F)	1.167	1.137	1.137
Hot, full power, xenon and Samarium equilibrium	1.113	1.080	1.080
Rod cluster control assemblies			
Material	5% Cd, 15% In, 80% Ag	5% Cd, 15% In, 80% Ag	5% Cd, 15% In, 80% Ag
Number of rod cluster control assemblies	37	33	33
Number of absorber rods per rod cluster control assembly	16	16	16
Total rod worth	7.1%	6.8%	6.8%
Boron concentrations (first cycle with burnable poison)			
To shut reactor down with no rods inserted, clean,(keff = .99) cold/ hot	1598 ppm/1676 ppm	1630 ppm/1580 ppm	1630 ppm/1580 ppm
To control at power with no rods inserted, clean equilibrium xenon and samarium	1465 ppm/1007 ppm	1470 ppm/1100 ppm	1470 ppm/1100 ppm
Boron worth, hot	1% Δk/k/130 ppm	1% Δk/k/120 ppm	1% Δk/k/120 ppm
Boron worth, cold	1% Δk/k/98 ppm	1% Δk/k/90 ppm	1% Δk/k/90 ppm
Kinetic characteristics			
Moderator temperature coefficient	$+0.3 \times 10^{-4}$ to -2.5×10^{4} Δk/k/°F	$+.3$ to -3.5×10^{-4} Δk/k/°F	$+0.3$ to -3.5×10^{-4} Δk/k/°F
Moderator pressure coefficient	-0.3×10^{-6} to 3.5×10^{-6} Δk/k/psi	-0.3×10^{-6} to $+3.5 \times 10^{-6}$ Δk/k/psi	-0.3×10^{-6} to $+3.5 \times 10^{-6}$ Δk/k/psi
Moderator void (density coefficient)	-0.10 to -0.30 Δk/k/g/cm^3	-0.10 to +0.30 Δk/k/g/cm^3	-0.10 to +0.30 Δk/k/g/cm^3
Doppler coefficient	-1.0×10^{-5} to -1.6×10^{-5} Δk/k/°F	-1.0×10^{-5} to -1.6×10^{-5} Δk/k/°F	-0.93×10^{-6} to -2.9×10^{-5} Δk/k/°F

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
REACTOR COOLANT SYSTEM - CODE REQUIREMENTS			
Component			
Reactor vessel	ASME III, Class A	ASME III, Class A	ASME III, Class A
Steam generator			
Tube side	ASME III, Class A	ASME III, Class A	ASME III, Class A
Shell side	ASME III, Class C[c]	ASME III, Class C	ASME III, Class C
Pressurizer	ASME III, Class A	ASME III, Class A	ASME III, Class A
Pressurizer relief tank	ASME III, Class C	ASME III, Class C	ASME III, Class C
Pressurizer safety valves	ASME III	ASME III	ASME III
Reactor coolant piping	USAS B31.1	USAS B31.1	USAS B31.1
PRINCIPAL DESIGN PARAMETERS OF THE REACTOR COOLANT SYSTEM			
Nuclear steam supply system heat output, MWt	1518.5	1300	1520
Core heat output, Btu/hr	5181×10^6	4437×10^6	5188×10^6
Operating pressure, psig	2235	2235	2235
Reactor inlet temperature, °F	552.5	551.9	551.9
Reactor outlet temperature, °F	610.1	601.4	602.4
Number of loops	2	2	2
Design pressure, psig	2485	2485	2485
Design temperature, °F	650	650	650
Hydrostatic test pressure (cold), psig	3110	3110	3110
Total reactor coolant system volume, ft^3 (hot)	6450	6245	6245

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Total reactor flow, gpm	178,000	180,000	179,400
PRINCIPAL DESIGN PARAMETERS OF THE REACTOR VESSEL			
Material	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel
Design pressure, psig	2485	2485	2485
Design temperature, °F	650	650	650
Operating pressure, psig	2235	2235	2235
Inside diameter of shell, in.	132	132	132
Outside diameter across nozzles, in.	224 1/16	219 5/16	219 5/16
Overall height of vessel and enclosure head, ft-in.	39-0	39-1	39 1-5/16
Minimum clad thickness, in	5/32	5/32	5/32
PRINCIPAL DESIGN PARAMETERS OF THE STEAM GENERATORS			
Number of units	2	2	2
Type	Vertical, U-tube with integral moisture separator	Vertical, U-tube with integral moisture separator	Vertical, U-tube with integral moisture separator
Tube material	Inconel	Inconel	Inconel
Shell material	Carbon steel	Carbon steel	Carbon steel
Tube side design pressure, psig	2485	2485	2485
Tube side design temperature °F	650	650	650
Tube side design flow, lb/hr	33.35×10^6	33.63×10^6	33.63×10^6
Shell side design pressure, psig	1085	1085	1085
Shell side design temperature, °F	556	556	556

	Point Beach Units 1 and 2 1518 MWt	Ginna 1300 MWt	Ginna 1520 MWt
Operating pressure, tube side, nominal, psig	2235	2235	2235
Operating pressure, shell side, maximum, psig	1020	989	989
Maximum moisture at outlet at full load, %	1/4	1/4	1/4
Hydrostatic test pressure, tube side (cold), psig	3110	3110	3110
PRINCIPAL DESIGN PARAMETERS OF THE REACTOR COOLANT PUMPS			
Number of units	2	2	2
Type	Vertical, single stage radial flow with bottom suction and horizontal discharge	Vertical, single stage radial flow with bottom suction and horizontal discharge	Vertical, single stage radial flow with bottom suction and horizontal discharge
Design pressure, psig	2485	2485	2485
Design temperature, °F	650	650	650
Operating pressure, nominal, psig	2235	2235	2235
Suction temperature, °F	551.5	551.9	551.9
Design capacity, gpm	89,000	90,000	90,000
Design head, ft	259	252	252
Hydrostatic test pressure (cold), psig	3110	3110	3110
Motor type	ac induction single speed air cooled	ac induction single speed air cooled	ac induction single speed air cooled
Motor rating	6000 hp	6000 hp	6000 hp
Material	Austenitic SS	Austenitic SS	Austenitic SS
Hot leg - I.D., in.	29	29	29
Cold leg - I.D., in.	27-1/2	27-1/2	27-1/2
Between pump and steam generator - I.D., in.	31	31	31
Design pressure	2485	2485	2485

a. Region 3 was of the non-pressurized rod design; Region 4 was of the pressurized rod design.

b. Assumes reload with pressurized rods.

c. The shell side of the steam generator conforms to the requirements for Class A vessels and is so stamped as permitted under the rules of Section III.

Table 1.3-2
COMPARISON OF DESIGN PARAMETERS WITH SAN ONOFRE AND CONNECTICUT YANKEE[a]

Steam and Power Conversion Design Parameters	San Onofre Final Report	Ginna 1520 MWt	Connecticut Yankee Final Report
Turbine generator			
Turbine type	Three element, tandem compound, four-flow exhaust	Three element, tandem compound, four-flow exhaust	Three element, tandem compound, four-flow exhaust
Turbine capacity, kW			
Maximum guaranteed	450,000	496,322	616,200
Maximum calculated	450,000	516,739	646,135
Turbine speed, rpm	1800	1800	1800
Generator rating, kVa	500,000	608,400	667,000
Condensers			
Type	Single pass, horizontal divided box, deaerating	Radial flow, semicylindrical water boxes, deaerating	Single pass, divided water box, deaerating
Number	2	2	2
Condensing capacity, lb of steam/ hr	3,293,000	3,448,805	---
Condensate pumps			
Type	Vertical, wet pit	Multi-stage, vertical pit-type centrifugal	Seven-stage vertical, pit-type
Number	4	3	2
Design capacity each, (gpm)	2900	6600	6200
Motor type	Vertical, induction	Vertical	Vertical, induction
Motor rating, hp	700	1500	1500
Feedwater pumps			

Steam and Power Conversion Design Parameters	San Onofre Final Report	Ginna 1520 MWt	Connecticut Yankee Final Report
Type	Two-stage, horizontal split case, double volute, centrifugal	High-speed, barrel-type, single stage, double-flow, centrifugal	Two-stage, horizontal centrifugal
Number	2	2	2
Design capacity (each), gpm	7000 (10,500 during safety injection)	7400	9600
Motor type	Horizontal, induction	Horizontal	Horizontal, induction
Motor rating, hp	3500	5000	4500
Emergency feedwater			
Source	240,000 gal condensate storage tank	30,000 gal in each of the two condensate storage tanks (CST); Service Water	100,000 gal demineralized storage tank
Emergency feedwater pumps			
Number	2 (1 steam-driven and 1 motor driven)	3 (1 steam-driven and 2 motor driven)	1
Design capacity, gpm	300 (steam-driven), 235 (motor-driven)	400 (steam-driven), 200 (motor-driven)	450

a. The data in this table are not current.

Table 1.3-3
COMPARISON OF GINNA AND KEWAUNEE UPRATE NSSS DESIGN PARAMETERS

Parameter	GINNA	KEWAUNEE
Total Core Power	1775 MWt	1772 MWt
System Pressure	2250 psia	2250 psia
Minimum Reactor Flow	85,200 gpm/loop	89,000 gpm/loop
Coolant Volume with Pressurizer	6084 ft^3	6435 ft^3
Pressurizer Volume	800 ft^3	1000 ft^3
Maximum Inlet Temperature	540.2°F	539.2°F
Maximum Average Temperature	576.0°F	573.0°F
Maximum Outlet Temperature	611.8°F	606.8°F

1.4 IDENTIFICATION OF AGENTS AND CONTRACTORS

The Rochester Gas and Electric Corporation (RG&E), as owner, engaged or approved the engagement of the contractors and consultants identified below in connection with the design and construction of the R. E. Ginna Nuclear Power Plant. However, regardless of the explanation of contractual arrangements offered below, Rochester Gas and Electric Corporation was the sole applicant for the construction permit and operating license and as owner and applicant was responsible for the design, construction, and operation of the plant.

The R. E. Ginna Nuclear Power Plant was designed and built by the Westinghouse Electric Corporation as prime contractor for RG&E. The project was directed by Westinghouse from the offices of its Atomic Power Division in Pittsburgh, Pennsylvania, and by Westinghouse representatives at the plant site during construction and plant startup. Westinghouse engaged the engineering firm of Gilbert Associates, Inc., of Reading, Pennsylvania, to provide the design of the structures and non-nuclear portions of the plant and to prepare specifications for the purchase and construction thereof. Rochester Gas and Electric Corporation reviewed the designs and specifications prepared by Westinghouse and Gilbert Associates to ensure that the general plant arrangements, equipment, and operating provisions were satisfactory to them. Rochester Gas and Electric Corporation inspected the construction work to ensure that the plant was built in accordance with the approved plans and specifications.

The plant was constructed under the general direction of Westinghouse through a general contractor, Bechtel Corporation, who was responsible for the management of all site construction activities and who either performed the work or subcontracted the work of construction and equipment erection. Preoperational testing of equipment and systems and initial plant operation was performed by RG&E personnel under the technical direction of Westinghouse.

Rochester Gas and Electric Corporation engaged the firm of Dames and Moore of New York, New York, as consultants on studies of plant site geology, hydrology, and seismology.

Rochester Gas and Electric Corporation engaged Dr. George Sutton of La Mont Geological Observatory, Palisades, New York, as an additional consultant on seismology.

Rochester Gas and Electric Corporation engaged the firm of Pickard, Lowe, and Associates, Washington, D.C., as consultants on reactor and plant engineering, site meteorology, and general site studies. In addition, specialists in environmental sciences participated in developing information concerning the site. These included: Dr. Ben Davidson, meteorologist and Director, Geophysical Science Laboratory, New York University College of Engineering; Drs. Donald Pritchard and James Carpenter, hydrologists, and Professor and assistant Professor, respectively, Department of Oceanography, Johns Hopkins University; Dr. G. Hoyt Whipple, health physicist, Professor of Radiological Health, School of Public Health, University of Michigan; and Dr. Robert Sutton, geologist, University of Rochester.

Westinghouse engaged the firm of Praeger-Kavanagh-Waterbury of New York, New York, as consultants on the structural design of the containment and other important structures.

The firm of Hansen, Holby, and Biggs, Massachusetts Institute of Technology, was engaged for structural engineering analyses. The Southwest Research Institute, San Antonio, Texas,

was engaged as a consultant for quality control and for the establishment of an operating surveillance program.

Contractual support available during operations is discussed in Section 13.1.1.3.5.

1.5 REQUIREMENTS FOR FURTHER TECHNICAL INFORMATION

This section is provided for historical purposes and has not been updated. It includes a discussion of research and development completed and the requirement for further research and development perceived to be necessary at the time of submission of the original FSAR.

1.5.1 INTRODUCTION

Research and development to the level necessary to ensure safe operation of the R. E. Ginna Nuclear Power Plant was conducted in the following areas:

1. Development of the final core design and final thermal, hydraulic, and physics parameters.
2. Core stability including adequacy of out-of-core instrumentation.
3. Development of long ion chambers.
4. Control rod ejection accident analyses.
5. Charcoal filters for the removal of organic forms of iodine from the containment atmosphere following an accident.
6. Reactor coolant pump controlled leakage seal testing.
7. Safety injection system both design and analytical methods.
8. Development of design, inspection, and acceptance criteria for prestressed reinforced concrete pressure vessels.
9. Development of containment hydrogen recombiner.

The term "research and development" as used in this section is the same as that used by the NRC in Section 50.2 of its regulations as follows:

(n) "Research and development" means (1) theoretical analysis, exploration or experimentation; or (2) the extension of investigative findings and theories of a scientific nature into practical application for experimental and demonstration purposes including the experimental production and testing of models, devices, equipment, materials, and processes.

The research and development done for the R. E. Ginna Nuclear Power Plant confirms the engineering and design values used to complete the equipment and systems designs. It did not, in general, involve the creation of new concepts or ideas.

The technical information generated demonstrates the safety of the design and more sharply defines margins of conservatism.

1.5.2 DEVELOPMENT OF THE FINAL CORE DESIGN AND FINAL THERMAL-HYDRAULIC AND PHYSICS PARAMETERS

The detailed final core design and thermal-hydraulics and physics parameters have been completed. The nuclear design, including fuel configuration and enrichments, control rod pattern and worths, reactivity coefficients, and boron requirements are described in the original FSAR. The final thermal-hydraulics design parameters, as well as the final fuel, fuel rod, fuel

assembly, and control rod mechanical design are also discussed in detail in the original FSAR. The core design incorporates fixed burnable poison rods (*Reference 1* and *2*) in the initial loading to ensure a negative moderator temperature coefficient of reactivity at operating temperature. This improves reactor stability and lessens the consequences of a rod ejection or a loss-of-coolant accident.

1.5.3 CORE STABILITY

1.5.3.1 Core Power Distribution

In the transition to 12 ft. long, zircaloy-clad fuel cores, a potential for core power distribution oscillations due to spatial oscillation in xenon concentration was created. Analytical methods have been developed to examine this problem, and their use has resulted in the development of suitable control hardware and a control strategy.

Nuclear calculation codes have been modified to simulate these power oscillations and the operator actions necessary to damp out these oscillations. The effect of power redistribution in the core on total power capability has been calculated and the control system is designed to automatically cut back turbine power, and therefore core power, if limits on power distribution are exceeded. The protection system is designed to automatically reset thermal trip levels if these limits on power distribution are exceeded.

The core of the R. E. Ginna Nuclear Power Plant contains burnable poison rods, which eliminate the positive moderator coefficient that was expected at operating temperatures early in the first fuel cycle in the original core design. The burnable poison rods will be borosilicate glass. Critical experiments have been conducted at the Westinghouse Reactor Evaluation Center using rods containing 12.8 wt % boron and zircaloy-clad uranium dioxide fuel rods, 2.27% enriched. These values are typical of this plant also. These experiments showed that standard analytical methods can be used to calculate the reactivity worth of the burnable poison rods. The design basis and critical experiments are described in *References 1* and *2*. (Note: burnable poison rods are no longer included in the core.)

In-core testing completed in the Saxton reactor has shown satisfactory performance. The tests are continuing and the research and development effort on these burnable poison rods is described in more detail in the R&D topical report presented at the Salem Public Hearing, August 15, 1968.

1.5.3.2 Out-of-Core Ion Chambers

The control system input from the nuclear instrumentation is the signals from four 10-ft long, two-section ion chambers (described in Section 1.5.4), mounted outside the reactor vessel. Calculations have shown that the response of these ion chambers should accurately indicate gross power redistribution in the core, both axial and transverse, and this has been confirmed by experimental measurements made on the SENA, San Onofre, and Haddam Neck reactors. Tests performed to date include forcing various axial and transverse power shapes with full-length control rods, and comparing the measured out-of-core readings with detailed in-core measurements. Excellent correlation has been obtained. The calculations and results are detailed and discussed in *Reference 3*.

1.5.3.3 In-Core Control Equipment

Calculations performed for this plant demonstrate that power oscillations across the core will be inherently highly damped and no control applied damping is either provided or necessary. In any case, there is no mode of normal operation (MODES 1 and 2) which could cause a transverse power tilt or, if one occurred, would make it worse.

There is, in a zircaloy core of this length, the possibility that xenon-induced axial power distribution oscillations may occur. Detailed calculations have shown that these oscillations can be simply and effectively controlled, and suitable equipment has been developed for this plant.

The in-core control equipment consists of four part-length rods, symmetrically placed about the core axial center line, and moved in unison. Each rod has absorber in the bottom quarter only, and is raised and lowered by a mechanism that holds the rod in a fixed position following a reactor trip or loss of power to the mechanism. Since the xenon oscillation period is about 1 day, the part-length rods are under operator control. The control strategy is based on maintaining the difference in output between the top and bottom sections of the long ion chambers within a specified range. If the operator allows axial power imbalance to exceed operating limits, automatic protection occurs (*Reference 3*). The operating band is well inside core thermal limits.

The part-length control rods permit axial power shaping as well as axial power oscillation control. (Note: The part-length rods have been removed from the in-core control equipment.)

The hardware, out-of-core instrumentation adequacy, control strategy, and rod insertion limits are described in *Reference 3*. The performance of the system will be verified and the calculated performance checked during the thorough startup test program, which is described below and in Chapter 14.

1.5.3.4 Startup Test Program

Experimental verification that spatial power redistribution transients can be monitored and controlled is to be obtained in four consecutive stages of power testing in the overall plant startup program. These states of power testing are described in the following.

A. Steady-state calibration of power range instrumentation in which the out-of-core power range nuclear channels (using the long ion chambers), in-core core exit thermocouples, and primary loop resistance temperature detectors are calibrated on the bases of measured secondary heat balances and detailed in-core power distributions measured with the movable detector system. These instrumentation intercalibrations are repeated at several power levels of interest between 30% and 100% of full power in typical operating control rod configurations. The results of these steady-state measurements are analyzed and correlations developed between out-of-core detector response and in-core detector measurements of power peaking. Design operational curves are verified or appropriate adjustments made to ensure that design limits on power peaking are not exceeded. Instrumentation accuracies are evaluated in these tests.

B. Follow of spatial power redistribution transients in which spatial transients are initiated at a reduced constant power level by prescribed control rod maneuvers and the resultant changes in core power distribution are monitored in terms of axial and azimuthal power offsets (*Reference 3*) as indicated by the out-of-core power range nuclear detectors and of assembly-wise power sharing factors and gross power tilts as indicated by the in-core thermocouple system. Concurrent periodic measurements of the core power distribution made with the in-core movable detectors allow verification of the inter-calibrations of the out-of-core power range instrumentation under transient conditions and direct evaluation of nuclear hot-channel factors. Transient reactivity changes are met by adjustment of the reactor coolant boron concentration.

C. Controlled follow of spatial power redistribution transients in which spatial transients are initiated, as before, by control rod maneuvering at constant power and the resultant power peaking transients are suppressed by subsequent maneuvering of the part-length control rods by the operator. The maneuvering scheme for limiting local power peaking during the induced transients is to be the normal procedure prescribed for plant operation where successive control rod maneuvers are dictated by the current values of axial offset ratios derived from the out-of-core power range nuclear detector responses (for example see *Reference 3*). Concurrent periodic power distribution measurements made with the in-core movable detector system allow verification both of the values of limiting power distribution parameters as deduced from the out-of-core instrumentation responses and of the adequacy of the prescribed operating procedure for limiting power peaking during spatial power distribution transients.

D. Controlled follow of dynamic power redistribution transients in which the operation of the plant reproduces a typical load variation cycle, but at a reduced power level. Spatial power redistribution transients resulting from the associated power level changes and the attendant control rod maneuvers are monitored with the out-of-core nuclear detectors and core exit thermocouples and power peaking is by part-length control rod manipulation according to standard operating procedures. Concurrent detailed core power distribution measurements with the movable detector system are made to evaluate nuclear hot-channel factors and verify correlations with out-of-core instrumentation.

The results of the several stages of measurement and verification are reviewed for adequacy, before the next stage of testing is undertaken.

As burnup of the core progresses, test 1 will be repeated at regular intervals under typical operating conditions in accord with normal operating practice. At less frequent intervals test 2 and test 4 during a normal load variation cycle, including in both cases comprehensive detailed power distribution measurements made with the moveable detector system, will be repeated to allow assessment of the effects of core depletion.

1.5.4 DEVELOPMENT OF LONG ION CHAMBERS

This plant uses four long ion chambers, mounted vertically outside the reactor pressure vessel for power range nuclear instrumentation. The chambers are 90 degrees apart in plan; each chamber has an active length of 10 ft with its center level with the core horizontal midplane.

Each chamber is split into an upper and lower section to effectively form two uncompensated ion chambers of equal size.

One purpose of these long ion chambers in this plant is to detect axial power redistributions when they occur, and any transverse power tilts that could arise if control rods become malpositioned. The efficiency of these out-of-core long ion chambers in accurately reflecting in-core power distribution is shown in *Reference 3*. Also, their long total active length minimizes differences in indicated core average power for the same actual power but different control rod positions.

This is the first U.S. plant to use uncompensated long ion chambers as standard instrumentation, but the design is similar in both size and configuration to chambers that have now been successfully tested over extended periods in similar reactor service. Four two-section (one section compensated, the other uncompensated) 8 ft. long ion chambers have been used on the SENA reactor as their standard instrumentation for about four months. An 8 ft. long two section ion chamber, similar to the Ginna design, was tested on the Carolinas Virginia Tube Reactor for about 12 months. This chamber was then transferred to the San Onofre reactor where it has had about 15 months operation. In addition, a long ion chamber, identical to those to be fitted on Ginna, was installed for testing at San Onofre in September 1968.

From this design, manufacturing, and test experience of long ion chambers, it is expected that the long ion chambers for this plant will perform satisfactorily.

1.5.5 CONTROL ROD EJECTION AND DROPPED CONTROL ROD ACCIDENT ANALYSES

The ejection of a control rod from the core would require the failure of its control rod mechanism housing. Although such a failure is not considered credible, single control rod ejection analyses using the final core design parameters, including abnormal conditions that could occur during plant operation and tolerances for instrumentation error and reactivity coefficient, have been completed. The four cases analyzed are zero and full power; beginning and end of core life. These show that no consequential damage to the reactor coolant system will occur under these adverse conditions.

This plant core was initially designed to use only movable absorber rods and chemical shim to control reactivity, but will now, in addition, have burnable poison rods installed. The consequences of a rod ejection accident are inherently limited in a core with chemical shim control since the amount of rod insertion is limited to that necessary to change load, while the chemical shim concentration is adjusted to compensate for fuel burnup. The addition of the burnable poison rods now also ensures that the moderator coefficient of reactivity is negative throughout core life at operating temperature, further reducing the consequences of an ejection accident. The research and development program on the burnable poison rods is discussed in Section 1.5.3.

The consequences of dropping single full-length control rods have been analyzed. Either the actual rod drop or its resultant effects on local power and flux distribution will be detected, and action to protect the core and coolant system against damage is automatic. This protection includes blocking control rod withdrawal.

1.5.6 CHARCOAL FILTERS

At the time the plant was proposed, it appeared that further development work would be required to prove the effectiveness of impregnated activated charcoal filters in removing radioactive iodine in both organic (methyl iodine) and inorganic (elemental) forms.

Tests on the extraction of methyl iodide by full-size charcoal filters were made in cooperation with the Connecticut Yankee Atomic Power Company for their Haddam Neck plant. These demonstrated the suitability of using iodized activated charcoal filters to remove radioactive methyl iodide from a containment environment under the most extreme conditions anticipated following a loss-of-coolant accident. The results of these tests (*Reference 4*) filed with the AEC under Docket No. 50-213 are applicable to the charcoal filter system employed in this plant.

Before any testing was started on the extraction of elemental iodine by the charcoal filters, a literature survey was made. This showed that sufficient experimental data was already available from other sources (*References 5 through 8*) to confirm that activated charcoal filters were even more efficient in extracting elemental iodine than methyl iodide under any typical post loss-of-coolant accident environmental conditions. It was therefore decided that tests for elemental iodine extraction were no longer necessary, and no further experiments were conducted. This conclusion that further research and development on elemental iodine extraction by charcoal filters was unnecessary was also expressed by the AEC staff at the Public Hearing in the matter of the Salem Nuclear Plant for the Public Service Electric and Gas Company, August 15, 1968, Docket Numbers 50-272 and 50-311.

The effectiveness of the charcoal filter units during plant use will be demonstrated by periodic tests at Haddam Neck and in this plant, as required by the Technical Specifications. These tests will determine if there is any need for filter replacement because of deterioration with time.

1.5.7 REACTOR COOLANT PUMP CONTROLLED LEAKAGE SEALS

The reactor coolant pump controlled leakage seal design for this plant has been fully developed. A full scale mock-up of this seal was operated for over 100 hr to confirm that seal deflection under load and leak rate are acceptable. These tests also showed that erosion and corrosion of the seal materials were not adversely affected by the slight increase in water velocity through the seal due to the increased seal size necessary to fit the larger shafts used in these pumps. A full-scale mock-up was used during the development of the controlled leakage seal to provide information on long-term performance and this life testing will continue.

One of the seals used in this plant was operated about 300 hr and the other about 100 hr, each in its pump motor unit. During hot functional testing in the plant, before the core is loaded, additional operation will bring the total operating time for each seal to well over 500 hours.

Successful operation of similar seals has been demonstrated with over 5000 hours total running time in San Onofre and over 3000 hours in Haddam Neck. More than 10 pumps have already been built for later plants and tested successfully for at least 100 hours each. The seals in these latter pumps are the same size as those used in this plant.

1.5.8 SAFETY INJECTION SYSTEM

1.5.8.l Development of Safety Injection System Design

The development effort on the Emergency Core Cooling System (ECCS) design has resulted in the modification of the system to include nitrogen pressurized accumulator tanks for rapid core reflooding with borated water. The accumulators are passive devices, and the only valves between them and their injection nozzles are swing check valves which open entirely automatically once the reactor coolant system pressure falls. The increased flooding capability limits the clad temperature after a loss-of-coolant accident to well below the melting temperature of Zircaloy-4, minimizes metal-water reaction, and ensures that the core remains in place and intact, thereby ensuring preservation of essential heat transfer geometry. The system design incorporates redundancy of components such that the minimum required water addition rates can be met assuming any active component to fail concurrent with the loss-of-coolant accident or, over the long-term period of post-accident core decay heat removal, a passive or active component failure in either the safety injection or service water systems, or an active failure in the component cooling water (CCW) system.

1.5.8.2 Development of Core Cooling Analysis

The loss-of-coolant analysis presented in the PSAR was based on a one-element code (LOCO) for the blowdown and reflooding portions of the transients. For the FSAR a more detailed blowdown code (FLASH) was used. The FLASH code divides the reactor coolant system into three regions. This division provides for a more precise description of the blowdown process, and in particular for the input to the reactor kinetics and core cooling analysis.

The FLASH code has been compared to many blowdown experiments primarily those performed at LOFT. It has been demonstrated that the code is conservative in two principal areas: rate of depressurization and mass of water left after blowdown. The FLASH code was required to analyze the performance of the improved Emergency Core Cooling System (ECCS) for large area ruptures.

The LOCTA-R2 transient digital computer program was developed during the final design of the Ginna reactor for evaluating fuel pellet and cladding temperatures during a loss-of-coolant accident.

The code is able to stack axial sections and thereby describe the behavior of a full-length region as a function of time. A mass and energy balance is used in evaluating the temperature rise in the steam as it flows through the core.

The present code is a more sophisticated version of LOCTA-R which was used in the loss-of-coolant accident analyses reported in the PSAR. LOCTA-R was able to describe the behavior of only one axial location on the rod while holding the environmental sink temperature constant throughout the accident.

The SLAP code has replaced LOCO for predicting the entire blowdown and reflooding characteristics of the smaller ruptures. The SLAP code is essentially an extension of the LOCO code, but it provides a better description of the transient on the steam generator shell side and the heat transferred between the reactor and steam generator during blowdown.

For the smaller breaks it is important to determine if departure from nucleate boiling occurs during blowdown. The SATAN-R and THINC codes were used for this purpose. Core parameters obtained from SATAN-R, such as pressure, power, and flow, were used as input to the THINC code. The THINC code is used to calculate coolant density, mass velocity, enthalpy, vapor voids, and static pressure distribution along parallel flow channels in the core.

Extensive work on the development of these new models was completed during the final design of the Ginna reactor.

1.5.9 DEVELOPMENT OF DESIGN, INSPECTION, AND ACCEPTANCE CRITERIA FOR PRESTRESSED REINFORCED-CONCRETE PRESSURE VESSELS

At the time Ginna Station was proposed, the unusual feature of the steel-lined reinforced-concrete reactor containment vessel was the use of post-tensioned prestressing tendons, although their use in construction is well proven. The developments and tests discussed below are therefore confined to those elements directly applicable to the prestressing of the containment vessel. These are:

- Rock anchor design criteria and test results.
- Rock anchor grout.
- Tendon inspection and acceptance criteria.
- Tendon corrosion protection system.

These topics are discussed in more detail below.

1.5.9.1 Rock Anchors

<u>1.5.9.1.1 Design Criteria and Assumptions</u>

The basic criterion in determining the length of rock anchors necessary to develop adequate hold-down capacity, is that the pull of the anchor is resisted only by the submerged weight of rock. The assumptions are made that (1) the rock has no tensile strength, (2) it breaks out at an angle of 45 degrees to the vertical, with the depth taken to the midpoint of the bond development length, and (3) the bond-stress between rock and grout is 170 psi.

<u>1.5.9.1.2 Test Verification and Results</u>

These assumptions and their historical justification are discussed in Section 3.8.1.4.2. In order to determine the factors of safety represented by these assumptions for the conditions pertaining to this plant site, a series of tests were carried out on three scaled-down test anchors, to demonstrate rock hold-down capacity and bond strength between grout and rock.

These tests and results are described in Section 3.8.1.7.

1.5.9.2 Rock Anchor Grout

Grouting techniques used followed closely those developed by the Swiss parent company of the BBRV system. The grout used is a mix of 5 gallons of water to one bag of cement, with 1 lb of a special BBRV additive. The latter, designed to reduce the water requirements of the

cement (and so retard the setting time), also provides a controlled expansion of the grout of about 8%, accomplished by the reaction of an aluminum powder with the alkalies of the cement. The additive is free from chlorides, sulfides, and other salts whose presence could possibly create a corrosion problem. The cement used is non-air entraining, Type II.

A test was carried out at the site to verify the grout application procedure and to ensure cohesion and hardening of the grout, even when pumped under water.

1.5.9.3 Tendon Inspection and Acceptance Criteria

Buttonhead dimensional accuracy and symmetry are important to ensure maximum development of both the rock anchor and wall tendon strength. Consistency of length of tendon wires is necessary to ensure uniform load distribution on individual wire elements. Uniformity of material properties is important in obtaining correct tendon characteristics compatible with those assumed for analysis, i.e., ductility and ultimate and yield strengths.

The acceptance criteria and the program to ensure conformity with these were developed after inspection of the fabricator’s initial production runs and are outlined in Section 3.8.1.6.7.

1.5.9.4 Wall Tendons

1.5.9.4.1 Corrosion Protection

The use of unbonded tendons gives, in addition to other advantages, accessibility for inspection or replacement. However, because the tendons are not in intimate and integral contact with surrounding concrete, the advantage of the high alkaline environment generally considered to promote adequate corrosion protection is lost. Therefore, these tendons must be provided with a corrosion preventive medium that gives protection equivalent to concrete, but still enables withdrawal of a tendon for inspection or replacement.

Consequently, one of the more important programs in connection with the tendons has been the selection of a complete corrosion protection system. The various elements involved are (1) a cathodic protection system in which all tendons are connected to the liner and then to a copper grounding system which is completed by the addition of reference cells and anodes, from which a protective potential can be generated if the need for cathodic protection is indicated by the reference cells, (2) a steel conduit surrounding each tendon providing shielding against stray electrical currents, (3) temporary shipping and erection protection of all wires in each tendon, by the application of a coating, followed by complete filling of each tendon conduit with a petroleum base wax, NO-OX-ID "CM," that provides a permanent, chemically stable environment for protection from corrosion, while still giving flexibility of withdrawal for inspection. The selection, testing, and application of the coating and wax was an important program in the development of the overall corrosion protection system. Tests at the W. R. Grace & Company Dearborn Division Research Center are outlined below.

Two tendon mock-up test rigs were set up for evaluation of individual wire coverage by the wax and for determination of pumping characteristics. One test rig consisted of a transparent pyrex glass tube test section containing a tendon section through which the wax could be circulated. Tests showed that as the wax moved through the test section it completely immersed all the wires, even though some were tightly bunched together. Subsequent inspection of

individual wires showed complete coverage. In a second test, a quantity of water was introduced into the test section and pumping started. The water "plug" was driven ahead of the wax, which preferentially wetted all the wires. There appeared to be no diffusion or mixing of the water into the wax.

A second test rig consisted of a 20 ft. high conduit section containing a short-length tendon, complete with all anchor heads and hardware. This was used to determine pump pressures for circulation under ambient conditions, flow rates, and friction losses.

Specimens coated with both the initial coating and the wax were compared to uncoated control plates under extreme conditions of continuous exposure to salt water, steam, relative humidity, and temperature in environmental testing cabinets. Results obtained after many hundreds of hours showed no deterioration of the coated specimens.

1.5.9.4.2 Inspection and Acceptance

Preoperational testing on the complete containment is discussed in Section 3.8.1.7.1.

1.5.10 DEVELOPMENT OF CONTAINMENT HYDROGEN RECOMBINER

Following a major loss-of-coolant accident in the Ginna Station reactor, hydrogen may be generated inside the containment by the mechanisms of radiolysis, zirconium-water reaction, and the reaction of alkaline spray solution with aluminum. Because of the high level of radioactivity in the containment which may also result from the accident, the containment must be sealed for an extended period to prevent the spread of contamination to the environment.

Under these circumstances, if the containment isolation is sufficiently long, the possibility of hydrogen reaching a flammable concentration of 4.1 volume percent in air must be considered. Equipment was therefore provided for the controlled recombination of hydrogen at a concentration. The system selected is a flame combustor using containment atmosphere (containing a low concentration of hydrogen) as primary oxidant and supplemental hydrogen as a fuel. The product of combustion, water vapor, is cooled and condensed from the atmosphere by the vital cooling systems of the containment. Operation of the system will control buildup of hydrogen to less than 2 volume percent or one-half of the lower flammable limit.

Inside the containment are two complete combustor systems, one a spare. Each system consists of a blower to circulate containment air to the combustor, a combustion chamber complete with main burner, two igniters (one a spare), pilot burner, and a dilution chamber downstream of the flame zone where products of combustion are mixed with a large excess of containment air to reduce the temperature of gas leaving the system.

Testing of a recombiner system will be used to:

- Demonstrate that the design is sound (proof testing).
- Determine certain limits for the combustor in performance.

A description of the recombiner and the research, development, and test program is discussed in more detail in *Reference 9*.

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1.7 DRAWINGS AND OTHER DETAILED INFORMATION

1.7.1 ELECTRICAL, INSTRUMENTATION, AND CONTROL DRAWINGS

Updated electrical drawings, schematics, logic diagrams, and elementary wiring diagrams were submitted to the NRC during the Systematic Evaluation Program (SEP) as necessary to permit the staff to review the safety-related aspects of Ginna Station.

Drawings representing the electrical, instrumentation, and control systems are referenced throughout the UFSAR.

A list of electrical, instrumentation, and control drawings, which previously were included as figures in earlier revisions of the UFSAR, is given in Table 1.7-1.

1.7.2 PIPING AND INSTRUMENTATION DIAGRAMS (P&ID)

Updated piping and instrumentation diagrams were submitted to the NRC during the SEP as necessary to permit the staff to review the safety-related aspects of Ginna Station.

Drawings representing the piping and instrumentation diagrams are referenced throughout the UFSAR. A list of piping and instrumentation diagrams, which previously were included as figures in earlier revisions of the UFSAR, is given in Table 1.7-2. The legend for symbols used in these diagrams is included in Drawing 33013-2242, Sheets 1-4.

1.7.3 OTHER DETAILED INFORMATION

References to detailed information submitted to the NRC are incorporated in the appropriate sections throughout the UFSAR and are not duplicated in this section.

Table 1.7-1
ELECTRICAL, INSTRUMENTATION, AND CONTROL DRAWINGS

Drawing Number	Title	Historical Link to UFSAR Figure Number
03201-0102	120-Volt AC Instrument Bus One-Line Diagram	8.3-4
03202-0102	One-Line Diagram, 125-Volt DC System	8.3-6
33013-623		
Sheet 1	Main One-Line Diagram	8.3-1, Sheet 1
Sheet 2	Main One-Line Diagram	8.3-1, Sheet 2
33013-652	480-Volt One-Line Diagram	8.3-3
33013-653	4160-Volt One-Line Diagram	8.3-2
33013-1353		
Sheet 1	Logic Diagram, Index and Symbols	7.2-3
Sheet 2	Logic Diagram, Reactor Trip Signals	7.2-4
Sheet 3	Logic Diagram, Turbine Trip Signals	7.2-9
Sheet 4	Logic Diagram, Electrical Protection Logic	7.2-8
Sheet 5	Logic Diagram, Emergency Diesel Generator Startup Logic	8.3-5
Sheet 6	Logic Diagram, Safeguards Actuation Signals	7.3-1, Sheet 1
Sheet 7	Logic Diagram, Safeguards Actuation Signals	7.3-1, Sheet 2
Sheet 8	Logic Diagram, Safeguards Sequence	7.3-3
Sheet 9	Logic Diagram, Feedwater Isolation and Auxiliary Feedwater Pump Actuation Signals	7.3-2
Sheet 10	Logic Diagram, Nuclear Instrumentation Trip Signals	7.2-6
Sheet 11	Logic Diagram, Nuclear Instrumentation Permissives, and Blocks	7.2-11
Sheet 12	Logic Diagram, Pressurizer Trip Signals	7.2-7
Sheet 13	Logic Diagram, Steam Generator Trip Signals	7.2-10
Sheet 14	Logic Diagram, Reactor Coolant System Trip Signals	7.2-5
Sheet 15	Logic Diagram, Rod Stops and Turbine Runbacks	7.7-5

Table 1.7-2
PIPING AND INSTRUMENTATION DIAGRAMS (P &ID)

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1231	Main Steam System (Safety Related) - P&ID	10.3-1
33013-1232	Main Steam System (Non Safety Related) - P&ID	10.3-2
33013-1233	Condensate Low Pressure Feedwater Heaters - P&ID	10.4-3
33013-1234	Condensate Storage System - P&ID	10.7-5
33013-1235	Condensate System (Condensate Booster Pumps to Hydrogen Coolers and Blowdown Recovery System) - P&ID	10.4-2
33013-1236		
Sheet 1	Feedwater System - P&ID	10.4-4, Sheet 1
Sheet 2	Feedwater System - P&ID	10.4-4, Sheet 2
33013-1237	Auxiliary Feedwater System - P&ID	10.5-1
33013-1238	Standby Auxiliary Feedwater System - P&ID	10.5-2
33013-1239		
Sheet 1	Diesel Generator "A" Supporting Systems - P&ID	9.5-5, Sheet 1
Sheet 2	Diesel Generator "B" Supporting Systems - P&ID	9.5-5, Sheet 2
33013-1242	Fire Protection System - Relay and Computer (MUX) Rooms - P&ID	9.5-3
33013-1245	Component Cooling Water System - P&ID	9.2-4, Sheet 1
33013-1246		
Sheet 1	Component Cooling Water System - P&ID	9.2-4, Sheet 2
Sheet 2	Component Cooling Water System - P&ID	9.2-4, Sheet 3
33013-1247	Residual Heat Removal System - P&ID	5.4-7
33013-1248	Spent Fuel Pool Cooling System - P&ID	9.1-6
33013-1250		
Sheet 1	Service Water System, Safety Related - P&ID	9.2-1, Sheet 1
Sheet 2	Service Water System, Safety Related - P&ID	9.2-1, Sheet 2
Sheet 3	Service Water System, Safety Related - P&ID	9.2-1, Sheet 3

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1251		
Sheet 1	Service Water System, Non Safety Related - P&ID	9.2-2, Sheet 1
Sheet 2	Service Water System, Non Safety Related - P&ID	9.2-2, Sheet 2
33013-1252	Condensate System - P&ID	10.4-1
33013-1256	Technical Support Center HVAC System - P&ID	9.4-17
33013-1258	Reactor Coolant Pressurizer - P&ID	5.1-1, Sheet 2
33013-1259	Miscellaneous Liquid Waste Disposal - P&ID	11.2-1
33013-1260	Reactor Coolant - P&ID	5.1-1, Sheet 1
33013-1261	Containment Spray - P&ID	6.2-11
33013-1262		
Sheet 1	Safety Injection and Accumulators - P&ID	6.3-1, Sheet 1
Sheet 2	Safety Injection and Accumulators - P&ID	6.3-1, Sheet 2
33013-1263	Reactor Coolant System Overpressure Protection, Nitrogen Accumulator System - P&ID	5.2-1
33013-1264	Chemical and Volume Control, Letdown - P&ID	9.3-14
33013-1265		
Sheet 1	Chemical and Volume Control, Charging - P&ID	9.3-13, Sheet 1
Sheet 2	Chemical and Volume Control, Charging - P&ID	9.3-13, Sheet 2
33013-1266	Chemical and Volume Control, Boric Acid - P&ID	9.3-15
33013-1267	Chemical and Volume Control, Holdup Tanks to Gas Strippers - P&ID	9.3-18
33013-1268	Chemical and Volume Control, Boric Acid Evaporator to Monitor Tanks - P&ID	9.3-17
33013-1269	Chemical and Volume Control, Reactor Makeup Water System - P&ID	9.3-16
33013-1270		
Sheet 1	Waste Disposal - Liquid, Waste Drains, Holdup Tank, Spent Resin Tanks - P&ID	11.2-3, Sheet 1
Sheet 2	Waste Disposal - Liquid, Waste Drains, Holdup Tank, Spent Resin Tanks - P&ID	11.2-3, Sheet 2
33013-1271	Waste Disposal - Liquid, Waste Condensate Tanks - P&ID	11.2-4

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1272		
Sheet 1	Waste Disposal - Liquid, Reactor Coolant Drain Tank - P&ID -	11.2-2, Sheet 1
Sheet 2	Waste Disposal - Liquid, Reactor Coolant Drain Tank - P&ID	11.2-2, Sheet 2
33013-1273		
Sheet 1	Waste Disposal - Gas - P&ID	11.3-2, Sheet 1
Sheet 2	Waste Disposal - Gas - P&ID	11.3-2, Sheet 2
33013-1274	Waste Disposal - Gas, H_2 and N_2 and Gas Analyzer - P&ID	11.3-1
33013-1275		
Sheet 1	Waste Disposal - Gas, Hydrogen Recombiner - P&ID	6.2-79, Sheet 1
Sheet 2	Waste Disposal - Gas, Hydrogen Recombiner - P&ID	6.2-79, Sheet 2
33013-1276	Waste Disposal - Liquid, Polishing Demineralizers - P&ID	11.2-5
33013-1277		
Sheet 1	Steam Generator Blowdown - P&ID	10.7-6, Sheet 1
Sheet 2	Steam Generator Blowdown - P&ID	10.7-6, Sheet 2
33013-1278		
Sheet 1	Nuclear Sampling System - P&ID	9.3-10, Sheet 1
Sheet 2	Nuclear Sampling System - P&ID	9.3-10, Sheet 2
33013-1279	Postaccident Sampling System - P&ID	9.3-12
33013-1607	Fire Protection System Yard Loop - P&ID	9.5-4
33013-1863	Containment HVAC Systems, Containment Recirculating and Cooling System, Postaccident Charcoal Filters - P&ID	9.4-1
33013-1864	Containment HVAC Systems, Containment Auxiliary Charcoal Filters, Refueling Water Ventilation, Reactor Compartment and Control Rod Drive Cooling - P&ID	9.4-2
33013-1865	Containment HVAC Systems, Purge Supply - P&ID	9.4-3
33013-1866	Containment HVAC Systems, Purge Exhaust and Penetration Cooling - P&ID	9.4-4

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1867	Control Building HVAC System, Control Room HVAC Control Room Postaccident Charcoal Filters, Control Room Lavatory Exhaust - P&ID	6.4-1
33013-1868	Control Building HVAC System, Relay Room Cooling, Battery Room Cooling and Ventilation - P&ID	9.4-18
33013-1869	Auxiliary/Intermediate Building HVAC Systems Cooling for Charging, Safety Injection, Containment, Spray, RHR, and Standby Auxiliary Feedwater Pumps, Nitrogen and Hydrogen Storage Vents - P&ID	9.4-8
33013-1870	Auxiliary/Intermediate Building HVAC Systems, Volume Control Tank Exhaust, Auxiliary Building Charcoal Filter, Auxiliary Building 1G Filter - P&ID	9.4-7
33013-1871	Auxiliary/Intermediate Building HVAC Systems, Intermediate Building Exhaust System, Spent Fuel and Decon Pit Exhaust System, Main Auxiliary Building Exhaust System - P&ID	9.4-6
33013-1872	Auxiliary/Intermediate Building HVAC Systems, Building Supply Air Systems - P&ID	9.4-5
33013-1873	Turbine/Miscellaneous Building HVAC Systems, Ventilation for Diesel Generators, Feed Pumps, Oil Storage, Turbine Building Gas Bottle Storage, Elevator, and Screen House - P&ID	9.4-9
33013-1874	Turbine/Miscellaneous Building HVAC Systems, Condensate Demineralizer (AVT) Building Ventilation - P&ID	9.4-10
33013-1875	Service Building HVAC Systems, Controlled Access Exhaust System and Air Handling Unit 1C - P&ID	9.4-11
33013-1876	Service Building HVAC Systems, Air Handling Units 1B and 1D - P&ID	9.4-12
33013-1877	Service Building HVAC Systems, Air Handling Unit 1A and Return Air Fan 1A - P&ID	9.4-15
33013-1878	Service Building HVAC Systems, Miscellaneous Service Building HVAC Systems - P&ID	9.4-16
33013-1879	Service Building HVAC Systems, Air Handling Unit 1E - P&ID	9.4-13
33013-1881	Service Building HVAC Systems, Service Building North End HVAC System - P&ID	9.4-14

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1885		
Sheet 1	Circulating Water - P&ID	10.6-1, Sheet 1
Sheet 2	Circulating Water - P&ID	10.6-1, Sheet 2
33013-1886		
Sheet 1	Service Air System - P&ID	9.3-1, Sheet 1
Sheet 2	Service Air System - P&ID	9.3-1, Sheet 2
33013-1887	Instrument Air, Containment Building - P&ID	9.3-3, Sheet 1
33013-1888	Instrument Air, Containment Building - P&ID	9.3-3, Sheet 2
33013-1889	Instrument Air, Auxiliary Building - P&ID	9.3-4, Sheet 1
33013-1890	Instrument Air, Auxiliary Building - P&ID	9.3-4, Sheet 2
33013-1891	Instrument Air, Auxiliary Building - P&ID	9.3-4, Sheet 3
33013-1892	Instrument Air, Auxiliary Building - P&ID	9.3-4, Sheet 4
33013-1893	Instrument Air, Intermediate Building - P&ID	9.3-5
33013-1894		
Sheet 1	Instrument Air, Turbine Building - P&ID	9.3-6, Sheet 1
Sheet 2	Instrument Air, Turbine Building - P&ID	9.3-6, Sheet 2
33013-1895	Instrument Air, Turbine Building - P&ID	9.3-6, Sheet 3
33013-1896	Instrument Air, Turbine Building and Screen House - P&ID	9.3-7
33013-1897		
Sheet 1	Instrument Air, Condensate Demineralizer (AVT) Building - P&ID	9.3-8, Sheet 1
Sheet 2	Instrument Air, Condensate Demineralizer (AVT) Building - P&ID	9.3-8, Sheet 2
33013-1898	Instrument Air, Service Building - P&ID	9.3-9, Sheet 3
33013-1899		
Sheet 1	Instrument Air, Service Building - P&ID	9.3-9, Sheet 1
Sheet 2	Instrument Air, Service Building - P&ID	9.3-9, Sheet 2
33013-1900		
Sheet 1	Instrument Air Compressors - P&ID	9.3-2, Sheet 1
Sheet 2	Instrument Air Compressors - P&ID	9.3-2, Sheet 2

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1901	Turbine Lube-Oil System - P&ID	10.7-9
33013-1903	Extraction Steam - P&ID	10.7-3
33013-1904	Turbine Gland Steam and Drains - P&ID	10.7-7
33013-1905	Gland Sealing Water - P&ID	10.4-5
33013-1907	Primary Water Treatment Chemical Supply Tanks - P&ID	9.2-5
33013-1908		
Sheet 1	Primary Water Treatment - P&ID	9.2-6, Sheet 1
Sheet 2	Primary Water Treatment - P&ID	9.2-6, Sheet 2
Sheet 3	Primary Water Treatment - P&ID	9.2-6, Sheet 3
33013-1909	Ammonia Addition and Secondary Plant Water Treatment - P&ID	10.7-10
33013-1910		
Sheet 1	Condensate Demineralizer Regeneration System - P&ID	10.7-12, Sheet 1
Sheet 2	Condensate Demineralizer Regeneration System - P&ID	10.7-12, Sheet 2
33013-1911		
Sheet 1	Condensate Demineralizer Service Vessels - P&ID	10.7-11, Sheet 1
Sheet 2	Condensate Demineralizer Service Vessels - P&ID	10.7-11, Sheet 2
33013-1912	Condensate Demineralizer Regeneration Waste Handling - P&ID	10.7-13
33013-1918		
Sheet 1	Moisture Separator Reheater System - Steam - P&ID	10.3-3, Sheet 1
Sheet 2	Moisture Separator Reheater System - Steam - P&ID	10.3-3, Sheet 2
33013-1919		
Sheet 1	Moisture Separator Reheater System - Drains - P&ID	10.3-4, Sheet 1
Sheet 2	Moisture Separator Reheater System - Drains - P&ID	10.3-4, Sheet 2
33013-1921	Condenser Air Removal and Priming - P&ID	10.7-8
33013-1922	Feedwater Heater Vents, Relief and Miscellaneous Drains - P&ID	10.7-2
33013-1923	Feedwater Heater Drain System - P&ID	10.7-1

Drawing Number	Title	Historical Link to UFSAR Figure Number
33013-1924	Extraction Steam - 1, 2, and 3 Heaters and Drains - P&ID	10.7-4
33013-1925	Service Water for Instrument Air Compressors - P&ID	9.2-3
33013-1989	Fire Protection Systems Fire Service Water, Plant Systems - P&ID	9.5-1
33013-1990		
Sheet 1	Fire Protection Systems - Fire Service Water, Turbine Building and Technical Support Center - P&ID	9.5-2, Sheet 1
Sheet 2	Fire Protection Systems - Fire Service Water, Turbine Building and Technical Support Center - P&ID	9.5-2, Sheet 2
33013-1991	Fire Protection Systems - Fire Service Water Auxiliary Building, Intermediate Building, Containment Building - P&ID	9.5-2a
33013-1992	Fire Protection Systems - Fire Service Water Fire Water Header "A", Auxiliary Building Header 1G Charcoal Filter - P&ID	9.5-2b
33013-1993		
Sheet 1	Fire Protection Systems Fire Service Water, Header "B" - P&ID	9.5-2c, Sheet 1
Sheet 2	Fire Protection Systems Fire Service Water, Header "B" - P&ID	9.5-2c, Sheet 2
33013-2242		
Sheet 1	Symbol Legend - P&ID	1.7-1, Sheet 1
Sheet 2	Symbol Legend - P&ID	1.7-1, Sheet 2
Sheet 3	Symbol Legend - P&ID	1.7-1, Sheet 3
Sheet 4	Symbol Legend - P&ID	1.7-1, Sheet 4
33013-2711		
Sheet 1	Secondary Sampling System - P&ID	9.3-11, Sheet 1
Sheet 2	Secondary Sampling System - P&ID	9.3-11, Sheet 2
Sheet 3	Secondary Sampling System - P&ID	9.3-11, Sheet 3
Sheet 4	Secondary Sampling System - P&ID	9.3-11, Sheet 4

1.8 CONFORMANCE TO NRC REGULATORY GUIDES

1.8.1 CONFORMANCE TO AEC SAFETY GUIDES

The information in this section represents the position of the R. E. Ginna Nuclear Power Plant in August 1972 at the time when RG&E applied for a Full-Term Operating License with respect to the AEC Safety Guides for Water Cooled Nuclear Power Plants, numbers 1 through 29. The information has not been generally updated. It has been revised to remove incorrect or misleading information. References to sections and figures refer to this UFSAR unless the references are to the original FSAR, in which case it is so stated and the referenced information has not been incorporated into the UFSAR.

1.8.1.1 Safety Guide 1 - Net Positive Suction Head for Emergency Core Cooling and Containment Heat Removal System Pumps

The net positive suction head (NPSH) of the residual heat removal pumps is evaluated for normal plant shutdown operation and for both the injection and recirculation phase operations of the design-basis accident. Recirculation operation gives the limiting NPSH requirements and the NPSH available is determined from the containment water level, the temperature and pressure of the sump water, and the pressure drop in the suction piping from the sump to the pumps.

The NPSH for the safety injection pumps is evaluated for both the injection and recirculation phase operations of the design-basis accident. The end of injection phase operation gives the limiting NPSH requirement and the NPSH available is determined from the elevation head and vapor pressure of the water in the refueling water storage tank (RWST) and the pressure drop in the suction piping from the tank to the pumps.

The NPSH for the containment spray pump is evaluated for both the injection and recirculation phase operations of the design-basis accident. The end of the injection phase operation gives the limiting NPSH requirement and the NPSH available is determined from the elevation head and vapor pressure of the water in the refueling water storage tank (RWST) and the pressure drop in the suction piping from the tank to the pumps.

1.8.1.2 Safety Guide 2 - Thermal Shock to Reactor Pressure Vessels

The effects of safety injection water on the integrity of the reactor vessel following a postulated loss-of-coolant accident have been analyzed using data on fracture toughness of heavy section steel both at beginning of plant life and after irradiation corresponding to approximately 40 years of equivalent plant life. The results show that under the postulated accident conditions, the integrity of the reactor vessel is maintained.

Fracture toughness data are obtained from a Westinghouse experimental program which is associated with the Heavy Section Steel Technology (HSST) Program at Oak Ridge National Laboratory and Euratom programs. Since results of the analyses are dependent on the fracture toughness of irradiated steel, efforts are continuing to obtain additional confirmatory data. Data on 2 in. thick specimens became available in 1970 from the HSST Program. This data indicated a strong temperature dependence with a rapid increase in toughness at approximately nil ductility temperature. Presently, 4 in. thick specimens are being irradiated and

these will be tested in the spring of 1974. The HSST Program is scheduled for completion by 1974, at which time the reactor vessel thermal shock program will have been completed.

A detailed analysis considering the linear elastic fracture mechanism method, along with various sensitivity studies, was submitted to the AEC staff and members of the Advisory Committee on Reactor Safety.

Revised material for this report plus additional analysis and fracture toughness data were presented at a meeting with the Containment and Component Technology Branch on August 9, 1968, and forwarded by letter for AEC review and comment on October 29, 1968.

The analysis for the pressurized water reactor under the postulated conditions of Safety Guide 2 shows that no thermal shock problem exists. It is not anticipated that the continuing HSST Program will lead to any new conclusions about reactor vessel integrity under loss-of-coolant accident conditions.

1.8.1.3 Safety Guide 3 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Loss-of-Coolant Accident for Boiling Water Reactors

This safety guide is not applicable to the R. E. Ginna Nuclear Power Plant which is a pressurized water reactor.

1.8.1.4 Safety Guide 4 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Loss-of-Coolant Accident for Pressurized Water Reactors

Safety Guide 4 gives the assumptions used by the AEC to evaluate the design basis loss-of-coolant accident. This methodology was used by RG&E at that time to perform loss-of-coolant accident analyses. Current information is provided in Chapter 15.

1.8.1.5 Safety Guide 5 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Steam Line Break Accident for Boiling Water Reactors

This safety guide is not applicable to the R. E. Ginna Nuclear Power Plant which is a pressurized water reactor.

1.8.1.6 Safety Guide 6 - Independence Between Redundant Standby (Onsite) Power Sources and Between Their Distribution Systems

The electrically powered safety systems are divided into two groups so that loss of either one will not prevent safety functions from being performed.

Each ac load group has a connection to the preferred (offsite) power source. In a situation where offsite power is not available, two diesel generators supply standby power to separate redundant load groups. There is no automatic connection between either the diesel generators or the load groups.

The dc system consists of two separate batteries, each connected to two battery chargers, which supply separate dc load groups. The Ginna design includes automatic transfers between the load groups. However, necessary fusing and electrical interlocks are provided to prevent paralleling of the two dc systems.

Routing and separation standards applicable to existing cables are those that were invoked at the time of cable installation. For more information, see Section 8.3.1.4.

1.8.1.7 Safety Guide 7 - Control of Combustible Gas Concentrations in Containment Following a Loss-of-Coolant Accident

Two hydrogen recombiner units are installed in the Ginna containment. The purpose of these units is to prevent the uncontrolled post-accident buildup of hydrogen concentrations in the containment.

The recombiner system consists of two full-rated subsystems, each capable of maintaining the ambient H_2 concentration at 2% by volume. Each subsystem contains a combustor, fired by an externally supplied fuel gas, employing containment air as the oxidant. Hydrogen in the containment air is oxidized in passing through the combustion chamber. Hydrogen gas is also used as the externally supplied fuel in order that noncondensable combustion products are avoided which would cause a progressive rise in containment pressure. Oxygen gas is made up through a separate containment feed to prevent depletion of 0_2 below the concentration required for stable operation of the combustor.

Each recombiner is equipped with an air supply blower to deliver primary combustion air and quench air to reduce the unit exhaust temperature, an ignition system, and associated monitoring and control instrumentation. The system is qualified to perform its function in a post-accident environment.

1.8.1.8 Safety Guide 8 - Personnel Selection and Training

Personnel selection and training for Ginna Station were completed before ANSI-18.1, Proposed Standards for Selection and Training of Personnel for Nuclear Power Plants, was published. However, the existing personnel and positions conformed very closely with the requirements of ANSI-18.1. Since that time, selection of personnel, their qualifications, training, and retraining have been done to conform to ANSI-18.1-1971 and subsequent regulatory guides.

1.8.1.9 Safety Guide 9 - Selection of Diesel-Generator Set Capacity for Standby Power Supplies

The diesel-generator capacities were based on a conservative evaluation of power requirements in the event of a loss-of-coolant accident simultaneous with a loss of station reserve power supply.

Each of the generators has a nameplate continuous rating of 1950 kW with a 0.8 power factor at 900 rpm with three-phase, 60-cycle, 480-V operation. The units also have extended ratings of 2300 kW for 0.5 hr. and 2250 kW for 2 succeeding hours. While paragraph 2 of the Safety Guide regulatory position does not specifically apply to the load ratings of the Ginna diesels, it does indicate the desired conservatism. During the initial injection phase, which lasts less than 2.5 hr., the power requirement is less than 90% of the 2-hr limit of 2250 kW. Once this initial phase is completed, the power requirements are less than 95% of the continuous duty rating of the diesel.

During preoperational testing, the diesel was operated at the power levels specified above. The power required to run the safeguards loads under preoperational testing was less than that estimated because of the difficulties in simulating accident loads. The containment air, for instance, was less dense than that experienced in an accident and thus reduced the power loading. Because of this the diesel was tested at rated rather than actual load.

Both diesels are capable of starting, accelerating, and attaining rated voltage within 10 seconds of a loss of voltage on a safeguards bus. During testing, the loading sequence and timing has been checked and has performed satisfactorily. During this loading sequence, the voltage has not dropped below 75% of rated output and has returned to within 10% of rated voltage within 40% of the load sequence time interval. A load loss from 100% to zero power will not cause an overspeed trip of either diesel. Frequency checks during tests have not been addressed specifically, however, no unusual variations have been noticed.

The suitability of both diesels was confirmed through preoperational testing and in periodic testing done since that time.

1.8.1.10 Safety Guide 10 - Mechanical (Cadweld) Splices in Reinforcing Bars of Concrete Containments

Tension splices for bar sizes larger than #11 were made with Cadweld splice. To ensure the integrity of the Cadweld splice, the quality control provided for a random sampling of splices in the field. The selected splices were removed and tested to destruction. A sampling of splices was initially tested to destruction to develop an average ($\overline{X}$) and deviation (σ). Sufficient samples were tested to provide a 99% confidence level that 95% of the splices met the specification requirements. The distribution established permitted the development of the lower limit below which no test data should fall. If the result of any test fell below this limit, the subsequent or previous splice was sampled. If the result was above the lower limit, the process was considered to be in control. If this result was again below the lower limit, the process average was recalculated and an engineering investigation was required to determine the cause of the excess variation and to reestablish control the average of all tests was required to remain above the minimum tensile strength. As additional data became available, the average and standard deviation were updated. The actual frequency of testing carried out was one specimen for each 25 splices made for each crew for the first 250 splices made by that crew and one test for each 100 splices thereafter. In addition, where deformed bars were attached to structural steel members, specimens were made and tested to ensure that the weld of the splice to the member did not fail before the rebar or the splice. The frequency of testing these specimens was the same as that for the normal splice.

In sampling the Cadweld splices a test was concurrently performed on the rebar. Where the rebar failed prior to the splice, a check was provided on the ultimate strength of the rebar, thus providing a check on conformance with the manufacturer's certifications and the ASTM standards. In addition, certified mill test reports were received from the rebar supplier and checked for conformance with specification requirements.

Where the special large size bars (i.e., 14S and 18S) were spliced, the Cadweld process was used so that the connection could develop the required minimum ultimate bar strength. Where Cadweld splice was used, including in the cylinder and dome, the splices were staggered a minimum of 3 ft. An exception to this practice is in the vicinity of the large openings. Where reinforcing bars are anchored to plates or shapes, such as is the case for the dome bars anchored

into the cylinder and the interrupted hoop bars at penetrations, the Cadweld splices all occur on one plane. Lapped splices are detailed in accordance with ACI-63.

Where Cadweld splices were used to anchor reinforcing bars to a structural steel member, a procedure of testing coupons was used to demonstrate that the welding process was under control. This procedure required each welder to initially make coupons as qualification procedure. The procedure was repeated at a frequency of one coupon for each 100 production units. Each coupon required testing of two Cadweld connections.

In addition, the welding procedure complied with the specifications of the American Welding Society and provided for 100% visual inspection of welds.

1.8.1.11 Safety Guide 11 - Instrument Lines Penetrating Primary Reactor Containment

The containment pressure transmitter instrument lines penetrate the containment. These must be open following an accident, but have a manual isolation valve outside containment. Therefore, Safety Guide 11 is met as well as General Design Criteria 56 on another defined basis.

1.8.1.12 Safety Guide 12 - Instrumentation for Earthquakes

A strong motion accelerograph is installed at the Ginna plant and is located in the basement of the intermediate building. This location was chosen rather than the basement of the containment since it more easily facilitates periodic surveillance of the instrument (this would be difficult should the instrument be located in the basement of the containment), and the retrieval of the shock record can more readily be made.

The response of the accelerograph located in the basement of the intermediate building will be virtually the same as one located in the basement of the containment.

1.8.1.13 Safety Guide 13 - Fuel Storage Facility Design Basis

The spent fuel pool (SFP) is a reinforced-concrete structure with a seam-welded stainless steel plate liner. This structure is designed to withstand the anticipated earthquake loadings as a Seismic Category I structure so that the liner prevents leakage even in the event the reinforced concrete develops cracks.

All structures have been designed for wind loads in accordance with the requirements of the State of New York State Building Construction Code. The wind loads tabulated in this code are based on a design wind velocity of 75 mph at a height of 30 ft. above grade level. In addition, the spent fuel pool (SFP) has been evaluated with regards to tornado winds and missiles and found to be acceptable.

Interlocks have been provided on the auxiliary building crane to prevent the crane hook from passing over stored fuel and thus prevent heavy loads from being dropped on the spent fuel.

The area around the spent fuel pool (SFP) is enclosed by the auxiliary building. In addition to other ventilation systems in this building, a ventilation system is provided to provide a sweep

of air specifically across the top of the spent fuel pool (SFP). Originally, air was only passed through a high efficiency particulate air filter before being exhausted to the atmosphere. Early in 1971, however, a charcoal filter, to be placed into operation during MODE 6 (Refueling), was added to this discharge system to filter out the iodine in the air and thus improve the design to account for the assumption that all fuel rods in one fuel bundle might be breached if a MODE 6 (Refueling) incident occurred.

The fuel pool has been evaluated on the basis of dropping a fuel cask into the spent fuel pool (SFP). While some damage could possibly occur to the liner, the cask will not break through the reinforced concrete to cause a major leak. In any case, the crane moving the cask would be single-failure proof, thus precluding the need to postulate the cask drop occurrence.

There are no spent fuel pool (SFP) designs, permanently connected systems, and/or other features that by maloperation or failure could cause loss of fuel storage coolant to the extent that fuel would be uncovered. A maloperation or failure in the filtering or cooling systems will not cause the fuel to be uncovered.

The spent fuel pool (SFP) is provided with level monitoring equipment which gives an alarm in the control room if the level drops. The radiation level just above the spent fuel pool (SFP) is also monitored. A reading of this level is indicated locally and at the control room. A radiation level above the setpoint will cause an alarm on the control board. The filtering system associated with the air just above the spent fuel pool (SFP) is always in operation. Before being exhausted from the plant this air always passes through high efficiency particulate air filters first. During MODE 6 (Refueling) operations this air is also filtered with impregnated charcoal filters. The addition of the charcoal filters to the airstream is done manually.

A spent fuel pool (SFP) cooling system is installed to remove decay heat. Also, nonseismic makeup systems including the fire protection system, are provided to add coolant to the pool.

1.8.1.14 Safety Guide 14 - Reactor Coolant Pump Flywheel Integrity

Precautionary measures, taken to preclude missile formation from primary coolant pump components, ensure that the pumps will not produce missiles under any anticipated accident condition.

The primary coolant pumps run at 1189 rpm, and may operate briefly at overspeeds up to 109% (1295 rpm) during loss of outside load. For conservatism, however, 125% of operating speed was selected as the design speed for the primary coolant pumps. For the overspeed condition, which would not persist for more than 30 seconds, pump operating temperatures would remain at about the design value.

Each component of the primary pumps has been analyzed for missile generation. Any fragments would be expected to be contained by the heavy stator.

The most adverse operating condition of the flywheels is visualized to be the loss-of-load situation. The following conservative design and operation conditions minimize missile production by the pump flywheels. The flywheels are fabricated from rolled, vacuum-degassed, ASME SA 533 Type B steel plates. Flywheel blanks are flame-cut from the plate, with allowance for exclusion of flame affected metal. A minimum of three Charpy V-notch tests

are made from each plate parallel and normal to the rolling direction, to determine that each blank satisfies design requirements. A nil ductility transition temperature less than +10°F is specified. The finished flywheels are subjected to 100% volumetric ultrasonic inspection. The finished machined bores are also subjected to magnetic particle or liquid penetrant examination.

These design fabrication techniques yield flywheels with primary stress at operating speed to less than 50% of the minimum specified material yield strength at room temperature (100°F to 150°F). Bursting speed of the flywheels has been calculated on the basis of Griffith-Irwin's results (*Reference 1*) to be 3900 rpm, more than three times the operating speed.

A fracture mechanics evaluation was made on the reactor coolant pump flywheel. This evaluation considered the following assumptions:

A. Maximum tangential stress at an assumed overspeed of 125% compared to a maximum expected overspeed of 109%.
B. A through crack through the thickness of the flywheel at the bore.
C. 400 cycles of startup operation in 40 years.

Using critical stress intensity factors and crack growth data attained on flywheel material, the critical crack size for failure was greater than 17 in. radially and the crack growth data was 0.030 in. to 0.60 in. per 1000 cycles.

The original inservice inspection program included a complete ultrasonic volumetric inspection and surface examination of all exposed surfaces at approximately 10-year intervals, and in-place ultrasonic volumetric examination of areas of higher stress concentration at the bore and keyway at approximately 3-year intervals. This was consistent with Safety Guide 14. The new inservice inspection program is described in Section 5.4.1.2.5.

1.8.1.15 Safety Guide 15 - Testing of Reinforcing Bars for Concrete Structures

The 1972 codes for testing of reinforcing bars for concrete structures were not available at the time that Ginna Station was built. The codes and practices followed do generally conform to these standards, however.

The concrete reinforcement used in the containment building and other Seismic Category I structures is deformed bar intermediate grade billet-steel conforming to the requirements of ASTM A15-64, Specifications for Billet-Steel Bars for Concrete Reinforcement, with deformations conforming to ASTM A305-56T, Deformed Bars for Concrete Reinforcement. Special large size concrete reinforcing bars are deformed bars of intermediate grade billet-steel conforming to ASTM A408-64, Specifications for Large Size Deformed Billet Steel Bars for Concrete Reinforcement. Reinforcing steel conforming to these specifications has a tensile strength of 70,000 psi to 90,000 psi and a minimum yield point of 40,000 psi.

All splicing and anchoring of the concrete reinforcement is in accordance with ACI 318-63. There was no splicing of bars by arc welding. The special large size bars were spliced by the Cadweld process.

It is to be noted that intermediate grade reinforcing steel is the highest ductility steel commonly used for construction. Certified mill reports of chemical and physical tests were submitted to the engineer, Gilbert Associates, Inc., for review and approval. Each bar was branded in the deforming process to carry identification as to the manufacturer, size, type, and yield strength, for example:

- B -Bethlehem.
- 18 -Size 18S.
- N -New billet steel.
- Blank -A-15 and A-408 steel.
- 6 -A-432 (60,000 psi yield).
- 7 -A-431 (75,000 psi yield).

Because of the identification system and because of the large quantity, the material was kept separated in the fabricator's yard. In addition, when loaded for mill shipment, all bars were properly separated and tagged with the manufacturer's identification number.

Visual inspection of the bars was made in the field for inclusions and representative randomly selected samples of reinforcing bar stocked onsite were tested for user's tensile tests.

The specifications stipulate that "arc welding concrete reinforcement for any purpose including the achievement of electrical continuity shall not be permitted unless noted otherwise on the drawings."

Concrete cover of reinforcing bar was at least the minimum specified by ACI-318.

1.8.1.16 Safety Guide 16 - Reporting of Operating Information

During the initial operating period that Ginna Station was producing power, reporting followed the intent of the regulations in effect at that time, specifically 10 CFR 20, 40, 50, 70, and 73. Therefore, RG&E conformed to the guidance of Safety Guide 16 as well as complying with all reporting requirements set forth in the Technical Specifications.

New reporting requirements have been instituted since this initial period and other requirements have been altered. RG&E has continued to comply with current NRC requirements. These include regulations such as 10 CFR 20, 21, 26, 50, 55, 70, 73, and 74, and selected NRC bulletins and generic letters such as GL 97-02. Other reporting requirements are contained in the Technical Specifications, Offsite Dose Calculation Manual (ODCM), and Technical Requirements Manual (TRM). Many of these various reporting requirements are addressed in plant procedures.

1.8.1.17 Safety Guide 17 - Protection Against Industrial Sabotage

The Rochester Gas and Electric Corporation submitted a proprietary document, Security at the Ginna Facility, to the AEC by cover letter dated October 8, 1971. This document describes in detail the implementation by RG&E of those sections of the Safety Guide applying to control of access and selection of personnel. The Security Plan was updated by RG&E

submittals of January 19, 1978, and April 12, 1983. The plan is maintained current in compliance with 10 CFR 50.54(p).

1.8.1.18 Safety Guide 18 - Structural Acceptance Test for Concrete Primary Reactor Containments

1.8.1.18.1 Structural Integrity Test

After completion of the construction of the entire containment vessel, a structural integrity test was performed, where a pneumatic pressure of 69 psig (115% of the design pressure of 60 psig) was maintained for approximately 4 hours. The pressurization of the vessel was done so as to permit readings and measurements which are more fully described hereafter. The readings and measurements were made during the initial pressurization (with pressure maintained a minimum of 3 hr at 0 psig, 14 psig, 35 psig, 60 psig, and at maximum test pressure of 69 psig, and thereafter during depressurization at 60 psig, 35 psig, and 0 psig. Except for the maximum pressure level (69 psig), the vessel pressure was slightly increased above the level at which the measurements were taken; and the pressure was then reduced to the specified value and observations made after at least 10 minutes to permit an adjustment of strains within the structure. Because the structure is so large, displacement measurements were made with sufficient precision to serve as confirmation of previously calculated response.

The test program further included, in addition to displacement measurements, a continuous visual examination of the vessel to observe concrete cracking. Observations of the entire vessel surface were made from existing or temporary platforms with special attention given to pertinent locations, including major discontinuities. A complete description of the instrumentation used to measure response is described below.

Predicted displacements developed for an internal pressure of 69 psig, which is the maximum pressure for the structural proof test, is included below. Although strain measurements were made, no predicted measurements are provided consistent with agreements previously documented in Appendices A, B, and C of Gilbert Associates, Inc., Report GAI 1720 (*Reference 2*). Strain values obtained, however, are analyzed to determine magnitude and direction of principal strains.

Maximum predicted crack widths for specifications are described below.

1.8.1.18.2 Instrumentation

The installation of all targets, linear variable differential transformers, whitewash for crack observations, load cells, tapes, strain gauges, photoelastic disks, cameras, junction boxes, wires, readout instruments, support structures, and platforms were completed prior to initiating pressurization of the vessel. The location for all instrumentation is shown in Table I of GAI 1720 (*Reference 2*). In addition, the covers on the enclosures over the tendon anchors and the wax surrounding the anchor head were removed to permit inspection of the anchorage, including button heads, during the test. People were stationed at the three locations for theodolite measurements, at the ledge for tendon anchorage inspection, and at each location where crack measurements were made. These people were equipped with communication means to maintain contact with a control located in the intermediate building at elevation 253

ft 6 in. where read-out instruments were located. In addition, three people were available to travel over accessible walkways to inspect the outer vessel surface.

The type of instruments used were as follows:

1. Jig transit with scales and targets.
2. Invar tapes.
3. Linear variable differential transformers.
4. Strain gauges.
5. Rosette strain gauges.
6. Photoelastic disks.
7. Load cells.

1.8.1.18.3 Displacement Measurements

Cylinder base rotation and displacement were measured utilizing linear variable differential transformers at three azimuths, one of which was directly below the equipment access opening. At each azimuth two linear variable differential transformers were located near the base of the structure with 6 ft. vertical separation. These radial displacements were used to determine the actual base rotation. Also, at each azimuth one linear variable differential transformer was used to determine the vertical displacement of the elastomer pad.

Radial displacement measurements were made at a total of 15 locations using a jig transit, base targets, and mounted scales.

A base target was attached to the structure at each of three different azimuths around the base of the cylinder. Five scales were attached (at each azimuth), three along the height of the cylinder and one each just above and below the ledge (i.e., elevation 343 ft. 2 in.). Relative radial displacements were determined at each scale location by aligning the transit with the base target and by plunging the scope up from the base target to each scale. Variations in the scale readings from the original reading indicated the amount of displacement.

The vertical displacement of the cylinder at the top (relative to the base ring at three azimuths for side wall elongation and average tendon strain) was determined using three invar tapes. The tapes were mounted at the ledge and extended down to the base ring, where weights tensioned the tapes. A scale at the base was read using an engraved mark on the tape to indicate relative elongations.

Linear variable differential transformers were utilized at 28 locations on concrete around the equipment access opening to measure horizontal and vertical displacements. Along the horizontal axis, on one side only, six horizontal and six vertical displacements were obtained to a point 21 ft. out from the edge of the hole. An identical set of displacements was obtained on the vertical axis above the hole. Additionally, on the horizontal and vertical axis, of those displacements previously mentioned, another point on each axis was selected to measure vertical and horizontal displacements at a point 2 ft from the opposite edge of the hole.

Displacement measurement accuracies are as follows: the jig transits, using an optical micrometer, had a resolution of 0.001 in. and an accuracy of 0.005 in. to 0.010 in. The linear variable differential transformers and associated instrumentation had a resolution of better than 0.001 in. and an accuracy of 0.002 in. to 0.005 in.

1.8.1.18.4 Strain Measurements

A total of 46 reinforcing bars were instrumented for strain measurements, 28 were at locations similar to linear variable differential transformer displacement measurement locations around the equipment access opening, and 18 were at locations above and below the ledge.

The liner was instrumented with rectangular rosettes at six locations, to indicate general strain in regions unaffected by geometric discontinuities, and at 32 locations around four typical penetrations. Eight rosettes were used at each penetration.

Strain gauges were attached to the tendon-anchorage bearing plates at tendons 13, 53, 93, and 133.

Load cells were installed under the button head of tendons 13, 53, 93, and 133. The strain gauges on reinforcing bars and associated instrumentation had a resolution of 0.4 micro-inch per inch strain and an accuracy of 2 to 3 micro-inches per inch. The strain gauges on the steel liner had a resolution of 1 micro-inch per inch and an accuracy of approximately 5 micro-inches.

The strain gauges on the bearing plates and the associated instrumentation had a resolution of 1 micro-inch per inch and an accuracy of approximately 5 micro-inches per inch. The instrumentation utilized for the tendon load cell had a measuring accuracy of 0.5% of full load capacity.

Photoelastic disks, 1.5 in. to 2 in. in diameter, were placed on the liner, around the same four penetrations where strain gauges were installed, to qualitatively augment the local values indicated by the strain gauges. Approximately 15 disks were located in one quadrant for each of four penetrations. (This resulted in approximately 25% surface coverage up to one diameter away from the opening.)

1.8.1.18.5 Test Results

Reading and recording of all measurements were made just prior to pressurizing, after depressurizing, and at each pressure increment, except that only one quadrant of photoelastic disks at each penetration were photographed while the structure was pressurized.

The identification and location of the instruments are shown on Figures 2 through 5 of GAI Report No. 1720 (*Reference 2*). These instruments were located in such a way that the actual response of the vessel during the test was determined and verified, with the criteria established prior to the performance of the test. The location of scales and gauges are as described in Table I of GAI Report No. 1720.

The results of the structural integrity test showed the stresses, strains, and displacements were within the specified limits and the GAI predicted results. The whitewash areas revealed crack

patterns and spacings in good agreement with the GAI prediction; there was no horizontal cracks in dome concrete except for construction joints. The base shear restraint was stiffer than anticipated. The strains and displacements of the cylinder wall, the discontinuity of dome and cylinder wall, and dome revealed that the structural stiffness of the containment vessel is greater than anticipated.

The structural capacity of the containment met and exceeded its imposed criteria. A detailed analysis and description of the Ginna containment structural integrity test is contained in GAI Report No. 1720.

1.8.1.19 Safety Guide 19 -Nondestructive Examination of Primary Containment Liners

1.8.1.19.1 Test Provisions

The weld seams in the liner plate are covered with a test channel to permit testing for leaks. Except for the equipment access hatch, all penetrations provide a double barrier against leakage and can be pressurized to permit testing of leak-tightness.

All penetrations through the containment reinforced concrete pressure barrier for pipe, electrical conductors, ducts, and access hatches are of the double barrier type.

In general, a penetration consists of a sleeve embedded in the reinforced concrete wall and welded to the containment liner. The weld to the liner is shrouded by a test channel which is used to demonstrate the integrity of the joint. The pipe, duct, or access hatch passes through the embedded sleeve and the ends of the resulting annulus are closed off, generally by welded end plates. Piping penetrations have a bellows type expansion joint mounted on the exterior end of the embedded sleeve where required to compensate for differential motions. The only exceptions to providing an annulus about piping occurs for the three drain lines from sump B.

Penetrations are designed with double seals so as to permit individual testing at the required test pressure.

All penetrations are provided with test canopies over the liner to penetration sleeve welds. Each canopy, except those noted below, is connected to and pressurized simultaneously with the annulus between the pipe and sleeve penetration when under test. The exceptions are the canopy for the fuel transfer penetration which must be pressurized independently of the annulus because of the separation posed by the transfer canal liner and the three pipe penetrations in sump B in which only the canopies are pressurized as there are no annuli.

1.8.1.19.2 Examination of Welds

All welded joints for the penetrations including the reinforcement about the openings (i.e., sleeve to reinforcing plate seam) were fully radiographed in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels, except that non-radiographable joint details were examined by the liquid penetrant method. For fully radiographed welds, acceptance standards for porosity are as shown in Appendix IV of the Nuclear Vessels Code. (The ASME Unfired Pressure Vessels Code states that porosity is not a factor in the acceptability of welds not required to be fully radiographed.)

Longitudinal and circumferential welded joints of the liner within the main shell, the welded joint connecting the dome to the cylinder, and all joints within the dome were inspected by the liquid penetrant method and spot radiography. All penetrations including the equipment access door and the personnel locks were examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. All other shop fabricated components, including the reinforcement about openings, were fully radiographed. All other joint details were examined by the liquid penetrant method. Full radiography is performed in accordance with the procedures and governed by the acceptability standards of Paragraph N-624 of the ASME Nuclear Vessels Code. Spot radiography is performed in accordance with the procedures and governed by the standards of Paragraph UW-52 of the ASME Unfired Pressure Vessels Code. Methods for liquid penetrant examination were in accordance with Appendix VIII of the ASME Unfired Pressure Vessels Code.

1.8.1.19.3 Pressure Tests

All piping penetrations and personnel locks were pressure tested in the fabricator’s shop to demonstrate leak tightness and structural integrity.

In order to ensure that the joints in the liner plate and penetrations as well as all weld connections of test channels were leak tight, it was required that all welds be examined by detecting leaks at 69 psig test pressure using a soap bubble test or a mixture of air and Freon, and 100% of detectable leaks be arrested. These tests were preliminary to the performance of the initial integrated leak rate test which ensured that the containment leak rate was no greater than 0.1% of the contained volume in 24 hours at 60 psig.

The liner weld seams were also examined by pressurizing the test channels to design pressure (60 psig) with a mixture of air and Freon, and checking all seams with a halogen leak detector. All detectable leaks were corrected by repairing the weld and retesting.

1.8.1.19.4 Quality Control Provisions

The following quality control provisions were employed in the welding procedure for the liner:

The qualification of welding procedures and welders was in accordance with Section IX, Welding Qualifications, of the ASME Boiler and Pressure Vessel Code. Contractor shall submit welding procedures to the Engineer for review.

The qualification tests described in Section IX, Part A, include guided bend tests to demonstrate weld ductility. All penetrations shall be examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. Other shop fabricated components including the reinforcement about openings shall be fully radiographed. All non-radiographable joint details shall be examined by the liquid penetrant method.

Conformance to this code was adhered to in all applicable cases.

1.8.1.20 Safety Guide 20 - Vibration Measurements on Reactor Internals

A vibration analysis and test program was developed for Ginna Station by Westinghouse Corporation. The preoperational test program and its results are discussed in Section 14.6. The results show that the vibration of the reactor internals for the Ginna plant are well within the existing criteria.

A program was conducted during the first MODE 6 (Refueling) shutdown of the Ginna reactor (March 1971) to inspect and evaluate the performance of the reactor internals and core components. This inspection program was based on an inspection of all components, with emphasis on the thermal shield area since the thermal shield has previously been the most vulnerable problem area.

The structures inside and outside of the lower internals, the upper internals, three control rod drive shafts, and all rod cluster control assembly control rods were inspected using a closed-circuit underwater television and/or boroscope. All of the inspections performed by television were recorded on video tape; photographs were taken through the boroscope to record that portion of the inspection. This inspection revealed no problem areas in any of the items inspected.

The inspection program is described in Westinghouse report WCAP 7780, October 1971, Robert E. Ginna Nuclear Generating Station, March 1971 Refueling Shutdown Reactor Internals and Core Components Evaluation.

1.8.1.21 Safety Guide 21 - Measuring and Reporting Effluents From Nuclear Power Plants

Starting on January 1, 1972, plant effluent monitoring and reporting was prepared in the format given in Appendix A of Safety Guide 21 and submitted to the State of New York on a monthly basis. A report in the format of Appendix A was provided to the AEC for the year 1971. The Technical Specifications, as revised on March 1, 1972, followed the intent of Safety Guide 21 for measuring and recording the plant effluents. Technical Specifications provide the requirements for a Radiological Effluent Controls Program. Plant records will be maintained to demonstrate that the sensitivity of analysis is within the limits given in the safety guide.

An onsite meteorological tower was fully operational early in 1965 and was used extensively in the collection of preoperational meteorological data. During early 1972, the recording instrumentation was relocated inside the turbine building, and subsequently the data collection was moved to the Plant Process Computer System (PPCS). Data are currently being used in upgrading calculations of dilution factors for radiological releases.

Preoperational onsite meteorological data were evaluated to provide a basis for controlled radiological gas release limits, accident analysis, and storm prediction criteria in the FSAR.

Basic and critical meteorological parameters are recorded at the Ginna site. See Section 2.3.3 for additional details. This information provides RG&E with the capability of assessing the potential dispersion characteristics of radioactive releases to the environment through the atmosphere. Such assessments provide RG&E with the ability to demonstrate that operations

are well within the limits of 10 CFR 20. Current practice is to maintain effluent releases within 10 CFR 50, Appendix I limits, as specified in the Offsite Dose Calculation Manual (ODCM).

1.8.1.22 Safety Guide 22 - Periodic Testing of Protection System Actuation Functions

The plant protection system has been designed to permit periodic testing to extend to and include actuation devices and actuated equipment whenever practicable. While it is not possible to operate all actuation devices (such as trip of control rods) or significantly vary most of the operating parameters (such as coolant pressure) during operation, it is possible to test most equipment when the plant is in full power operation.

The bistable portions of the protective system (i.e., relays, bistables, etc.) provide trip signals only after signals from analog portions of the system reach preset values. Capability is provided for calibrating and testing the performance of the bistable portion of protective channels and various combinations of the logic networks during reactor operation.

The analog portion of a protective channel (i.e., sensors and amplifiers) provides analog signals of reactor or plant parameters. The following means are provided to permit checking the analog portion of a protective channel during reactor operation:

A. Varying the monitored variable.
B. Introducing and varying a substitute transmitter signal.
C. Cross checking between identical channels or between channels which bear a known relationship to each other and which have readouts available.

During operation it is also possible to test the pumps used in a safety injection. For instance, each high-head safety injection pump can be and is tested in accordance with the inservice pump and valve testing program.

Testing that cannot be done during operation is completed during MODE 6 (Refueling) shutdowns. The safety injection system is tested to see that as a system it can perform according to design. When completed, the test shows that separate and redundant actuation signals are operative and that the valves and pumps that are required for safety injection are indeed operable.

Where the ability of a system to respond to a bona fide accident signal is intentionally bypassed for the purpose of performing a test during reactor operation, the expansion of the bypass condition to redundant systems is prevented. In addition, the condition is automatically indicated to the reactor operator in the main control room.

1.8.1.23 Safety Guide 23 - Onsite Meteorological Programs

The Ginna plant site meteorology is described in Section 2.3. The 2 year preoperational meteorological program data is summarized in Section 2.7 of the original FSAR.

These data were utilized by the NRC and RG&E for accident analysis and gaseous release limit determination during the initial license application for a 1300 MWt rating and, more recently, during the review of the application by RG&E to increase its licensed power level

from 1300 MWt to 1520 MWt. More information on the meteorological tower is provided in the discussion of Safety Guide 21.

1.8.1.24 Safety Guide 24 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Pressurized Water Reactor Radioactive Gas Storage Tank Failure

The activity in a gas decay tank is taken to be the maximum amount that could accumulate from operation with cladding defects in 1% of the fuel rods. The maximum activity is obtained by assuming the noble gases xenon and krypton are accumulated with no release over a full core cycle. This postulated amount of activity, one reactor coolant system equilibrium cycle inventory, is 4.6×10^4 Ci equivalent Xenon-133. This value is particularly conservative because some of this activity would normally remain in the coolant, some would have been dispersed earlier through the stack, and the shorter lived isotopes would have decayed substantially. Current assumptions for postulated activity are provided in Section 15.7.1.1.4.

Samples taken from gas storage tanks in pressurized water reactor plants in operation show no appreciable amount of iodine.

To define the maximum doses, the release is assumed to result from gross failure of a gas decay tank giving an instantaneous release of its volatile and gaseous contents to the atmosphere.

The maximum whole-body beta-gamma dose, based on meteorology previously described in Safety Guide 4, is less than a few rem (less than three). This is well below the 25 rem guide line value in 10 CFR 100.

1.8.1.25 Safety Guide 25 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Fuel Handling Accident in the Fuel Handling and Storage Facility for Boiling and Pressurized Water Reactors

The Ginna spent fuel pool (SFP) charcoal filter system was designed and constructed prior to the issuance of Safety Guide 25. An analysis based on Regulatory Guide 1.25 was performed and was described in Section 15.7.3 of the original FSAR. Radiological consequences were calculated to be less than 34 rem to the thyroid at the exclusion area boundary, which was well below the 10 CFR 100 exposure guidelines. An analysis based on Regulatory Guide 1.25 was performed and is described in Section 15.7.3. Current calculated radiological consequences are provided in Section 15.7.3.

1.8.1.26 Safety Guide 26 - Quality Group Classification and Standards

Although Safety Guide 26 was not in effect when Ginna Station was constructed, RG&E subsequently classified the systems in Ginna Station in accordance with this guide.

1.8.1.27 Safety Guide 27 - Ultimate Heat Sink

The circulating water intake system of Ginna Station is designed to provide a reliable supply of Lake Ontario water, regardless of weather or lake conditions, to a suction of the condenser circulating water pumps, house service water pumps, and the fire water pumps. With two

pumps operating, the nominal flow of the circulating water system is approximately 333,000 gpm. Operation of a single circulating water pump reduces the nominal flow rate by about 50%.

In meeting the high reliability requirements of this safety guide, the intake system is completely submerged below the surface of the lake. A 10 ft. diameter reinforced concrete lined tunnel, driven through bedrock, extends 3100 ft. northerly from the shore line. The tunnel rises vertically and connects to a reinforced-concrete inlet section. The minimum mean monthly lake level of record (243.0 ft. msl) will result in a depth of water of 26 ft. above the lowest entrance into the intake structure.

The probability of water stoppage due to plugging of the inlet has been reduced to an extremely low value by incorporating certain design features in the system. Heavy screen racks with bars spaced at 14 in. on center will prevent large objects from entering the system on six out of eight sides of the octagonal intake. Two of the eight ports are non-heated and have open space of approximately 68-in. x 112-in. to prevent accumulation of frazil ice.

Redundant traveling water screens, located in the screen house will remove trash from the cooling water. At conditions of full flow (approximately 355,000 gpm) the velocity at the intake screen racks is 0.8 ft/sec. The plant cooling water requirements during an accident would be approximately 10,000 gpm, which would result in a velocity of 0.02 ft/sec.

In addition, water enters on a full 360-degree circle thereby protecting against the possibility of stoppage by a single large piece of material. The low velocity, plus the submergence, provides assurance that floating ice will not plug the intake. The only phenomenon that is credible to contribute to the plugging would be the accumulation of frazil ice on the screen racks. The bars have electric heaters that will keep the metal surface above 32□F, which minimizes the adhesive characteristics of frazil ice to metal objects (see Section 10.6.2.1); however, bridging of accumulated frazil ice from unheated portions of the metal heater racks to the rest of the surface area of the rack has still proven to be a credible scenario. Two sides of the octagonal structure prevent plugging of the intake structure by providing a large, open flow path for plant cooling water.

Warm water recirculation is provided in the screen house to melt any ice that might reach this point. Additional information is provided in Section 2.4 and Appendix 2A. Refer to Section 10.6.2.1 for an update to this historical information.

1.8.1.28 Safety Guide 28 - Quality Assurance Program Requirements

The standards, specifications, and guidelines existing at the time Ginna Station was constructed, pertinent to quality assurance, were at least met or exceeded. Details of the quality assurance program implemented are described in Chapter 1 of the original FSAR.

A quality assurance program was instituted for the operation, maintenance, and system redesign of the Ginna plant that conformed to the guidelines of N45.2-1971.

1.8.1.29 Safety Guide 29 - Seismic Design Classification

Although this Safety Guide had not been published at the time of the Ginna Station design and construction, the seismic classifications generally conform to this Guide. The seismic classification of equipment is provided in Section 3.2 and in the UFSAR system descriptions and is noted on the Ginna piping and instrumentation diagrams (P&IDs).

1.8.2 CONFORMANCE TO DIVISION I REGULATORY GUIDES

The information in this section represents the position of the R. E. Ginna Nuclear Power Plant with respect to certain of the NRC Division 1 Regulatory Guides in December 1973. The information was submitted to the NRC as Supplement 1 to the Technical Supplement Accompanying the Application for a Full-Term Operating License. Regulatory Guides 1.3, 1.5, and 1.5.6 are not applicable to the R. E. Ginna Nuclear Power Plant and are not discussed. Regulatory Guides 1.4, 1.10, 1.15, 1.17, 1.18, 1.19, and 1.29 are addressed because either the guides or the R. E. Ginna positions were revised since the submission of the positions relative to like-numbered Safety Guides presented in Section 1.8.1. Regulatory Guides 1.30 through 1.143 were not addressed as Safety Guides in Section 1.8.1 and are included in this section.

1.8.2.1 Regulatory Guide 1.4 - Assumptions Used for Evaluating the Potential Radiological Consequences of a Loss-of-Coolant Accident for Pressurized Water Reactors

This subject is discussed in Section 1.8.1.4.

1.8.2.2 Regulatory Guide 1.10 - Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures

This subject is discussed in detail in Section 1.8.1.10.

1.8.2.3 Regulatory Guide 1.15 - Testing of Reinforcing Bars for Category I Concrete Structures

This subject is discussed in detail in Section 1.8.1.15.

1.8.2.4 Regulatory Guide 1.16 - Reporting of Operating Information

This subject is discussed in detail in Section 1.8.1.6.

1.8.2.5 Regulatory Guide 1.17 - Protection of Nuclear Plants Against Industrial Sabotage

This subject is discussed in detail in Section 1.8.1.17.

1.8.2.6 Regulatory Guide 1.18 - Structural Acceptance Test for Concrete Primary Reactor Containments

This subject is discussed in detail in Section 1.8.1.18.

1.8.2.7 Regulatory Guide 1.19 - Nondestructive Examination of Primary Containment Liner Welds

A description of the inspection methods employed during construction is presented in Section 1.8.1.19.

1.8.2.8 Regulatory Guide 1.26, Revision 3 - Quality Group Classifications & Standards for Water, Steam, and Radioactive - Waste Containing Components of Nuclear Power Plants

A classification process is established within station procedures to identify components, systems, and structures that are safety related (SR), safety significant (SS), or Non-Nuclear Safety (NS). Criteria are based on information contained in the Updated Final Safety Analysis Report (UFSAR), licensing commitments, guidelines contained in NRC Regulatory Guides, and functional guidance derived from ANSI/ANS 51.1 -1983.

1.8.2.9 Regulatory Guide 1.29, Revision 3 - Seismic Design Classification

The Ginna plant components, systems, and structures were classified for seismic design as tabulated in Section 3.2. Current seismic classifications are provided in Section 3.2, applicable sections of the UFSAR, and on the Ginna P&IDs. Comparison of the Ginna plant seismic classification system with that recommended by Regulatory Guide 1.29 shows close agreement between the two classification systems.

Plant Operation

Seismic design requirements for existing structures, systems, and components performing functions listed in positions C.1 and C.3 of the Regulatory Guide are specified in the UFSAR. New structures, systems, and components, and configuration changes meet the seismic design requirements of this regulatory guide or the UFSAR. The pertinent quality assurance requirements of 10CFR50, Appendix B are applied as required by positions C.1 and C.4 of this Regulatory Guide, irrespective of an item's seismic design. Portions of existing structures, systems, and components with failure consequences described in position C.2 of this guide are designed and constructed to seismic requirements specified in the UFSAR. New structures, systems, and components, and configuration changes meet the design and construction seismic requirements of the UFSAR or this Regulatory Guide. A quality assurance program similar to 10CFR50, Appendix B is applied to the SSE failure prevention function of these items. These items are not considered basic components pursuant to 10CFR21.

1.8.2.10 Regulatory Guide 1.30 - Quality Assurance Requirements for the Installation, Inspection, and Testing of Instrumentation and Electrical Equipment

Regulatory Guide 1.30 and the related IEEE Standard 336-1971 were published after the construction of the R. E. Ginna Nuclear Power Plant. The IEEE Standard 336-1971 is, however, discussed in Section 1.8.3 as it applied to Ginna Station in August 1972.

Plant Operation

Operational commitments to this Regulatory Guide are discussed in detail in the Quality Assurance Topical Report (QATR). The QATR is cited in Section 17.2 of the UFSAR and is submitted to the NRC in accordance with the provisions of 10 CFR 50.54(a).

Requirements for checks, calibrations, and tests of instrument channels are given in the Technical Specifications.

1.8.2.11 Regulatory Guide 1.31 - Control of Stainless Steel Welding

Regulatory Guide 1.31 was published after the fabrication cycle for the Ginna plant. However, the stainless steel welding for the Ginna plant meets the intent of Regulatory Guide 1.31.

All welding was conducted using those procedures that have been approved by the ASME Code Rules of Section III and IX. The welding procedures were qualified by nondestructive and destructive testing according to the ASME Code Rules of Section III and IX.

When these welding procedure tests were performed on test welds made from base metal and weld metal materials which were from the same lots of materials used in the fabrication of components, additional testing was frequently required to determine the metallurgical, chemical, physical, corrosion, etc., characteristics of the weldment. The additional tests that were conducted on a technical case basis are as follows: light and electron microscopy, elevated temperature mechanical properties, chemical check analysis, fatigue tests, intergranular corrosion tests, or static and dynamic corrosion tests within reactor water chemistry limitations.

The following welding methods were tested individually and in multiprocess combinations, using the following energy input ranges for the respective method as calculated by the formula:

$$H = \frac{(E)(I)(60)}{S}$$

(Equation 1.8-1)

where:

H = J/in.

E = volts

I = amperes

S = travel speed, in./min

Welding Process Method	Energy Input Range (kJ/in.)
Manual shielded tungsten arc	20 to 50
Manual shielded metallic arc	15 to 120
Semiautomatic gas shielded metal arc	40 to 60
Automatic gas shielded tungsten arc-hot wire	10 to 50
Automatic submerged arc	60 to 140
Automatic electron beam-soft vacuum	10 to 50

The interpass temperature of all welding methods was limited to 350°F maximum. All full penetration welds were inspected in accordance with Article NB5000 of the 1965 ASME Section III Code rules. Welding materials were required to conform and were controlled in accordance with Subarticle NB2400 of the 1965 ASME Section III Code rules.

In addition, the austenitic stainless steel welding material used for joining austenitic stainless steel base materials in the reactor coolant pressure boundary, systems required for reactor shutdown and emergency core cooling, and the core structural load-bearing members conforms to ASME Material Specifications SA-298 and SA-371. These materials were tested and qualified according to the requirements stipulated in the 1965 ASME Boiler and Pressure Vessel Code Sections II, III, and IX, respectively. All of these welding materials conform to ASME weld metal analysis A-7.

Plant Operation

Regulatory Guide 1.31 is the basis for stainless steel welding procedures. Each procedure is designed to produce high quality welds using the variables and methods outlined in the procedure. Qualification of these procedures is done in accordance with Section III and Section IX of the ASME Boiler and Pressure Vessel Code.

In production welding, strict control is maintained to ensure that every step that may affect the quality of the final weld is supervised and checked for compliance with the proper criteria and that the welding procedure is being followed. The consumables used for stainless steel welding jobs meet the requirements of Section II of the ASME Code and are purchased with actual chemical composition and mechanical properties certified. All stainless steel welds are nondestructively examined to verify their quality and code compliance.

1.8.2.12 Regulatory Guide 1.32 - Use of IEEE Standard 308-1971, Criteria for Class IEE Electric Systems for Nuclear Power Generating Stations

Conformance to IEEE Standard 308-1971 is discussed in Section 1.8.3. Regulatory Guide 1.32 (formerly Safety Guide 32, August 1972) identifies two areas of possible conflict between IEEE Standard 308 and Criterion 17: availability of offsite power and battery charger supply.

The availability of offsite power is discussed fully in Chapter 8. The electrical power system is designed with a single station auxiliary (startup) transformer, which gives immediate access to two independent sources of offsite power. In the event that this access is not available, either of the two backup diesel generators is capable of supplying safeguards loads. As an independent additional source of offsite power, the unit auxiliary transformer can be supplied from the normally outgoing power feeder by disconnecting the flexible generator bus disconnects. This can be accomplished in a short time, (less than 8 hr) after which all the vital loads could be supplied from the unit auxiliary transformer. Because of the multiple immediate access power sources, the one delayed access power source conforms to Regulatory Guide 1.32 and General Design Criteria 17.

The battery chargers are discussed in Section 1.8.3. Operating experience has proven that the battery charger capacity is more than sufficient to supply all long-term plant loads while restoring the batteries from the minimum charge to the fully charged state.

1.8.2.13 Regulatory Guide 1.33 -Quality Assurance Program Requirements (Operation)

ANSI N18.7-1972, Administrative Controls for Nuclear Power Plants, and ANSI N45.2-1971, Quality Assurance Program Requirements for Nuclear Power Plants, were used as a basis for developing the initial Ginna Station Operational Quality Assurance Program that is cited in Section 17.2. Appendix A to Regulatory Guide 1.33 was used as guidance in developing procedures for operating and maintenance activities.

Plant Operation

Operational commitments to this Regulatory Guide are discussed in detail in the Quality Assurance Topical Report (QATR). The QATR is cited in Section 17.2 of the UFSAR and is submitted to the NRC in accordance with the provisions of 10 CFR 50.54(a).

1.8.2.14 Regulatory Guide 1.34 -Control of Electroslag Weld Properties

Regulatory Guide 1.34 was published after the construction of the Ginna Nuclear Power Plant; however, the electroslag welding performed for the Ginna plant meets all of the guidelines of Regulatory Guide 1.34. The specific applications of electroslag welding for the Ginna plant were for the shop assembly welds of the primary coolant system, 90-degree piping elbows, and the reactor coolant pump casings, as discussed in detail in Section 5.2.3.1.2.

Assembly of the elbows was accomplished using a procedure specifying the following parameters:

A. Slag - electrically conductive type ARCOS BV-1 Vertomax or equivalent; pool depth 1 to 2 in.

B. Current - 60 cycle ac; 500 to 620 amp.

C. Voltage - 44 to 50 V.

D. Feed rate - 35 lb/hr; 1/8-in. single wire; 8 to 10 oscillations/min, nominal 2-in. oscillation.

Assembly of the pump casings was accomplished using a similar procedure, with identical welding parameters, but using two and three wires.

No electroslag welding is now being done at the Ginna plant and it is not anticipated that any will be done in the future.

1.8.2.15 Regulatory Guide 1.35 - Inservice Surveillance of Ungrouted Tendons in Prestressed Concrete Containment Structures

The tendon surveillance program for Ginna Station as required by the Technical Specifications is in accordance with Regulatory Guide 1.35, Revision 2. A detailed discussion of this inservice surveillance program is provided in Section 3.8.1.7.

1.8.2.16 Regulatory Guide 1.36, Revision 0 - Nonmetallic Thermal Insulation for Austenitic Stainless Steel

Although Regulatory Guide 1.36 had not been published before the completion of construction of the R. E. Ginna Nuclear Power Plant, the quality of the thermal insulation applied to austenitic stainless steel components was carefully specified and checked.

The practice employed during construction of the Ginna plant meets the requirements of Regulatory Guide 1.36 and is more stringent in several respects. The tests for qualification specified by the guide (ASTM C692-71 or RDT M12-1T) allow use of the tested insulation material if no more than one of the metallic test samples crack. Westinghouse procedure rejected the tested insulation material if any of the test samples cracked. The procedure followed for the R. E. Ginna Nuclear Power Plant was more specific than the procedures suggested by the guide, in that the Westinghouse specification required determination of leachable chloride and fluoride ions from a sample of the insulating material.

Experience has shown that of the three analysis methods allowed under ASTM D512 and ASTM D1179 for leachable chloride and fluoride, the referee method, which was used in the analysis of the Ginna insulation, is the most accurate and most suitable for nuclear applications.

Plant Operation

Insulating materials are not considered basic components pursuant to 10CFR21 and thus the supplier is not required to have a quality assurance program to cover the testing, lot control, and contamination control provisions of this Regulatory Guide. A quality assurance program similar to 10CFR50, Appendix B is applied to insulating materials on or near Ginna Station safety related stainless steel piping and components.

1.8.2.17 Regulatory Guide 1.37, Revision 0 - Quality Assurance for Cleaning of Fluid Systems and Associated Components of Water-Cooled Nuclear Power Plants

The Ginna plant obtained its construction permit in April 1966. Regulatory Guide 1.37 and related ANSI Standard N45.2.1-1973 were published in 1973; therefore, these standards were not available during the construction phase of the Ginna plant. However, a formal program for the cleaning of the fluid components of the power plant was followed and documented.

The flushing water for the nuclear steam supply system met the following maximum water chemistry specifications: chlorides, maximum ppm -0.15; undissolved solids, maximum ppm -5.0; conductivity, maximum mhos/cm -5; pH -6.0 to 8.0; and visual clarity -no turbidity, oil, or sediment.

Pipe and units large enough to permit entry by personnel were cleaned by locally applying approved solvents (Stoddard solvent, acetone, and alcohol) and demineralized water. A line or equipment was considered clean when flush cloths showed no grindings, filings, or insoluble particulate matter larger than 40 microns (naked eye visibility lower limit) or oil stains visible to the naked eye. The final cleaned equipment was free of visible dust, grit, rust, weld splatter, scale, oil, grease, pickling solution residue, cleaning fluid film, or other foreign matter. Only iron-free aluminum, oxide grinders were used to remove trapped foreign particles.

The cleaning of the component cooling system was accomplished first by flushing separate lines to waste and, second, by flushing the complete system. Stainless steel strainers were installed and utilized during the second phase. The system was considered clean when no significant buildup was noted on the strainers. The demineralized water used met the same water chemistry specifications as the nuclear steam supply system flushing water and was treated with 100 ppm hydrazine for oxygen control.

For the secondary plant, the condensate and feedwater system was cleaned by manual cleaning of condenser surfaces and hotwells, cold water flush, and alkaline cleaning. The main steam system cleaning procedures included manual cleaning, cold water flush, alkaline cleaning, and acid cleaning.

These examples indicate the concern for system cleanliness during construction of the R. E. Ginna Nuclear Power Plant, even before the existence of the current guidelines.

Plant Operation

For new construction activities, the cleanliness requirements of ANSI N45.2.1-1973 as modified by the Regulatory Guide are followed. Consistent with Position C.2 of the Regulatory Guide, the cleanliness requirements of this standard are used when applicable to maintenance on operating systems. The cleanliness requirements applied to operational systems are established in station procedures.

1.8.2.18 Regulatory Guide 1.38, Revision 2 - Quality Assurance Requirements for Packaging, Shipping, Receiving, Storage, and Handling of Items for Water-Cooled Nuclear Power Plants

Regulatory Guide 1.38 and related ANSI Standard N45.2.2-1972, were published after the construction of the R. E. Ginna Nuclear Power Plant.

However, each piece of equipment has detailed equipment specifications. The detailed requirements for preparation of equipment for shipment were included in the equipment specifications. These included sealing of all openings, protection of nozzle preparations, the use of dessicants if required, etc. Where required, the suppliers submitted detailed plans for review and approval.

For example, the reactor vessel supplier provided a cover and seal system to protect all internal surfaces and external stainless steel and machined surfaces from exposure to ambient environments during shipment, storage at the site, and installation. The protective means included pressurized inert gas with covers.

For the reactor internals, the lower assembly was shipped on an up-ending skid, shock-mounted to limit loads transmitted to the assembly during shipment. Prior to installation onto the skid, the lower internals were wrapped in a plastic film and sealed. Internal bracing was used inside the assembly. The upper internal assembly was shipped in a shock-mounted, dual-purpose shipping assembly stand in the vertical position. This package was also wrapped and sealed in a plastic film. Both the skid and the stand had a protective metal covering to provide weather protection and long-term storage protection at the site. All other

components had similar protection, as required, against mechanical or environmental damage during shipment and/or site storage.

These detailed examples indicate the concern for components during transportation and handling.

Plant Operation

Ginna currently maintains conformance with this Regulatory Guide.

1.8.2.19 Regulatory Guide 1.39 - Housekeeping Requirements for Water-Cooled Nuclear Power Plants

The housekeeping awareness was generally followed for quality assurance jobs at Ginna Station. This was generally handled through precautions listed in maintenance, repair, and modifications procedures and also through quality control inspection and surveillance. Additional quality assurance information is provided in Chapter 17.

Plant Operation

Operational commitments to this Regulatory Guide are discussed in detail in the Quality Assurance Topical Report (QATR). The QATR is cited in Section 17.2 of the UFSAR and is submitted to the NRC in accordance with the provisions of 10 CFR 50.54(a).

1.8.2.20 Regulatory Guide 1.40 - Qualification Tests of Continuous-Duty Motors Installed Inside the Containment of Water-Cooled Nuclear Power Plants

Conformance to IEEE Standard 334-1971 is fully discussed in Section 1.8.3.

The containment recirculation fan cooler (CRFC) and filtration system fan motors are the only continuous-duty Class 1E motors within the containment. Environmental qualification is discussed in Section 3.11.

1.8.2.21 Regulatory Guide 1.41 - Preoperational Testing of Redundant Onsite Electric Power Systems to Verify Proper Load Group Assignments

This Regulatory Guide describes an acceptable method for verifying power load group assignments for onsite emergency power systems described in Regulatory Guides 1.6 and 1.32. Regulatory Guide 1.6 is discussed in Section 1.8.1.6. Regulatory Guide 1.32 is discussed in Section 1.8.2.10. The underlying standard, IEEE Standard 308-1971, is discussed in Section 1.8.3. Initial startup tests are discussed in Chapter 14.

The capability of adequately supplying the demand of the safeguards bus load groupings was preoperationally demonstrated. Buses 14 and 18 comprise one redundant safeguards train and buses 16 and 17 comprise the other. The two trains were isolated from each other and from offsite power sources. One diesel was started and the timing sequence for starting of all associated equipment was checked against design. The test was repeated for the other diesel. It was particularly important to test the diesels separately since one of the high-head safety injection pumps is designed to operate from either diesel generator, switching to an operating generator if one is not operating. Tests were continued for a sufficient time to guarantee

proper starting sequence. The plant auxiliary startup transformer was also used as a power source. All equipment was monitored during the tests.

1.8.2.22 Regulatory Guide 1.42 - Interim Licensing Policy on As Low As Practicable for Gaseous Radioiodine Releases from Light-Water-Cooled Nuclear Power Reactors

Ginna Station is meeting as-low-as-practicable releases for gaseous iodine by the use of charcoal filters on all exhaust air from restricted areas. As a check on the efficiency of the charcoal filter system, all plant vent exhaust air is continually monitored for iodine. A further check is made by monthly analysis of samples of milk taken from nearby dairy herds. These three systems of control are referred to in the Offsite Dose Calculation Manual (ODCM).

In the initial design and construction of Ginna Station, all air purged from the containment vessel passed through high efficiency particulate air and charcoal filters. There was the further option of using a recirculating high efficiency particulate air and charcoal filter system within the containment. Air from high activity areas of the auxiliary building passed through charcoal and all air from restricted areas passed through high efficiency particulate air filters.

Prior to the first spent fuel handling in 1971, a bank of charcoal beds was installed to filter the air from the spent fuel pool (SFP) area. A charcoal filter was also added to the laboratory exhaust air system in 1971. In June 1972, another charcoal and high efficiency particulate air unit was added to filter iodine from the remaining auxiliary building air.

These filter systems are periodically tested for efficiency of operation. A leak test using Freon is done in the plant according to the Ventilation Filter Testing Program schedule and the efficiency of the activated charcoal adsorber is determined by an independent laboratory.

Both the plant vent and the containment vent have an iodine sampler with continuous monitoring. The monitor is read out and recorded in the control room and is programmed to alarm at a fraction of the release limit value calculated by methods described in the Offsite Dose Calculation Manual (ODCM). Action can then be taken, using the appropriate procedure, to meet the 24-hour limit allowed by the ODCM.

Thus the Ginna plant can be shown to meet the guidelines of Regulatory Guide 1.42 on an analytical basis and, in fact, several years of operations confirm this conclusion. Subsequently, 10 CFR 50, Appendix I, was published. Ginna LLC conforms to 10 CFR 50, Appendix I, as described in the Technical Specifications.

1.8.2.23 Regulatory Guide 1.43 - Control of Stainless Steel Weld Cladding of Low-Alloy Steel Components

The R. E. Ginna Nuclear Power Plant reactor vessel and pressurizer carbon steel surfaces in contact with primary coolant were clad with stainless steel type 304 equivalent weld deposit. For the replacement steam generators all ferritic steel surfaces in contact with the primary coolant are clad with weld deposited austenitic stainless steel (Types 308L and 309L) or Alloy 600. These ferritic base steels are either SA-508 Cl 3 or SA-533 Type B Cl 1 procured to fine grain practice and are not considered susceptible to underclad cracking. The Ginna Nuclear Power Plant reactor vessel shell and nozzle forgings were fabricated from SA-508

Class 2 material. However, these surfaces were stainless steel weld clad only by single-wire low energy input weld processes, which are not restricted by Regulatory Guide 1.43. The Ginna pressurizer SA-302 grade B plate and SA-216 WCC casting surfaces in contact with primary coolant were clad with weld deposited stainless steel. These base materials are not restricted by the requirements of Regulatory Guide 1.43.

Underclad cracking is not expected for the Ginna plant stainless steel weld clad components. Of those components clad only the reactor vessel shell and nozzle forgings are SA-508 Class 2 base material. All of the welding processes used to clad components in contact with primary coolant are single-wire low energy input processes.

No stainless steel weld cladding of low-alloy steel components is now being done at the Ginna plant, and it is not anticipated that any will be done in the future.

1.8.2.24 Regulatory Guide 1.44 - Control of the Use of Sensitized Stainless Steel

Regulatory Guide 1.44 was published after the construction of the R. E. Ginna Nuclear Power Plant. However, the R. E. Ginna Nuclear Power Plant meets the intent of Regulatory Guide 1.44.

All austenitic stainless steel materials used in the fabrication, installation, and testing of nuclear steam supply components and systems were handled, protected, stored, and cleaned according to recognized and accepted contemporary methods and techniques. To ensure that these methods and techniques were followed, surveillance of operations was conducted by Quality Assurance personnel of the applicant and the nuclear steam supply system supplier. Stainless steel material from which components were fabricated were procured in the solution heat-treated condition as required by the ASME Section II materials specifications.

Methods and materials used in manufacturing stainless steel components of the Ginna reactor coolant pressure boundary are described in detail in a letter dated October 6, 1970, from Edward J. Nelson, RG&E, to Peter A. Morris, AEC (Docket No. 50-244).

For internals where austenitic stainless steel was given a stress relieving treatment above 800°F, a high-temperature solution heat treatment procedure was used. This was performed in the temperature range of 1600°F to 1900°F with sufficient holding times.

For core support structural load bearing members and stainless steel reactor coolant pressure boundary welds, all welding on stainless steel was conducted by procedures that limit the interpass temperature to 350°F maximum. All of the reactor vessel and pressurizer nozzles, as well as the reactor vessel control rod drive mechanism adapters[1] and reactor vessel head

[1] The control rod drive mechanism (CRDM) adapters on the replacement reactor vessel closure head (RVCH) were fabricated from SA-182 Type F304LN stainless steel forgings also supplied in the solution heat-treated condition (annealed at 1950°F ± 25°F and water quenched). In addition, one sample from each heat of material used for the adapters, was given a simulated postweld heat-treatment (i.e., exposed to a temperature on the sensitizing range (1250°F ± 25°F) for 20 hours) and tested in accordance with ASTM A262, Practice E to verify the absence of sensitization. Therefore, postweld stress relief heat-treatment was not required on the adapters after welding, and consequently no potential for sensitization exists. The metallurgical condition of the adapters in the replacement RVCH is therefore superior to that of the adapters in the original head.

gasket monitor tubes, were postweld stress relief heat-treated for the minimum practical time (3 hours to 11 hours depending on size) at 1125°F ± 25°F. However, the reactor vessel primary coolant nozzles' weld deposits are calculated to contain at least 5% ferrite according to the Schaeffler Diagram. Thus, a duplex (austenite plus ferrite) structure can be expected in the safe ends of these nozzles. The guide recognizes that weld metal with duplex structures have demonstrated adequate resistance to intergranular attack. Although the remainder of the items listed above underwent a process which could result in sensitization, Westinghouse technical background and service experience, as detailed in Westinghouse topical reports, (*Reference 3*) support the conclusion that serious intergranular attack of sensitized stainless steel is unlikely in Westinghouse PWR nuclear steam supply systems, since water chemistry and contamination are kept under control. Water chemistry control is discussed in Sections 5.2.3.2 and 9.3.4.

NOTE: The primary nozzles on the replacement steam generators are integrally forged with the head. Nozzle safe ends are stainless steel forgings welded to Inconel buttering on the ends of the primary nozzles. Thus, the nozzles are not exposed to post weld heat treat temperatures.

In addition, as part of the procedures of the nuclear steam supply system supplier and RG&E, all safe ends were dye penetrant inspected after shop fabrication prior to shipping to the site and were subsequently reinspected upon completion of installation welds at the site. Also, all of the reactor coolant pressure boundary installation welds, including safe ends, were reinspected by dye penetrant upon completion of hydro and hot functional testing. No evidence of discontinuities associated with corrosion were found. Ginna LLC has and will continue to check stainless steel welds according to the inservice inspection program.

Plant Operation

Regulatory Guide 1.44 is now being used as a guide for handling, storing, and the fabrication of all stainless steel material. All welding and related activities are controlled to ensure that the chemical composition of the stainless steel is not affected. When welding is being done, the interpass temperature is maintained below 350°F to ensure the stainless steel will not become sensitized. This temperature is checked using temperature level devices during the welding fabrication process.

1.8.2.25 Regulatory Guide 1.45 -Reactor Coolant Pressure Boundary Leakage Detection System

Methods for detecting leakage from the reactor coolant system boundary are discussed in Section 5.2.5. Two radiation sensitive instruments provide the capability for detection of leakage: the containment air particulate monitor (R-11) and the less sensitive containment radiogas monitor (R-12). Additional monitors include the coolant inventory indication, containment sump A level indication (LT-2039 and LT-2044), sump A pump actuation indication, humidity detector, the condensate measuring system, and others.

Leakage from the reactor coolant system to the component cooling system would be reflected in an increase in the makeup water flow rate but not by the leakage monitors described

previously. The radiation monitor in the component cooling system would annunciate in the control room and would initiate closure of the vent line from the surge tank in the component cooling system in the event of leakage to this system.

Sensitivities of some of the systems are discussed in detail in Section 5.2.5.

Airborne radioactivity monitors alarm in the control room. Each actuation of the containment sump pump causes an alarm in the control room. Each time makeup water is added to the primary system, an alarm is sounded in the control room. The time and amount of makeup is logged by the operators.

Calibration is performed on systems at specified frequencies.

The Technical Specifications present in detail leakage limits, instrument sensitivities and limitations on instruments out of service.

1.8.2.26 Regulatory Guide 1.46 - Protection Against Pipe Whip Inside Containment

The reactor vessel, steam generators, reactor coolant pumps, and pressurizer are supported to ensure that a postulated rupture of the main reactor coolant piping does not propagate into failures of connected safety-related systems, such as the Emergency Core Cooling System (ECCS) and secondary systems. Barriers are also provided to minimize the potential for pipe whip and jet impingement.

Additional information concerning protection against dynamic effects due to postulated pipe failures in Ginna Station is provided in Section 3.6.

1.8.2.27 Regulatory Guide 1.47 - Bypassed and Inoperable Status Indication for Nuclear Power Plant Safety Systems

Regulatory Guide 1.47 and the related IEEE Standard 279-1971 were published after the construction of the Ginna plant. The IEEE Standard is, however, discussed in Section 1.8.3.

Bypassing or defeating any portion of a protective channel results in an alarm in the control room indicating the channel affected.

1.8.2.28 Regulatory Guide 1.48 - Design Limits and Loading Combinations for Seismic Category I Fluid System Components

The Ginna Nuclear Power Plant equipment was designed and analyzed to ensure structural integrity and operability. However, Regulatory Guide 1.48 had not been published at the time of the Ginna Station design and construction. The codes and procedures employed in the Ginna design have been widely used and proven adequate by the nuclear industry for the design of components in operating plants.

The valves were designed to function at normal operating conditions, maximum design conditions, and earthquake conditions per the detailed equipment specifications. The requirements of the ANSI B31.1, ANSI B16.5, and MSS-SP-66 codes were adhered to in the design. The allowable stresses in the above codes are considerably less than the limits presently proposed by the ASME Task Group on Design Criteria for Class 2 and 3 Components, e.g., the allowable stress in ANSI B16.5 is 7000 psi as opposed to the maximum limit accepted by the ASME task group of 2.4 times the ASME Section VIII

allowable stress.

Prior to shipment, the valves were subjected to hydrostatic leak tests in accordance with MSS-SP-61 and functional tests to show that the valves will open and close within the specified time limits when subjected to the design differential pressure. In addition, representative valves were checked for wall thickness to ANSI B16.5 and MSS-SP-66 requirements and subjected to nondestructive tests in accordance with ASME and ASTM codes. After installation of the valves they were subject to cold hydrostatic tests and hot functional tests to verify operation. Also, periodic inservice inspections and operation tests are performed as required.

Active pumps were designed to the requirements of the Standards of the Hydraulic Institute and/or the ASME Code for Pumps and Valves for Nuclear Power, depending on the pumps purchase order date. In addition, the pumps and their supports were designed to withstand horizontal and vertical earthquake forces.

The pumps were hydrostatically tested to 1.5 times the design pressure and were subjected to ASME Section VIII nondestructive tests. Performance tests were conducted to check the capacity, total dynamic head or pressure, and net positive suction head. After the pumps were installed in the plant, they were subjected to cold hydrostatic tests and hot functional tests to verify operation. Also, periodic inservice inspections and operation tests are performed as required.

Additional information is provided in Section 3.9 and in the specific sections of the UFSAR applicable to the fluid system components.

1.8.2.29 Regulatory Guide 1.49 -Power Levels of Water-Cooled Nuclear Power Plants

The R. E. Ginna Nuclear Power Plant is licensed to operate at 1775 MWt, the maximum calculated turbine thermal power. This is less than the guideline of 3800 MWt.

1.8.2.30 Regulatory Guide 1.50 -Control of Preheat Temperature for Welding of Low-Alloy Steel

Regulatory Guide 1.50 was published after the construction of the Ginna Nuclear Power Plant. However, the Westinghouse practice for the Ginna plant was in agreement with the requirements of Regulatory Guide 1.50, except for Regulatory Position 1(b) and 2.

In the case of Regulatory Position 1(b), the welding procedures were qualified within the preheat temperature ranges required by Section IX of the ASME Code. High quality qualification welds were obtained using the ASME qualification procedures.

In the case of Regulatory Position 2, the Ginna pressurizer and steam generators were fabricated without maintaining the preheat temperature until the postweld heat treatment had been performed. However, for the replacement steam generators, either the maximum interpass temperature is maintained four hours or the minimum preheat temperature is maintained eight hours after welding. Additionally, as required by Regulatory Position 2, the soundness of the welds is verified by an acceptable examination procedure appropriate to the weld under consideration.

In the case of the Ginna reactor vessel main structural welds, the practice of maintaining preheat until the intermediate or final postweld heat treatment was followed by the fabricator. For each of the above components, the qualification welds have shown high integrity, using the ASME Boiler and Pressure Vessel Code criteria. In all cases the welding parameters specified in the procedure were closely monitored during production welding.

Regulatory Position 4 of the guide was met for the Ginna plant in that, for ASME Section III Class 1 components, the examination procedures required by Section III and the inservice inspection requirements of Section XI were met.

Plant Operation

The recommended practice of Regulatory Guide 1.50 is followed in the format of the welding procedures used at Ginna Station. Welding procedures are designed according to the criteria outlined in Section III and Section IX of the ASME Boiler and Pressure Vessel Code. All welding procedures are qualified following the preheat, interpass temperature, and heat treatment outlined in the procedure. Production welds are controlled to ensure that the welding procedures, variables, and requirements are carried out properly.

1.8.2.31 Regulatory Guide 1.51 - Inservice Inspection of ASME Code Class 2 and 3 Nuclear Power Plant Components

The original 5-year inservice inspection program, as defined in the Technical Specifications at that time, was developed before ASME Section XI was issued. This program addressed Class 1 components only and completed its first 5-year cycle at the Spring 1974 MODE 6 (Refueling) outage. As a result of pipe whip considerations, some of the Class 2 requirements for main steam and main feedwater were fulfilled during the 1974 MODE 6 (Refueling) outage.

Following the 1974 outage, the inservice inspection program was revised to meet the new Section XI of the ASME Code and Regulatory Guide 1.51 requirements for Class 1, Class 2, and Class 3 Nuclear Plant Components.

1.8.2.32 Regulatory Guide 1.52 - Design, Testing, and Maintenance Criteria for Atmospheric Cleanup System Air Filtration and Adsorption Units of Light-Water-Cooled Nuclear Power Plants

Ginna Station was designed in conformance with the General Design Criteria in effect in 1968. The atmosphere cleanup systems were designed under the applicable criteria (i.e., 41, 52, 58, 59, 61, 62, 63, 64, 65, 70). This is discussed in Sections 9.4.1, 6.2.2, and 6.5.1. The cleanup system was designed to operate under the environmental conditions resulting from a postulated design-basis accident. All components of the cleanup system are compatible with other engineered safety features and have been designed to be consistent with radiation fields and isotopes expected during the design-basis accident. There are no components of systems in unheated compartments. Charcoal filter units are provided with spray systems to limit adsorber fires.

All cleanup systems are designed for ease of maintenance and ready removal of elements. Lighting is provided in the housings and test probe holes for in-place testing are available.

Filter units were tested prior to startup of Ginna Station and are retested according to the schedules of the Ventilation Filter Testing Program. These tests are subcontracted to a reliable vendor who prepares the report of test results. Samples from the charcoal filter trays are sent for organic iodides and elemental iodine efficiency tests according to the Ventilation Filter Test Program (ITS 5.5.10).

1.8.2.33 Regulatory Guide 1.53 - Application of the Single-Failure Criterion To Nuclear Power Plant Protection Systems

This guide endorses the use of IEEE Standard 379-1972, Trial-Use Guide for the Application of the Single-Failure Criterion to Nuclear Power Generating Station Protection Systems. Subjects which are covered in the standard include identification of undetectable failures, analysis of channel interconnections for failures which could compromise independence, testing to determine independence between redundant parts of the protection system, and analysis to show that no single failure can cause a loss of function due to improper connection of actuators to a power source.

Routing and separation standards applicable to existing cables are those that were invoked at the time of cable installation. For more information, see Section 8.3.1.4.

Protection system failure analyses and reliability studies applicable to the Ginna plant were performed as described in the topical report WCAP 7486-L, December 1970, An Evaluation of Anticipated Operational Transients in Westinghouse Pressurized Water Reactors. This report was submitted to the AEC by Westinghouse in March 1971. Subsequent evaluations have demonstrated the conformance of the Ginna Station design to this guide.

1.8.2.34 Regulatory Guide 1.54, Revision 0 - Quality Assurance Requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants

Contemporary standards were specified to ensure that protective coatings applied would perform their functions under environmental conditions experienced during MODES 1 and 2 and the design-basis accident and to do so without hazard of interfering with other nuclear components.

One standard specified was SP-5485 dated January 18, 1968, entitled Technical Specification, Painting of Structures and Equipment, Robert Emmett Ginna Nuclear Power Plant Unit No. 1, which includes techniques for preparation of surfaces to be painted, sampling, thickness measurement and control, and a detailed paint schedule including components and paint materials for plant structures and equipment. Also, SP-5339 dated March 31, 1967, entitled Technical Specification for Painting the Interior Surface of the Containment Vessel Dome for the Robert Emmett Ginna Nuclear Power Plant Unit No. 1, gives the specifications for the preparation, application, material, and paint sampling for the interior of the containment dome.

The painting of the containment structure and components inside the containment was governed by Westinghouse process specification PWR 597755, dated February 20, 1968. This specification covered the application of paint systems to equipment and structures in containments which use additive spray systems for fission product removal and/or containment cooling.

Regulatory Guide 1.54 and related ANSI Standard N101.4 were published after construction of the Ginna plant and thus were not available to be applied. However, the previously referenced process specifications demonstrate that care was taken in the selection and application of protective coatings for the Ginna plant.

Plant Operation

For new coatings and configuration changes to existing coatings, which have the potential to adversely affect a safety related function, the quality assurance requirements of 10CFR50, Appendix B, in conjunction with engineering specifications, are used instead of the detailed requirements included in this Regulatory Guide and its referenced standard, ANSI N101.4-1972.

1.8.2.35 Regulatory Guide 1.55 -Concrete Placement in Seismic Category I Structures

All concrete placement for the Ginna plant was accomplished in accordance with the proposed specification for structural concrete for buildings ACI-301 and the detailed construction specification.

In accordance with the specification, the contractor submitted placing drawings, reinforcing bar details, and bar lists, etc., for engineer approval to ensure that the details were in general compliance with the engineering drawings. Construction joints not shown on the drawings were located in accordance with the requirement of the specification and only after their influence on the structural integrity was reviewed and approved in writing by the engineer. Field generated revisions were reviewed and approved by the engineer.

The services of Pittsburgh Testing Laboratory were obtained to ensure the quality control on the job. Well before the concrete work started, representative samples of ingredients for the concrete work were tested and concrete mix design was established to conform to the design requirements. During concrete operation, the Testing Laboratory had an inspector at the batch plant who certified the mix proportions of each batch delivered to the site, took samples of the concrete ingredients, and tested them periodically. Another inspector was stationed at the construction site who inspected rebar, form placements, took slump tests, made test cylinders, checked air content, and recorded weather conditions. Cylinder tests were made in accordance with the provision of the ACI Code.

1.8.2.36 Regulatory Guide 1.57 -Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components

The Ginna containment is a composite structure as opposed to a metal primary reactor containment; thus this guide is not applicable.

1.8.2.37 Regulatory Guide 1.59 -Design-Basis Floods for Nuclear Power Plants

The R. E. Ginna Nuclear Power Plant site has been evaluated for the probable maximum flood coincident with wind and wave activity as outlined in Section 2.4.

The analysis for flood, storm, waves, and hardened protection is generally consistent with Regulatory Guide 1.59. Site Contingency Procedures are available to be implemented in the

event of potential flooding conditions. A recent review of Ginna flood protection measures described the conformance of Ginna Station to this guide.

1.8.2.38 Regulatory Guide 1.94, Revision 1 - Quality Assurance Installation, Inspections, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants

This Regulatory Guide applies to plants in the construction phase and was issued after Ginna was built. The specific details of the Ginna controls during construction are discussed in Section 17.1.

1.8.2.39 Regulatory Guide 1.143, Revision 1 - Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants

The specific UFSAR sections discuss the design and quality assurance provisions applied to existing radioactive waste management systems, structures, and components. New systems, structures, and components and configuration changes to existing items meet the design and quality assurance provisions described in the UFSAR sections or those specified by this Regulatory Guide.

1.8.3 CONFORMANCE TO IEEE CRITERIA

The information in this section is generally that submitted in the August 1972 Technical Supplement Accompanying the Application for a Full-Term Operating License as to the adequacy of the R. E. Ginna Nuclear Power Plant design with respect to IEEE Standards 279-1971, 308-1971, 317-1971, 323-1971, 334-1971, 336-1971, 338-1971, and 344-1971.

1.8.3.1 Criteria for Protection Systems for Nuclear Power Generating Stations (IEEE 279-1971)

Conformance with IEEE 279-1971 is discussed in Section 7.1.2.

1.8.3.2 Class 1E Electric Systems for Nuclear Power Generating Stations (IEEE 308-1971)

1.8.3.2.1 Principal Design Criteria

The criteria states that Class 1E electric systems shall be designed to ensure that any design-basis event as listed in Table 1 of the standard will not cause a loss of electric power to a number of engineered safety features, surveillance devices, or protection system devices sufficient to jeopardize the safety of the station. The design-basis events include earthquakes, winds, tornadoes, other natural phenomena, and various postulated accidents.

All electrical systems and components vital to plant safety, including the emergency diesel generators, are designed as Class 1E and are designed so that their integrity is not impaired by the design-basis earthquake, wind storms, floods, or disturbances on the external electrical system. Power, control and instrument cabling, motors, and other electrical equipment required for operation of the engineered safety features are suitably protected against the effects of either a nuclear system accident or of severe external environmental phenomena in

order to ensure a high degree of confidence in the operability of such components in the event that their use is required.

The preferred power supply (offsite power) has a voltage variation of not more than plus or minus 10% and a frequency variation of not more than plus or minus 0.5%. Variations of voltage and frequency of the standby power supply (diesel generators) will not degrade the performance of any load to the extent of causing significant damage to the fuel or to the reactor coolant system.

Controls and indicators are provided in the control room and locally for the standby power supply and for the circuit breakers required to switch the Class 1E buses between the preferred and standby power supply. Transfer is automatic on loss of the preferred supply.

All components of the Class 1E electric systems are identified with permanently installed equipment piece-number tags. Design, operating, and maintenance documents for each major component were identified as they were received from the equipment suppliers, and the identification associates each component with its particular system.

Class 1E electrical equipment is physically separated to the extent practical from its redundant counterpart either by distance, barrier walls, or by location on different floors.

Each type of Class 1E electric equipment was designed, manufactured, and tested in accordance with the latest standards in existence at the time of manufacture. This equipment was analyzed to ensure that it would successfully perform its function under normal and design-basis events. In addition to this, preoperational testing was performed to verify equipment operation.

Failure mode analyses have been done for all Class 1E electrical systems. These analyses show that a single component failure does not prevent satisfactory performance of the Class 1E systems required for safe shutdown and maintenance of post-shutdown or postaccident station security.

The Class 1E electric systems are described in detail in Chapter 8. The systems consist of an ac power system, a dc power system, and an instrumentation and a control system to supply acceptable power to the station for any design-basis event.

1.8.3.2.2 Alternating Current Power Systems

1.8.3.2.2.1 General

The ac power systems include power supplies, distribution systems, and load groups arranged to provide ac electric power to the Class 1E loads. Sufficient physical separation, electrical isolation, and redundancy are provided to minimize the occurrence of a common failure mode in the Class 1E systems.

The Class 1E electric system is divided into two redundant load groups. Safety actions by each group of loads is redundant and independent of the safety actions provided by its redundant counterpart. Each load group has access to both the offsite and standby power supply.

Two independent 34.5-kV transmission lines make up the preferred offsite power supply and two independent diesel generators make up the standby power supply.

1.8.3.2.2.2 Distribution Systems

By design, each distribution circuit is capable of transmitting sufficient energy to start and operate all required loads in that circuit. Distribution circuits to redundant equipment are physically and electrically independent of each other, to the extent practical.

Auxiliary devices required to operate dependent equipment are supplied from related bus sections such that loss of electric power in one load group does not cause the loss of function of equipment in another load group. By means of circuit breakers located in the auxiliary building and the screen house (both Seismic Category I structures), it is possible to disconnect portions of the Class 1E system that are located in other than Seismic Category I structures. The distribution system is monitored to the extent that it is shown to be ready to perform its intended function. The surveillance program is included in the Technical Specifications.

1.8.3.2.2.3 Preferred Power Supply

The preferred power supply consists of two 34.5-kV circuits that are independent. This system is designed to furnish the starting and operating power requirements for the shutdown of the station and for the operation of emergency systems and engineered safety features. It also functions as startup power and reserve power for all unit auxiliaries.

A minimum of one circuit is available from the transmission network during MODES 1 and 2.

1.8.3.2.2.4 Standby Power Supply

The standby power supply provides power for the operation of emergency systems and engineered safety features during and following the shutdown of the reactor when the preferred power supply is not available.

The standby sources become available automatically following the loss of the preferred power supply within a time consistent with the requirements of the engineered safety features and the shutdown systems under normal and accident conditions. A failure of any unit of standby power source does not jeopardize the capability of the remaining standby power sources to start and run the required shutdown systems, emergency systems, and engineered safety features loads.

Two 6000 gallon underground storage tanks serve only the two emergency diesel generators. These tanks have the minimum required capacity of 10,000 gallons for 48 hours operation of both diesel generators at load, simultaneously, or one diesel generator at load for 80 hours. See Section 9.5.4 for an update of this historical information. The actual load on a diesel generator needed to place the station in a safe shutdown condition is less than the full-load rating of the diesel generator. This supply allows adequate time for makeup supplies of oil if required. The standby power supplies are started and operated at specified loads on a monthly basis. This program is included in the Technical Specifications.

1.8.3.2.3 Direct Current Power Systems

1.8.3.2.3.1 General

The dc power systems include power supplies, a distribution system, and load groups arranged to provide dc electric power to the Class 1E dc loads and for control and switching of the Class 1E systems. Sufficient physical separation, electrical isolation, and redundancy are provided to minimize the occurrence of common failure modes in the station Class 1E systems and include the following:

a. The electric loads are separated into two redundant load groups.

b. Safety actions by each group of loads are redundant and independent to the safety actions provided by its redundant counterpart.

c. Each redundant load group has access to a battery and two battery chargers.

These items are discussed in Chapter 8.

1.8.3.2.3.2 Distribution System

Each distribution circuit is capable of transmitting sufficient energy to start and operate all required loads connected to it. Distribution circuits to redundant equipment are independent of each other to the extent practical. Auxiliary devices required to operate dependent equipment are supplied from a related bus section to comply with this criterion. It is possible to disconnect portions of Class 1E systems located in Seismic Category I structures from those portions located in other than Seismic Category I structures. The disconnecting means are located in distribution panels in the Seismic Category I battery rooms. The system is monitored with indicators and alarms in the control room to the extent that it is shown to be ready to perform its intended function.

1.8.3.2.3.3 Battery Supply

Each battery supply consists of storage cells, connectors, and connections to the dc distribution system supply breaker. Each battery supply is independent of the other supply and is capable of starting and carrying all required loads. Each battery supply is immediately available during MODES 1 and 2 and following the loss of power from the ac system.

Each battery is kept fully charged and floating across its battery charger. Stored energy is sufficient to operate all necessary breakers to provide an adequate source of power for all connected loads. Battery instrumentation located in the control room indicates the status of the battery supplies.

1.8.3.2.3.4 Battery Charger Supply

The battery chargers provide all the dc power required for normal station operation as long as ac power is available. Each battery can be supplied by a full capacity charger or a full capacity backup charger. Each full capacity charger has sufficient capacity to restore the battery from the design minimum charge to its fully charged state while supplying normal steady-state loads. The two supplies are independent of each other. The capability for isolating each charger is provided by means of circuit breakers in the ac feeder and the dc output circuit.

1.8.3.2.3.5 Protective Devices

Protective devices are provided to isolate failed equipment automatically. Indication is also provided to identify the equipment that is made unavailable.

1.8.3.2.3.6 Performance Discharge Test Provisions

To be sure that all cells, connections, jumpers, etc., satisfactorily handle full-rated current if necessary, each battery has been tested under full load and each component individually examined.

1.8.3.2.4 Vital Instrumentation and Control Power Systems

Dependable power supplies are provided for the vital instrumentation and control systems of the unit including the following.

A. The nuclear plant protection instrumentation and control systems.

B. The engineered safety features instrumentation and control systems.

Power is supplied to these systems in such a manner as to preserve their reliability, independence, and redundancy.

1.8.3.2.5 Surveillance Requirements

Preoperational Equipment Tests and Inspection

The initial equipment tests and inspections were performed with all components installed. They demonstrated the following:

C. All components were correct and properly mounted.

D. All connections were correct and circuits were continuous.

E. All components were operational.

F. All metering and protective devices were properly calibrated and adjusted.

Initial System Test

The initial system test was performed with all components installed. The test demonstrated the following:

A. The Class 1E loads can operate properly on the preferred power supply.

B. The loss of the preferred power supply can be detected.

C. The standby power supply can be started automatically and can accept design load within the design-basis time.

D. The standby power supply is independent of the preferred power supply.

Periodic Tests

The periodic test programs are included in the Technical Specifications. Tests are performed at scheduled intervals to

A. Detect possible deterioration of the system toward an unacceptable condition.

B. Demonstrate that standby power equipment and other components that are not exercised during MODES 1 and 2 of the station are operable. If surveillance tests indicate that any Class 1E systems are degraded, the Technical Specifications impose operating limitations.

1.8.3.3 Electrical Penetration Assemblies in Containment Structures for Nuclear Fueled Power Generating Stations (IEEE 317 - April 1971)

Electrical penetrations are designed and demonstrated by test to withstand, without loss of leak tightness, the containment post-accident environment and meet the following guide that was available during construction: IEEE Proposed Guide for Electrical Penetration Assemblies in Containment Structures for Stationary Nuclear Power Reactors (Eighth Revision). The electrical penetration sleeves, being part of the containment vessel, were designed in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Subsection B, for Class B vessels.

The penetration assemblies are qualified to prevent leakage from the containment under the worst-case environmental conditions associated with a loss-of-coolant accident or main steam line break.

All welded joints for the penetrations including the reinforcement about the openings are fully radiographed in accordance with the requirements of the ASME Nuclear Vessel Code for Class B Vessels except that non-radiographable joint details are examined by the liquid penetrant method. Verification of leak tightness is by means of pressurizing test channels.

There are generally five types of electrical cable penetrations required depending on the type of cable involved:

- Type 1 - High voltage power 4160 V.
- Type 2 - Power, control and instrumentation; 600 V and lower.
- Type 3 - Thermocouple leads.
- Type 4 - Coaxial and triaxial circuits.
- Type 5 - Fiber Optic

All five types of penetration designs are a cartridge type. The cartridge length and the supporting of cables immediately outside containment are designed to eliminate any cantilever stresses on the cartridge flange.

The specification for penetrations cover all aspects of equipment design, manufacture, inspection, qualification, and testing.

1.8.3.4 Qualifying Class I Electric Equipment for Nuclear Power Generating Stations (IEEE 323-April 1971)

The components of the protection system are designed and qualified so that the mechanical and thermal environment accompanying any emergency situation in which the components are required to function does not interfere with that function.

The equipment that must withstand the most severe environment is that which is in the containment. The instrumentation, motors, cables, and penetrations located inside containment are either protected from containment accident conditions or are designed to withstand, without failure, exposure to the worst combination of temperature, pressure, and humidity expected during the required operational period.

Quality standards of material selection, design, fabrication, and inspection governing the above features conformed to the applicable provisions of recognized codes and good nuclear practice.

1.8.3.5 Type Tests of Continuous Duty Class I Motors Installed Inside the Containment of Nuclear Power Generating Stations (IEEE 334-1971)

Of those motors installed within the containment of Ginna Station only the motors on valve operators and the fan motors of the containment air recirculation, cooling, and filtration system are required to be Class I. The valve motors, however, are not subjected to continuous duty. Therefore, IEEE 334-1971 does not apply to them.

The containment recirculation fan cooler (CRFC) and filtration system fan motors are continuous duty. The fans, motors, electrical connections, and all other equipment in the containment necessary for operation of the system are capable of operating under the environmental conditions following a loss-of-coolant accident. These environmental conditions are defined in Section 3.11.

All components are capable of withstanding or are protected from differential pressure which may occur during the rapid pressure rise to 60 psig in 10 seconds.

Any single active component failure in the system will not degrade the overall required heat removal capability.

Overload protection for the fan motors is provided at the switchgear by overcurrent trip devices in the motor feeder breakers. The fan motor feeder breakers can be operated from the control room and can be reclosed from the control room following a motor overload trip.

1.8.3.6 Installation, Inspection, and Testing Requirements for Instrumentation and Electric Equipment During the Construction of Nuclear Power Generating Stations (IEEE 336-1971)

An evaluation of prospective suppliers was conducted prior to awarding of a contract for important components. This evaluation established that the supplier has acceptable design, manufacturing, and quality control capability. The supplier was provided with individual equipment specifications covering all aspects of equipment design, manufacture, inspection, and testing. For Class 1E components, such as those in the reactor coolant system, a specification which defined the quality control requirements was made a part of each purchase order.

The instrumentation and electrical equipment for engineered safety features and reactor protection were subjected to receiving inspection, pre-installation operability and calibration checks, and preoperational functional and calibration tests. The quality assurance requirements during construction are described in Chapter 17; initial tests are described in Chapter 14.

1.8.3.7 Trial Use Criteria for the Periodic Testing of Nuclear Power Generating Station Protection Systems (IEEE 338-1971)

The station has the capability for sensor checks, channel tests, and channel calibration. The testing program is based on the calculations that were presented on the basis of the Technical Specifications.

All protective instrumentation has the capability of being tested and calibrated. Instrumentation that requires testing between reactor shutdowns also has the capability for being tested during MODES 1 and 2. The satisfactory operation of each redundant channel may be verified and credible failures can be detected. A scheduled test program is presented in the Technical Specifications.

All sensor checks and tests are either done by perturbing the monitored variable, introducing a substitute input, or comparing sensors which measure like variables. The test signal amplitude is varied to determine that the protective action will occur when the setpoint is reached. These setpoints include the effects of instrumentation errors.

Written procedures are maintained for all tests. The results are documented and records are kept.

1.8.3.8 Seismic Qualification of Class I Electrical Equipment for Nuclear Power Generating Stations (IEEE 344-1971)

All systems and components designated Class I are designed so that there is no loss of function in the event of the design-basis earthquake ground acceleration acting in the horizontal and vertical directions simultaneously. Subsequent reviews of the qualification of this equipment is described in Section 3.10.

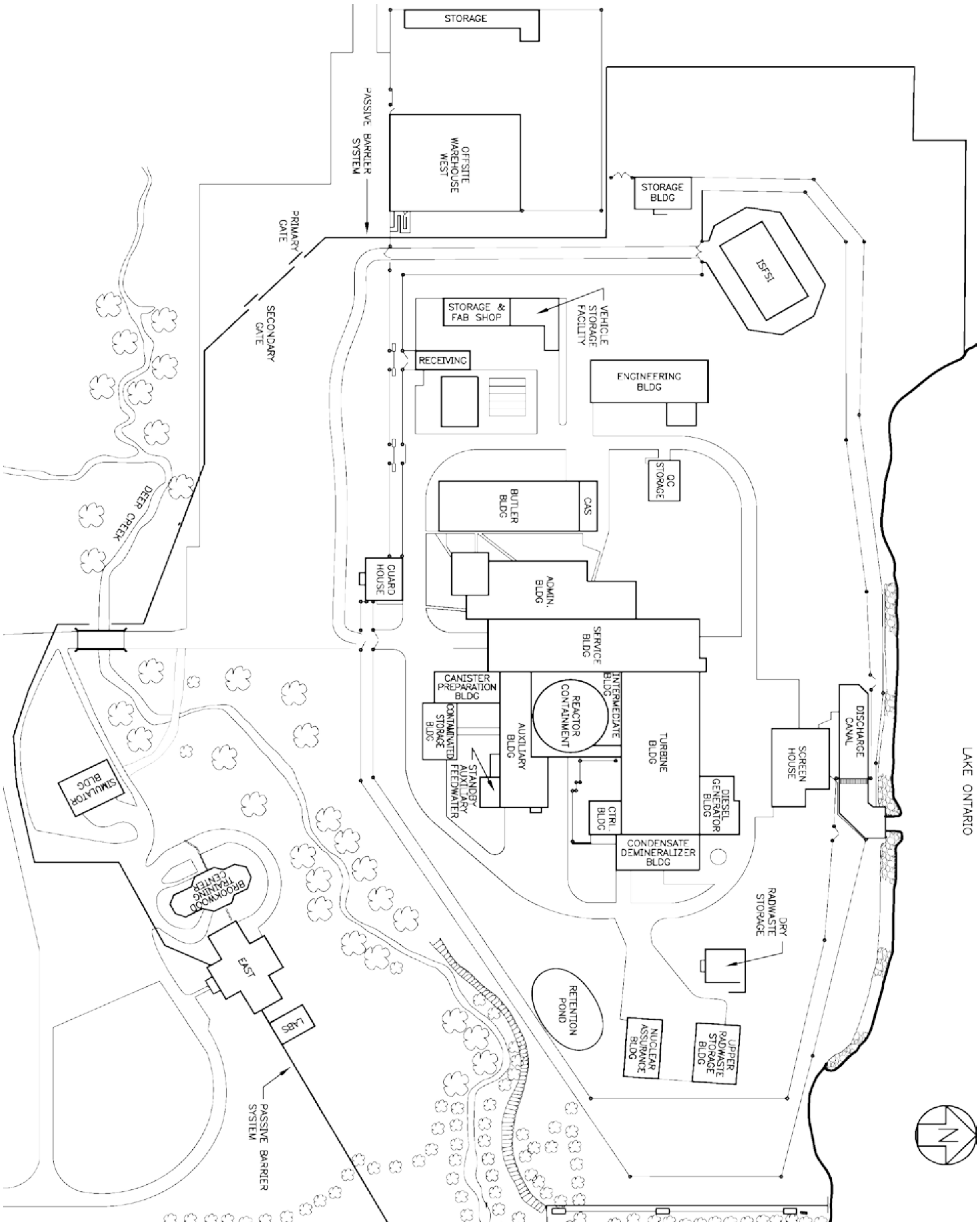
REFERENCES FOR SECTION 1.8

1. Ernest L. Robinson, Bursting Test of Steam-Turbine Disk Wheels, Transactions of ASME, July 1944.

2. Gilbert Associates, Inc., Structural Integrity Test of Reactor Containment Structure, GAI Report No. 1720, October 3, 1969.

3. Westinghouse Electric Corporation, Sensitized Stainless Steel in Westinghouse PWR Nuclear Steam Supply Systems, WCAP 7477-L, WCAP 7477-L Addendum 1, WCAP 7735, May 15, 1973.

Figure 1.2-1 Ginna Station Plot Plan



2 SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 SITE LOCATION AND DESCRIPTION

The site is in the township of Ontario, in the northwest corner of Wayne County, New York, on the south shore of Lake Ontario about 16 miles east of the center of the city of Rochester and 40 miles west-southwest of Oswego, at longitude 77°18.7’W and latitude 43° 16.7’N. The general location is shown in Figure 2.1-1.

The site comprises approximately 426 acres owned by Ginna LLC. Figure 2.1-2 shows the site and its relationships to topographic features.

The surface of the land on the southern shore of Lake Ontario, at the site and east and west of it, is either flat or gently rolling. It slopes upward to the south from an elevation of about 255 ft above mean sea level (msl) near the edge of the lake; to 440 ft at Ridge Road (New York State Highway 104), 3.5 miles south of the lake; and then to about 1600 ft at the northern edge of the Appalachian Plateau, 30 to 40 miles to the south. Southward from Ridge Road the terrain progressively roughens, with a series of small abrupt hills, commencing about 10 miles south of the site.

Wayne County, in which the site is located, is primarily of an agrarian nature and sparsely populated. The location is shown in Figure 2.1-1. There are no substantial population centers, industrial complexes, transportation arteries, parks or other recreational facilities within a 3-mile radius of the Ginna site (*Reference 1*). Roughly 70% of the county’s 600 square miles are utilized for approximately 2500 farms, which primarily produce apples, grapes, cherries, dairy products, field crops, and vegetables. About 34% of Wayne County’s workers are employed in manufacturing operations, 18% in service industries, 16% in retail trade, 14% in agriculture, and 18% in other occupations. Typical industries are listed in Table 2.2-1.

Monroe County, located adjacent to and west of Wayne County, has many manufacturing activities centered in and around Rochester. Approximately 22% of the county’s 673 square miles is in urban development, about 28% is vacant, wooded, or water surface, and 50% is farm land upon which dairy products, field crops, poultry, livestock, fruits, and horticultural specialties are produced. Of Monroe County’s workers, about 45% are employed in manufacturing, 20% in service industries, 16% in retail trade, 1.4% in agriculture, and the rest in other activities. Typical industries are listed in Table 2.2-2.

The land within a radius of 5 miles of the site is used for agricultural purposes, principally for growing apples, cherries, grapes, and field crops. There are only a few dairy farms in a 5-mile radius of the plant. They average between 50 to 75 milk cows per farm. Part of the site is under lease for fruit farming.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

The site boundary is the line beyond which the land is neither owned, nor leased, nor otherwise controlled by Ginna Station (see Figure 2.1-2). The exclusion area is completely within the site boundary (see Figure 2.1-2). The distance from the containment to the nearest exclusion area boundary (EAB) (excluding the boundary on the lakefront) is 1550 ft but the minimum exclusion distance is assumed to be 450 meters or 1476 ft. No public highways or railroads traverse the exclusion area.

The Constellation Energy Nuclear Group, LLC (CENG) owns and controls all of the land, including mineral rights, within the exclusion area. Technical Specification Amendment 115 was issued on April 1, 2014, which approved the transfer of the license for R.E. Ginna Nuclear Power Plant (Ginna) held by R.E. Ginna Nuclear Power Plant, LLC, (Ginna LLC) to Exelon Generation Company, LLC, as approved by Order dated March 24, 2014. The joint venture held between Constellation Energy Nuclear Group, LLC, (CENG) and Electricite de France, S.A., was not modified as part of Amendment 115. The joint venture consists of a 50.01% ownership interest of an ultimate domestic parent Exelon Generation Company, LLC, and a 49.99% ownership interest of an ultimate foreign parent, Electricite de France, S.A., a French Corporation (*Reference 8*). Regarding the lakeshore frontage within the exclusion area, CENG, by New York State procedures (*Reference 2*), owns the land above 243.8 ft msl. This is well below the average lake stage of 246 ft msl, but is above the extreme low water level of 242.23 ft msl and the lowest regulated level of 243 ft msl (see Section 2.4); however, since the low period is generally in the winter and the high period in the summer, it is not expected that there would be any beach use of this area. The exclusion area is not defined over the waters of Lake Ontario adjacent to the Ginna site. While CENG has not specifically defined an exclusion area over the water, arrangements have been made with the U.S. Coast Guard, as documented in the Ginna Nuclear Emergency Response Plan, for emergency response in the event of a plant emergency.

CENG has established a security zone on the waters of Lake Ontario for the purpose of excluding watercraft in the vicinity of the waters that surround the plant. The boundaries of the security zone have been established by the U.S. Coast Guard and the boundaries are marked by a system of buoys. The buoys are removed during winter months to prevent them from becoming a boating hazard if they were to break free as a result of ice or winter snow. The establishment of this security zone complies with the requirements of an NRC Order (*Reference 7*), and 33 CFR 165.911 (a)(2).

2.1.3 POPULATION DISTRIBUTION

2.1.3.1 Population Within Five Miles

The population distribution by 1-mile increments within 5 miles of the plant, projected for the years 1970, 1980, 1990, and 2010, is shown in Figure 2.1-4. The 1970 estimates were based on a 1967 count of houses and electric meters and includes summer residents. The estimates for 1980, 1990, and 2010 were made by the Rochester Gas and Electric Rate and Economic Research Department and were derived from a study of past trends and probable future industrial, commercial, residential, and recreational development.

Updated population data based on preliminary estimates from the 1980 Census (*Reference 3*) are shown on Figure 2.1-5. Rochester Gas and Electric Corporation estimated that 10,864 persons resided within 5 miles of the plant in 1980, a density of 138 persons per square mile averaged over the entire area. It should be noted that this figure compares favorably with the 1980 population projection of 10,934 persons shown in Figure 2.1-4.

Updated 1992 population estimates based on data obtained from the Center for Government Research and 1990 Census data are shown in Figure 2.1-5a. Rochester Gas and Electric Corporation estimated that 11,277 persons resided within 5 miles of the plant in 1992. It should be noted that this figure is significantly lower than the 1990 population projection of 14,491 persons shown in Figure 2.1-4.

Based on the original FSAR for Ginna Station published in 1968, four schools were located approximately 3.5 miles south of the plant, and had a total enrollment of 2272 pupils and a teaching staff of 180. The nearest offsite residence is about 2000 ft southwest of the plant, and there are two occupied farmhouses on the site. The farms are owned by RG&E and the occupants have leases renewable annually at the option of RG&E. One farmhouse is about 2200 ft southeast of the plant and the other is about 1500 ft south. Both farmhouses are outside the exclusion area. Other buildings (horse barns) are located about 800 ft east and 1400 ft south of the plant.

2.1.3.2 Population Within Forty Miles

The population distribution projections by 10-mile increments within 40 miles of the plant, for the years 1970, 1980, 1990, and 2010, are shown in Figure 2.1-6. The 1970 estimates were based on extrapolations of the 1960 Census and a special census of Monroe County (Rochester area) dated April 1, 1964. The estimates for 1980, 1990, and 2010 were made by the RG&E Rate and Economic Research Department and were derived from a study of past trends and probable future industrial, commercial, residential, and recreational development.

2.1.3.3 Transient Population

Based on the original FSAR, there is a summertime increase of about 500 people in the lakeside population within a 5-mile radius of the plant, and a summer-time increase of 4000 to 5000 people in the lakeside population within a 20-mile radius of the plant. The nearest group of houses are summer cottages, 0.8 miles west. Other groups are located at Bear Creek, 1.5 miles east, and at Ontario-on-the-Lake, 2 miles west.

Other than the summertime residents of the area, there are no large groups of transients within 5 miles of the site. The only parks near the site are Webster Beach Park in Monroe County, approximately 6 miles west of the plant site, and B. Forman Park in Wayne County, approximately 8 miles east of the plant site. There are no federal recreational facilities in the area. There are no state parks, public campsites, or special use areas within 10 miles of the plant (*Reference 3*). Wayne County does have a migrant labor population during the June-October season, primarily for apple picking. Approximately 115 farm-worker camps of five or more persons are scattered throughout Wayne County, with a total population of about 4400 migrants. Information from Rural New York Farmworker Opportunities shows that there are only 12 camps, with about 130 migrants, located in the vicinity of the Ginna site (*Reference 4*).

2.1.3.4 Low-Population Zone

The low-population zone specified for the Ginna site is the area within a 3-mile (4827 m) radius of the plant (*Reference 5*). A review in 1981 of population estimates and projected

growth estimates indicates that the population growth in the area since the plant received an operating license in 1969 has been modest, and this trend is expected to continue. No population center of 25,000 residents has developed, or appears likely to develop, closer than the eastern boundary of the Rochester urbanized area.

2.1.3.5 Population Center

Figure 2.1-7 shows the locations of population centers (over 25,000 people) within a radius of 100 miles of the plant site. Figure 2.1-8 shows the locations and sizes of population centers of over 2000 people within a radius of 50 miles. These figures are based on the 1960 census, except the Rochester urbanized area, which is based on the 1980 census. There has been no significant change in population since that time.

The nearest population center to the Ginna site containing more than 25,000 residents is the Rochester urbanized area, whose eastern boundary is about 10 miles from the site (*Reference 1*). The only other population center of more than 25,000 persons is the city of Auburn (population 32,442) (*Reference 3*), located more than 40 miles southeast of the site.

2.1.3.6 1989 Updated Population Data

RG&E reviewed Ginna Station projected population changes through the year 2009 in support of the October 5, 1989, application for an extension of expiration of the Ginna Operating License from April 25, 2006, to September 18, 2009 (*Reference 6*). RG&E obtained 1984 population data for the thirteen county area included within a 50-mile radius of the plant. The population in this area had increased by only 3% overall since 1970, which was substantially below the RG&E 1970 estimates for 1984. The 1980 population within 2 miles of the plant was 1078 people. This population was estimated to increase to 1390 by the year 2015 based on the 1980-1985 population growth rate for Wayne County. The population centers with populations greater than 25,000 people, within the 50-mile radius of the plant, continued to be Monroe County, which includes the city of Rochester (Rochester 1984 population equaled 243,000), and the city of Auburn, New York. Population projections for the year 2015, based on the 1970-1980 growth rates, were as follows:

		Population	
Population Center	**Location**	**1984**	**2015**
Monroe County	20 miles WSW	711,200	742,100
Auburn, New York	45 miles ESE	32,000	35,000

REFERENCES FOR SECTION 2.1

1. Rochester Gas and Electric Corporation, R. E. Ginna Nuclear Power Plant Unit No. 1, Environmental Report, Volume 1, Sections 2.1 and 2.2.

2. New York State Policy as established by the Land Utilization Department within the New York State Office of General Services.

3. U.S. Census Bureau, "General, Social, and Economic Characteristics," Characteristics of Population, TC80-1-C34, U.S. Government Printing Office, November 1983.

4. Letter from Thomas J. Harris, Rural New York Farmworker Opportunities, to George Wrobel, RG&E, April 10, 1981.

5. Safety Evaluation by the Division of Reactor Licensing, U.S. Atomic Energy Commission in the Matter of Rochester Gas and Electric Corporation Robert Emmett Ginna Nuclear Power Plant Unit No. 1, Docket No. 50-244 (SER), Section 2.1, June 19, 1969.

6. Letter from A. R. Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Environmental Assessment - Ginna Nuclear Power Plant, dated April 17, 1991.

7. NRC Order from S. J. Collins, NRC, to P. S. Wilkens, RG&E, Subject: Issuance of Order for Interim Safeguards and Security Compensatory Measures for - R. E. Ginna Nuclear Power Plant, dated February 25, 2002.

8. Letter from Nadiyah S. Morgen, NRC, to Mary G. Korsnick and Bryan P. Wright, Constellation Energy Group: R.E. Ginna Nuclear Power Plant – Issuance of Amendment to Conform the Renewed Facility Operating License to Reflect the Direct Transfer of Operating Authority (TAC No. MF2588), dated April 1, 2014.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES

There is little industrial activity in the vicinity of the R. E. Ginna Nuclear Power Plant. Wayne County, where Ginna Station is located, is primarily a rural area. Typical industries in Wayne County and Monroe County are listed in Tables 2.2-1 and 2.2-2, respectively. Industrial activity is most heavily concentrated in the town of Webster, about 6 miles from the site, and consists primarily of light manufacturing (Xerox copiers). No industrial development is expected to occur in the vicinity of the Ginna site.

The nearest transportation routes to the plant are Lake Road and U.S. Route 104, which pass about 1700 ft and 3.5 miles, respectively, from the plant at their closest points of approach. The highway separation distances at Ginna Station exceed the minimum distance criteria given in Regulatory Guide 1.91, Revision 1, and, therefore, provide reasonable assurance that transportation accidents resulting in explosions of truck-size shipments of hazardous materials will not have an adverse effect on the safe operation of the plant. Any large quantities of hazardous material would be shipped via U.S. Route 104 which is sufficiently distant (3.5 miles from the plant site) not to be of concern.

2.2.2 DESCRIPTION

The effects of nearby railroads, pipelines, waterways, and airports, and the effects of stored chemicals onsite and offsite are discussed in the following sections.

2.2.2.1 Railroads

The railroad nearest to the plant is the Ontario Midland Railroad about 3.5 miles to the south. Comparing this distance with the guidance provided in Regulatory Guide 1.91, potential railroad accidents involving hazardous materials are not considered to be a credible risk to the safe operation of the plant.

2.2.2.2 Pipelines

The nearest large pipelines to the plant are a 12-in. gas line located about 6 miles southwest of the plant and a 16-in. gas line located about 10 miles south of the plant. These pipelines are far enough away to ensure that pipeline accidents will not affect the safety of the plant. The gas line service to the Ginna house heating boiler and the boiler controls were reviewed and compared with National Fire Protection Association (NFPA) 85 and were found acceptable (*Reference 1*). On the basis of the resolution of all gas line items during the fire protection review, the gas line on the plant site does not present a safety hazard. Fire protection is discussed in Section 9.5.1.

2.2.2.3 Waterways

There are no large commercial harbors along the southern shore of Lake Ontario near the plant. Some freight is shipped through Rochester harbor about 20 miles to the west. Major shipping lanes in the lake are located well offshore, at least 23 miles or more from the plant.

As discussed in the NRC Safety Evaluation Report for SEP Topic II-1.C, shipping on Lake Ontario is not considered to be a hazard to the plant.

The possibility of shipping damage to the service water intake structure has also been considered (*Reference 2*). The intake system (Section 10.6.2) is completely submerged below the surface of the lake. A 10-ft reinforced-concrete-lined tunnel, driven through bedrock, extends 3100 ft north from the shoreline. The tunnel rises vertically and connects to a reinforced concrete inlet section. The occurrence of historical low water level will result in a depth of water of 30 ft at the inlet and with 15 ft of cover over the intake structure. This is sufficient to prevent damage from any boating which might pass in the vicinity of the structure. Furthermore, plugging of inlet water flow by a single large piece of material is prevented by the design of the intake structure, in that water enters on a full 360-degree circle (*Reference 2*). Another design feature at Ginna Station which ensures continued availability of essential service water is that service water intake can be directly drawn from the discharge canal, which is located on the plant site, protected from any potential lake boating. Thus, lake navigation is not considered to be a hazard to the plant.

2.2.2.4 Airports

The closest airport to the plant is the Williamson Flying Club Airport, a small, privately owned, general aviation facility located approximately 10 miles east-southeast of the plant. In 1981, the airport had one paved runway, designated 10-28 and oriented in an almost east-west direction, which was 3377 ft long and 40 ft wide. The runway is equipped with low-intensity runway lights. The airport has instrument approach capability to runway 10-28 from the Rochester VORTAC. There is no control tower at this airport. The airport is used for general aviation activities such as business and pleasure flying, and for agricultural spraying operations. As of 1981, there were 5000 operations per year at the facility and about 30 based aircraft, including part-time based crop dusters. The great majority of the aircraft are single-engine propeller airplanes, which typically weigh on the order of 1500 to 3600 lb. The small number of operations at this airport is substantially less than the criteria in Section III.3 of Section 3.5.1.5 of the Standard Review Plan (SRP) and is sufficiently small that in the SER for SEP Topic II-1.C the NRC staff determined that their operations are not a potential hazard.

Monroe County Airport, in Rochester, New York, about 25 miles southwest of the plant, is the nearest airport with scheduled commercial air service. The NRC has reviewed the probabilities for an airline crash from the low-altitude Federal airways in the vicinity of Ginna Station. The calculated probabilities are 5.1×10^{-8} for airway V2 and 1.4×10^{-8} for airway V2N. (The current FAA designation is airway V483, vice V2N.) Because both probabilities are less than the 1×10^{-7} acceptance criteria, the NRC concluded in the Safety Evaluation Report for SEP Topic II-1.C, dated September 29, 1981, that the probability of a commercial air traffic crash at Ginna is acceptable.

2.2.2.5 Military Facilities

Air Force Restricted Area R-5203 is located about 8 miles north of the plant site. Whenever flight activity is conducted by the Air Force within R-5203, radar surveillance is maintained by the 174th Fighter Wing, the 108th Tactical Control Group, or possibly the Cleveland Air

Route Traffic Control Center. Pilots rely upon onboard navigational equipment to maintain their presence within the specified limits of the restricted area. Pilots can also be advised if their aircraft stray beyond their limits by the radar surveillance unit covering the area at the time. The restricted area is used for military flight training which includes high-speed interceptor training maneuvers, operational flight checks, and air-to-air refueling. The current altitude ranges in 1981 were from 2000 to 50,000 ft above the surface.

There is also an inactive slow-speed low altitude military training route (SR-826) which passes about 6 miles west of the plant. Route SR-826 is not currently a military controlled air space. Acceptance criterion II.2 of Standard Review Plan 3.5.1.6 states that, for military air space, a minimum distance of 5 miles is adequate for low-level training routes, except those associated with unusual activities such as practice bombing. Air Force Restricted Area R-5203 is about 8 miles away at its closest boundary, and no unusual activities, such as bombing practice, take place. The inactive slow-speed low altitude military training route SR-826 is about 6 miles from the plant. Therefore, this criterion is met.

2.2.2.6 Toxic Chemicals

An onsite and offsite toxic chemical evaluation was performed by RG&E in response to the requirements of NUREG 0737, Item III.D.3.4 (*Reference 3*). Sources of chemicals identified during and following the chemical survey and the associated chemical hazards evaluation are discussed below and in *Reference 5*.

2.2.2.6.1 Onsite Toxic Chemicals

A. A 500-gal anhydrous ammonia tank was located next to the all-volatile-treatment building about 40 m from the control room intake. The tank would have posed a problem with respect to control room concentrations following a postulated tank or line rupture. This tank has been removed.

B. Two 6000-gal tanks (one containing 98% H_2SO_4, the other containing 50% NaOH) are located in the all-volatile-treatment building, about 40 m from the control room. Two similar tanks were located in the primary water treatment facility about 100 m from the control room intake. These tanks were permanently removed per PCR-2006-0017.

   The all-volatile-treatment tanks are contained in separate areas of large enough volume to contain the entire contents of both tanks. Each area is drained to a common sump through separate lines. Valves in the lines are maintained in the closed position so that no mixing of the H_2SO_4 and the NaOH is likely to occur. H_2SO_4 is not considered a hazard to the control room operator unless heated as a result of dilution or mixture with the caustic. Neither is likely to occur.

C. Several 55-gal drums of 30% NH_4OH, 50-gal drums of 15% NH_4OH and 5% N_2H_4, and a 35-gal drum of 35% N_2H_4 are located in the turbine building about 75 m from the control room intake. Also, a variety of gas bottles are maintained throughout the plant. The drums of NH_4OH and N_2H_4 are dilute and stored in small quantities and thus are not considered a hazard. The individual bottles do not pose a threat to the control room operators and

there is no potential identified for damage to a large number of bottles as the result of a single event.

D. There are two Halon 1301 systems for fire protection. The fire control agent is Bromotrifluoromethane which is stored in tanks outside the relay room. This agent is not considered a toxic hazard except as an asphyxiant. The gas is much heavier than air and unless it is stirred up, it will settle to the floor. The control room is above the relay room. The system should not be activated unless a fire has been detected isolating the control room from the relay room. However, if it is assumed that half the gas (640 lb) is injected into an unisolated relay room and that the gas is well mixed, concentrations as high as 2×10^5 mg/m^3 may be attained. This is less than the generally accepted limit for protective action (requiring use of self-contained breathing apparatus) of 5.9×10^5 mg/m^3. The Halon 1301 system does not pose a threat to control room habitability.

E. A plastic tank containing a maximum of 3000 gal of sodium hypochlorite (NaOCl) at a concentration of up to 17% by weight is located east of the screen house. The tank is situated on a foundation slab approximately 3 ft below grade and is surrounded by a reinforced concrete containment dike. Postulated rupture of the tank yields a negligible concentration outside of the control room, primarily because of the low volatility of the chemical. Sodium hypochlorite is not considered a threat to control room habitability (*Reference 3 and 5*).

   The original underground sodium hypochlorite tank (*Reference 3*) has been abandoned and closed in place, in compliance with the requirements of 6 NYCRR Part 598.10(c) (*Reference 4*).

F. Two 350-gal ethanolamine (ETA) tanks are located in the turbine building basement outside of the turbine pump room. The ETA is stored and injected at a concentration not to exceed 80% solution strength. The storage and processing system is contained such that any spill will not come into contact with any plant materials that are not compatible with 80% ETA and any spill will not come within 50 feet of a control room air intake.

2.2.2.6.2 Offsite Toxic Chemicals

A. The town of Ontario water plant, about 1.1 miles from the site, stores chlorine in two 2000-lb tanks. One tank is refilled each month from a truck containing 2750 lb of chlorine housed in a 2000 lb cylinder and five 150-lb cylinders.

   The chlorine tanks may pose a hazard to control room habitability following a postulated catastrophic rupture with stable meteorology. This hazard is discussed in Section 6.4.3.2.1 and in *Reference 5*. The truck which refills the chlorine tanks transports the chlorine via Route 104 and poses no hazard more severe than that discussed in Section 6.4.3.2.1 and in *Reference 5*.

B. Chemicals used by local fruit growers are transported to local distribution firms about 50 times per year. These chemicals are generally solids stored in small containers. They are not stored in large quantities anywhere in the Ginna Station area.

C. The Monroe County Water Authority operates a pumping station that is approximately 4.1 miles from the site. The pumping station contains a tank of Sodium Permanganate with 6,000 gallons working capacity and a tank of Sodium Hypochlorite with a working capacity of 6,000 gallons.

   The Sodium Permanganate tank poses no hazard to control room habitability. The Sodium Hypochlorite volume is below the level of concern for control room habitability.

REFERENCES FOR SECTION 2.2

1. U.S. Nuclear Regulatory Commission, Fire Protection Safety Evaluation Report, Supplement 2, February 6, 1981.

2. Rochester Gas and Electric Corporation, Technical Supplement Accompanying Application for a Full-Term Operating License, August 1972.

3. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: NUREG 0737 Requirements, dated September 4, 1981.

4. Letter from K. Sahler, RG&E, to New York State Department of Environmental Conservation (NYDEC), Hazardous Substance Bulk Storage Registration Number 8-000170 Rochester Gas and Electric Corporation's Ginna Station, dated March 11, 1997.

5. Constellation Energy Corporation, Design Analysis, DA-NS-2000-053, Revision 1, Control Room Toxic Hazards Analysis, dated December 14, 2004.

Table 2.2-1

TYPICAL INDUSTRIES IN WAYNE COUNTY (CIRCA 1969)

Company and Product	Distance From Site (miles)	Direction From Site
National Distillers & Chemical Corporation (Kordite Division)	14.5	South
Macedon		
Polyethelene products		
Duffy-Mott Company, Incorporated	8.5	Southeast
Williamson		
Baby foods		
Garlock, Incorporated	15.0	Southeast
Palmyra		
Mechanical packings		
Bloomer Bros. Company	19.0	Southeast
Newark		
Folding paper boxes		
Jackson Perkins Company	19.0	Southeast
Newark		
Nurserymen		
Sarah Coventry, Incorporated	19.0	Southeast
Newark		
Direct-mail sales of costume jewelry		
National Biscuit Company (Dromedary Company Division)	19.0	Southeast
Lyons		
Cake mixes, dates, and peels		

Company and Product	Distance From Site (miles)	Direction From Site
General Electric Company	27.5	Southeast
Clyde		
Electronic equipment		
Comstock Foods, Incorporated	31.0	East
Red Creek		
Canned foods		
Kenmore Machine Products, Incorporated	22.0	Southeast
Lyons		
Refrigerant products		
Olney & Carpenter, Incorporated	27.5	East
Wolcott		
Canned foods		
C. W. Stuart & Company	19.0	Southeast
Newark		
Nurserymen		
Francis Leggett Company	12.5	East
Sodus		
Canned foods		
The Waterman Food Products Company	3-4	South
Food processing		
Ontario Kraut Corporation	3-4	South-south-west

Company and Product	**Distance From Site (miles)**	**Direction From Site**
Food processing		
Victor Preserving Company	3-4	South
Food processing		
Ontario Cold Storage	3-4	South-south-west
Food processing		
Waterman Fruit Products Company	3-4	South-south-west
Food processing		
Ontario Food Products	3-4	South-south-west
Food processing		
Lyndan Products Company	3-4	South-south-west
Food processing		

Table 2.2-2

TYPICAL INDUSTRIES IN THE ROCHESTER AREA OF MONROE COUNTY (CIRCA 1969) (LOCATED 18 MILES WEST OF THE SITE)

Associated Dry Goods Corporation (Sibley, Lindsay & Curr Company subsidiary)	department store
Bausch & Lomb, Incorporated	optical instruments and lenses
Bond Stores, Incorporated	men's and boys' apparel
Burroughs Corporation (Todd Company Division)	business forms
Eastman Kodak Company	photographic equipment
Fashion Park Incorporated	men's and boys' apparel
Friden, Incorporated (Commercial Controls Corporation subsidiary)	special business machines
Gannett Company, Incorporated	newspaper publishing
General Dynamics Corporation (General Dynamics-Electronics Division)	communication equipment
General Motors Corporation (Delco Appliance - Division)	electric motors
General Motors Corporation (Rochester Products Division)	motor vehicle parts
General Railway Signal Company	signaling equipment
Gleason Works	machine tools
Hart's Food Stores Incorporated	
Lehigh Valley Railroad Company	
Lincoln Rochester Trust Company	
McCurdy & Company	department store
Michaels, Stern & Company, Incorporated	men's and boys' apparel
New York Central System	
Pfaudler Permutit, Incorporated (Pfaudler Company Division)	food products and machinery
Rochester Gas and Electric Corporation	
Rochester Telephone Corporation	
Taylor Instrument Companies	thermometers and instruments
Xerox Corporation	photographic copying equipment

2.3 METEOROLOGY

2.3.1 REGIONAL CLIMATOLOGY

Atmospheric characteristics of the site region have been evaluated to provide a basis for regulated radioactive gas release limits (Section 2.3.4.1), accident analysis (Section 2.3.4.2), and storm protection (Section 2.3.2).

General climatic conditions at the site are influenced by its location in open rolling terrain on the lakeshore and by strong winter weather systems which move across the Great Lakes, usually from the northwest. Winters are rigorous with abundant snowfall (averaging about 75 inches of snow per year) and with a high percentage of cloud cover. Summers are moderately warm with an average of 2.5 to 3 inches of rainfall per month.

The site is well-ventilated. Calms (wind speeds less than approximately 1 mile/hr at about 50 ft above grade) occur about 1% of the time. Prevailing winds are from west-southwest (away from Rochester).

2.3.2 LOCAL METEOROLOGY

2.3.2.1 Meteorological Parameters

The climate in the site region, as typified by more than 30 years of records at Rochester airport, 20 miles west-southwest of the site, is shown in Figure 2.3-1. Average wind direction distribution measured at the site, at the Rochester airport, and at the Rochester Coast Guard station, 15 miles west of the site, is shown in Figure 2.3-2. Direction distribution during precipitation is also shown for the site and the airport in Figure 2.3-2. Average wind velocity distribution for these places is shown in Tables 2.3-1 through 2.3-6.

The normal wind speed to be used in the design and structural upgrade of Ginna Station safety-related structures, in conjunction with a normal ground snow load of 40 lb/ft^2, is 75 mph at 30 ft.

2.3.2.2 Severe Weather

The NRC evaluated severe weather phenomena for the Ginna site as part of the Systematic Evaluation Program (SEP) Topic II-2.A and concluded in *Reference 1* that the following phenomena applied to the Ginna site.

Through 1981, normal daily temperatures have ranged from a minimum of 18°F in January to a maximum of 82°F in July (*References 2 & 4*). Measured extreme temperatures for the site region are 100°F, which occurred in June 1953, and -16°F, which occurred in February 1961 (*Reference 5*). The extreme minimum and maximum temperatures appropriate to the Ginna site are 2°F (equaled or exceeded 99% of the time) and 91°F (equaled or exceeded 1% of the time) (*Reference 6*).

Mean annual snowfall in the site region is approximately 86 inches. In the site area, a maximum monthly snowfall occurred in February 1958 and totaled 72.6 inches (*Reference 7*). The maximum measured snow depth on the ground for the site region is 48 inches (*Reference 8*).

Highly localized effects operate to produce snowfalls in the Lake Ontario "snow belt" along the southern and eastern shores of the lake. A study of the area (*Reference 9*) has shown that snow loads for these sections of the lakeshore are about 40 to 50 lb/ft^2. If the 48-hr probable maximum winter precipitation (*Reference 8*) is added to the load, a total load of 100 lb/ft^2 results (*Reference 10*).

Thunderstorms occur an average of 29 days per year in the site area. Based on the annual number of thunderstorm days, the calculated annual flash density of ground lightning strikes is four flashes per km^2 (*Reference 11*). A structure with the approximate dimensions of the Ginna reactor building can expect, on the average, one strike every 10 years.

As a result of the SEP program (Topic III-7.B) (*Reference 12*), RG&E initiated the Ginna Structural Upgrade Program (Section 3.3.2) with acceptance criteria corresponding to event with a probability of 10^{-5} per reactor year. These criteria included the design tornado for the Ginna site with a wind velocity of 132 mph (*Reference 13*). The design criteria for steel structures are as follows:

A. No significant yielding at wind speeds up to 132 mph

B. No instability or collapse that might affect components or systems needed for safe shutdown at wind speeds up to about 200 mph.

2.3.3 ONSITE METEOROLOGICAL MEASUREMENTS PROGRAM

A 250-ft primary meteorological tower is located on the Ginna site. A backup tower is located at substation 13A, approximately 0.5 miles south of the Ginna site. Lightning protection is provided on the primary tower to protect the weather instrumentation.

The primary tower measures wind speed, wind direction, and temperatures (Dewpoint was removed in 1998 because it is not currently used in monitoring post-accident releases, see *Note 1*) as shown on Figure 2.3-3. The backup tower measures wind speed and wind direction as shown on Figure 2.3-4. Precipitation is measured on a separate pad near the primary tower.

The operational meteorological measurements program for Ginna consists of the primary 250-ft guyed tower located near the Lake Ontario shoreline approximately 850 ft northwest of the containment building. Listed below are the instrumentation and the heights of measurement on the tower.

Measured Parameter	Elevation Above Ground (ft)
Wind direction and speed	33, 150, 250
Dry bulb temperature	33, 150, 250
Vertical temperature gradient	33 to 150, 33 to 250
Dewpoint	33

Examination of the measurements system indicates that it conforms to the position stated in Regulatory Guide 1.23 (*Note 1*) for system accuracies, except for one of the wind direction and speed sensors at the 150 ft level and those at the 250 ft level. These wind sensors are not low-threshold instruments (i.e., starting speed of less than 1 mph) due to the short lifetime expected from the more sensitive sensors at a relatively windy site near a large lake, such as Ginna. The use of less sensitive, more sturdy wind instrumentation at the upper levels of the meteorological tower at Ginna is acceptable.

Strip-chart recorders for wind speed, wind direction, and temperature, measurements from the primary tower are located in an environmentally controlled equipment shelter located approximately 70 ft southwest of the tower. Precipitation, measured by means of a rainfall bucket mounted at about the 3-ft level on a separate concrete pad located approximately 30 ft northwest of the equipment shelter is also recorded on a strip chart in the shelter. A stripchart recorder for wind speed and wind direction measurements from the backup tower is located in an enclosure shed adjacent to the tower.

NOTE: The meteorological dewpoint monitor was originally installed to gather information used for making a determination if the property location had acceptable meteorological characteristics for siting a power reactor. The unit was part of a collection of instrumentation whose purpose was to provide data used to estimate the atmospheric diffusion of potential radionuclide releases (Regulatory Guide 1.23). Rochester Gas and Electric gathered and submitted the required meteorological data as part of the application for a full term operating license. NRC review of the onsite meteorological measurement program was planned as part of the Systematic Evaluation Program (SEP) topic II-2.B. This topic review was subsequently deleted based on the requirement to comply with the TMI action plan task II-F.3, "Instrumentation for Monitoring Accident Conditions" (NUREG-0660). (Regulatory Guide 1.97 contains the required meteorological instrumentation to quantify offsite exposures.) The information gathered by the dewpoint system is not required to satisfy our Regulatory Guide 1.97 commitments nor is it used as an input to other dose calculations.

A wind speed and direction recorder (33 ft) and three temperature displays (33, 150, and 250 ft) are located in the control room. Additional recording and display of meteorological data is provided by the plant process computer system (PPCS). Data from the backup tower can be reviewed in the technical support center (TSC) or the emergency operations facility (EOF) by means of modem connection. A minicomputer at the main tower can be accessed by telephone to get average and instantaneous values of wind speed, wind direction, temperature, and rainfall as well as the meander range for wind direction.

In 1981, RG&E committed to performing semi-annual primary and backup tower instrumentation calibrations (*Reference 20*). In 1992, the meteorological tower system was replaced with state of the art measurement equipment. In 1995, a review of 1994 instrumentation calibration data resulted in a determination that the as-found values were within tolerances and that no instrumentation adjustments were required. Based on the demonstrated reliability of the upgraded instrumentation, the calibration frequency was modified in 1996 to include annual instrumentation calibrations.

2.3.4 DIFFUSION ESTIMATES

2.3.4.1 Long-Term Diffusion Characteristics

The long-term diffusion characteristics for the Ginna site were reevaluated in June 1976 pursuant to the requirements of Appendix I to 10 CFR 50 (*References 14* and *15*). The atmospheric diffusion models used are those described in Regulatory Guide 1.111. The meteorological data used for the calculations were data from 1975. Wind roses for 4 years (1966, 1967, 1973-74, and 1975) were used to demonstrate that the 1975 data used in the analysis were consistent with longer term conditions at the site. The diffusion factors are given in Section 2.3.4.1.8.

2.3.4.1.1 Meteorological Data

Table 2.3-7 summarizes data bases available from the site monitoring program. The data periods used for the Regulatory Guide 1.111 calculations are indicated in the right hand column of the table. Data used for the analyses are presented as joint frequency tables. These tables were compiled for the 33 ft level for the 1975 period of record. Table 2.3-8 is a joint frequency table of wind speed, wind direction, and stability group for the 33-ft level using deltaT between 150 ft and 33 ft. These data are used for evaluation of all plant vent locations.
Joint frequency tables similar to Table 2.3-8 for the years 1966, 1967, and 1973-74 are shown in Table 2.3-9. *Figures 2.3-5 through 2.3-8* represent wind roses for each year of data collected up to 1975 from the lower level of the meteorological tower. The lower sensor array was moved from 50 ft to 33 ft in 1974.

Hourly data on meteorological conditions occurring during intermittent release periods have not been included since data for 1973-1975 show that release times were well distributed over the 24-hours period. Because of the intermittent release distribution with regard to time of day, annual average meteorology is considered applicable to such releases.

Inspection of the available records showed that the 1975 data were similar to longer term records previously collected at the site and therefore were appropriate for analyses at the site. For example, diffusion calculations using the wake-split model were made for the plant vent using 3-year composite joint frequency data for comparison with the calculation using 1975 joint frequency data. Results were not significantly different. Thus, from a diffusion standpoint the 1975 data are considered representative of longer term conditions. Another check of long-term representativeness was made by comparing wind roses from the four 1-year site data periods. Figures 2.3-5 through 2.3-8 were compared. They showed close similarity for most years, which further supports the conclusion that the 1975 data were representative of longer term conditions.

2.3.4.1.2 Airflow Trajectory and Terrain Influences

The general flow pattern in the Ginna site region, as indicated by the four wind roses, is from the northwest to the south. During the fall and winter, the eastern two-thirds of the U.S. and the northeastern U.S. in particular is dominated by high pressure centers generally passing to the south of the Ginna region. With their clockwise flow of air, these high pressure centers produce west or southwest winds when to the west of Ginna and south or southeast winds when to the east of Ginna. In the spring and summer there is a general west to east flow

across the U.S., which produces northwest to southwest winds in the Ginna site region depending on the position of the high pressure center. Also, mostly in the summer and scattered through the year, there are some Canadian high pressure centers that pass to the north of Ginna, producing clockwise circulation that accounts for most of the northerly and easterly winds in the area. Low pressure centers are rather frequent in the Ginna area particularly because of its close proximity to the St. Lawrence Valley cyclone storm track. However, these low pressure centers generally move rapidly and affect the area usually with east or northeast winds for only short periods of time.

During periods of light winds, local terrain features and the presence of the lake have some effect on flow patterns in the area. Balloon soundings were made at Ginna and at Oswego about 60 miles east of Ginna in support of a fossil plant application to the state of New York (*Reference 16*). Over 100 soundings were made at various times during a 1 year period. A lake effect circulation pattern was only identified in one of the soundings. Since winds are generally strong in the site region, it is expected that lake effect circulations will occur infrequently. Land breezes during periods when the lake is warmer than the land may also occur; however, these are not apparent from the sounding program results or from the meteorological tower records.

Since the terrain is gently rolling in the site region, it should not have a strong influence on wind patterns or cause flow channeling in any particular direction at the site. This is also confirmed by measurements at the meteorological tower. Since it is not considered practical at the present time to compute estimates using particle-in-cell or puff trajectory diffusion models, correction factors suggested in Regulatory Guide 1.111 for open terrain were used in this analysis. This is considered to result in estimates at distances near the plant which are very unlikely to be exceeded.

2.3.4.1.3 Atmospheric Diffusion Model

Average atmospheric dispersion evaluations were made using the straight line airflow model shown below:

$$\overline{(X/Q^{1})}_D = 2.032 \sum_{ij} n_{ij} [Nx\, \bar{u}_i\, \Sigma_{zj}(x)]^{-1} \exp[-h_e^2 / 2\sigma_{zj}^2(x)]$$

(Equation 2.3-1)

where:

h_e = the effective release height.

n_{ij} = the length of time (hours of valid data) weather conditions are observed to be at a given wind direction, windspeed class, i, and atmospheric stability class, j.

N = the total hours of valid data.

u_i = the geometrical mean of all speeds in the windspeed class, i, at a height representative of release, calms are one-half the threshold anemometer speed or less; extrapolation to higher levels, if necessary, is done by raising the ratio of the two heights to the n

power where n = 0.25, 0.33, and 0.5 for unstable, neutral, and stable conditions, respectively.

$_{z\,j}$ (x) = the vertical plume spread without volumetric correction at distance, x, for stability class, j (see Figure 1 of Regulatory Guide 1.111) based on vertical temperature difference (delta T) and Regulatory Guide 1.23 categorization of Pasquill Groups by delta T.

$\Sigma_{z\,j}$ (x) = the vertical plume spread with a volumetric correction for a release within the building wake cavity, at a distance, x, for stability class, j; otherwise $\Sigma_{z\,j}(x) = \sigma_{z\,j}(x)$.

$\overline{(X/Q')_D}$ = the average effluent concentration, X, normalized by source strength, Q′, at distance, x, in a given downwind direction, D.

2.032 = $(2/\pi)^{1/2}$ divided by the width in radians of a 22.5° sector.

In some cases hourly data were used and the summation over i and j in the above equation was deleted and the summation was accomplished for all hours at all distances for each direction. Dilution was decreased according to terrain correction factors in Figure 2 of Regulatory Guide 1.111. These factors were multiplied by the results from *Equation 2.3-1* and varied in accordance with the direction and distance being evaluated.

2.3.4.1.1 Source Configuration Considerations

2.3.4.1.1.1 Unobstructed Release Point

If a release point is elevated and there are no buildings which would obstruct the plume in its normal trajectory, *Equation 2.3-1* is used with the height of release defined as follows:

$$h_e = h_s + h_{pr} - h_t - c$$

where:

c = correction for low relative exit velocity.

h_e = effective release height.

h_{pr} = rise of the plume above the release point based on Briggs (see below).

h_s = physical height of the release point (the elevation of the stack base should be assumed to be zero).

h_t = maximum terrain height between the release point and the point for which the calculation is made.

Values of h_{pr} are computed follows for a "jet" since nuclear plant vents have an insignificant amount of buoyancy due to heated discharges

$$h_{pr} = 1.44 D \left(\frac{W_o}{u}\right)^{2/3} \left(\frac{X}{D}\right)^{1/3}$$

(Equation 2.3-2)

up to the point where h_{pr} is the minimum of the following two equations:

$$h_{pr\ max} = 3\left(\frac{W_o}{u}\right)D$$

(Equation 2.3-3)

or

$$h_{pr\ max} = 1.5\left(\frac{F_m}{u}\right)^{1/3} s^{-1/6}$$

(Equation 2.3-4)

where symbols are as before, and

D = stack or vent effective inside diameter (m)

W_o = stack or vent exit velocity (m/sec)

u = wind speed at discharge level (m/sec)

F_m = momentum flux (m^4/sec^2)

s = stability parameter (sec^{-2})

2.3.4.1.1.2 Obstructed Release Point

If the plume trajectory from a release point (vent) does not remain outside of building wake influences near large structures, all or portions of the plume are considered to be entrapped and brought to ground level in the turbulent wake of the building. The criteria for determining the portion of the plume treated as an elevated or ground release follow from Equations 6, 7, and 8 of Regulatory Guide 1.111 and are repeated here for completeness.

If $W_o/\bar{u} > 5.0$ use h_e as calculated above

If $W_o/\bar{u} \leq 1.0$ use $h_e = 0$

$$\text{If } 1 < \frac{W_o}{\bar{u}} \leq 1.5 \qquad E_t = 2.58 - 1.58\left(\frac{W_o}{\bar{u}}\right)$$

(Equation 2.3-5)

$$\text{If } 1.5 < \frac{W_o}{\bar{u}} \leq 5.0 \qquad E_t = 0.3 - 0.06\left(\frac{W_o}{\bar{u}}\right)$$

(Equation 2.3-6)

The appropriate diffusion estimate is then computed by assuming an elevated release 100 (1 - E_t) percent of the time and by assuming ground release 100 E_t percent of the time. Calculations utilizing this mixed model are referred to as wake-split calculations.

A building wake correction is computed for all ground releases near structures in accordance with the following general equation:

$$\Sigma = \sqrt{\sigma_z^{\,2} + cH^2 / \pi} \leq 1.73\sigma_z$$

(Equation 2.3-7)

where:	$\Sigma =$	effective dispersion coefficient for use in *Equation 2.3-1* (m_p)
	$c =$	building wake coefficient (c = 0.5).
	$H =$	height of the tallest structure in the nuclear plant power block (m)

2.3.4.1.5 Removal Mechanisms

As radioactive effluent in a plume travels downwind, it is subject to several removal mechanisms including radioactive decay, dry deposition, and wet deposition (during rain). Corrections for radioactive decay are not made in the estimates reported in this section.

Dry deposition which results in depletion of halogen and particulate isotopes from the plume is considered only to the extent suggested in Regulatory Guide 1.111. Depletion factors in these curves are a function of height and distance; therefore, for sites where elevated releases occur the terrain must be subtracted from the plume height before entering the curves at the appropriate distance. Each elevated or ground level X/Q is multiplied by the depletion and the terrain correction factors before combining to give the final depleted X/Q value.

To determine relative deposition rate as a function of distance and stability the curves given in Figures 7 through 10 of Regulatory Guide 1.111 are used. Again, terrain heights are subtracted before the table look-up is made. Terrain correction factors, if any, multiply each D/Q value. Values from the curves are divided by the sector cross width (arc) at the point of calculation.

Dry deposition is believed to adequately represent overall deposition rates, since seasonal rainfall is fairly uniform; therefore, wet deposition has not been considered.

2.3.4.1.6 Summary of Plant Discharges

A summary of plant vent information for each discharge point is given in Tables 2.3-10 and 2.3-11. Only vents used during routine operation are considered in this evaluation.

2.3.4.1.7 Input Assumptions

Table 2.3-12 tabulates all pertinent input information utilized in making the model calculations. Table 2.3-13 gives terrain elevations for all distances out to 10 miles. Terrain height is

conservatively not allowed to decrease with increasing distance or to decrease below plant grade in accordance with Regulatory Guide 1.111.

2.3.4.1.8 Results

Resulting X/Q, depleted X/Q, and D/Q values are listed in *Reference 14*, (pages 89-94) for each direction sector for ten distances. These results are used as input for the dose calculations described in Section 11.3.

Tables 2.3-14 through 2.3-19 are reproduced from *Reference 14* (pages 96-102). These tables summarize the resulting diffusion factors for each of the receptor locations. Each table represents model results for one vent location for each season being evaluated. One set of calculations was made for the plant vents located on the intermediate building roof through which most effluents are discharged and a second set of calculations was made for vents which are assumed to release into the building wake in all wind conditions.

2.3.4.2 Accident Analysis Diffusion Characteristics

The atmospheric transport and diffusion characteristics for accident analysis at the Ginna site were reevaluated during the review of Systematic Evaluation Program (SEP) Topic II-2.C. Specifically, the NRC calculated the X/Q values for the Ginna site, with the results appearing in *Reference 17*. The results of the RG&E evaluation appear in *Reference 18*. The two evaluations are discussed below, but have been superseded and are included here for historical purposes.

As part of the Control Room Emergency Air Treatment System (CREATS) Modification, new accident dose analyses were required for the control room because of the characteristics of the new system. For consistency, it was decided to calculate new dose values for the Exclusion Area Boundary (EAB) and Low Population Zone (LPZ), and to update the analysis using the alternate source term per *Reference 21*. New atmospheric dispersion coefficients (X/Q) were calculated as part of this effort and are detailed in *Reference 22*. The NRC approved these new X/Q values and dose analyses in *Reference 23*, as supplemented by *Reference 24*.

2.3.4.2.1 Nuclear Regulatory Commission Evaluation (Historical)

The atmospheric dispersion factors were calculated using the direction dependent method described in Regulatory Guide 1.145. The model considers the directionally dependent atmospheric dispersion conditions. Specifically, the model considers the following effects:

A. Lateral plume meander, as a function of atmospheric stability, wind speed, and distance from the source, during periods of low wind speeds (less than 6 m/sec) and neutral and stable atmospheric conditions.
B. Exclusion area boundary distance as a function of direction from the plant.
C. Atmospheric dispersion conditions when the wind is blowing in a specific direction.
D. The fraction of time that the wind can be expected to blow into each of the 16 compass directions.

For the purpose of this evaluation available onsite meteorological data for the periods 1966-1967 and 1973-1974 were used (see Table 2.3-9). For the composite data set, wind speed and wind direction were measured at the 50-ft level with the wind speeds reduced by means of a power law to represent conditions at the 33-ft level. Atmospheric stability was defined by the vertical temperature gradient measured between the 33-ft and 150-ft levels. The maximum X/Q value was calculated for the southeast direction, 503 m from the plant.

Using the composite of onsite meteorological data, the following X/Q values for an assumed ground-level release with a building wake factor, cA, of 440 m^2 were determined at distances corresponding to the exclusion area boundary (EAB) and the outer boundary of the low-population zone (LPZ) in an onshore direction.

Time Period	Distance	X/Q (sec/m 3)
0 to 2 hours	EAB (503 m SE)	4.8×10^{-4}
0 to 8 hours	LPZ (4827 m)	3.0×10^{-5}
8 to 24 hours	LPZ (4827 m)	2.1×10^{-5}
1 to 4 days	LPZ (4827 m)	8.6×10^{-6}
4 to 30 days	LPZ (4827 m)	2.5×10^{-6}

2.3.4.2.1 Rochester Gas and Electric Corporation Evaluation (Historical)

The atmospheric dispersion factors were calculated using the direction dependent method described in Regulatory Guide 1.145 (0.5% probability). Considerations used in the analysis include the following:

A. Lateral plume meander, as a function of atmospheric stability, wind speed and distance from the source, during periods of low wind speeds (less than 6 m/sec) and neutral and stable atmospheric conditions.

B. Atmospheric dispersion conditions when the wind is blowing in each specific onshore direction using hourly meteorological data in the WINDOW program (*Reference 19*).

C. The release was assumed to be at ground level.

D. A building wake factor has been applied (cA = 440 m^2).

E. Three years of meteorological data (1966, 1967, and 1973-1974) were used (see Table 2.3-9).

F. The exclusion area boundary in each of 16 directions is as shown in Table 2.3-20. The distance to the low-population zone is 4827 m.

Time Period	Location	X/Q (sec/m 3)
0 to 2 hours	EAB	2.2×10^{-4}
0 to 8 hours	LPZ (4827 m)	2.3×10^{-5}
8 to 24 hours	LPZ (4827 m)	7.0×10^{-6}
1 to 4 days	LPZ (4827 m)	2.7×10^{-6}
4 to 30 days	LPZ (4827 m)	1.1×10^{-6}

2.3.4.2.2 Current Approved Evaluation

The current approved evaluation was performed using the KRPavan computer code, and is detailed in *Reference 22*. Assumptions and results are below:

- Meteorological data for the years 1999 through 2003 was used in this analysis. Unlike ARCON96 (used in the control room χ/Q determinations), KRPavan does not consider missing or invalid data. As such, missing and invalid hours are deleted from the SQRT files.
- There are a total of 43,824 available hours. Of these, 556 are missing (not recorded) and 835 hours are determined to be invalid. The net hours of available data is 42,433. A sample KRPavan output file shows that only 42,430 hours of data were read, i.e., 3 hours were omitted from the joint frequency distribution. No effort was made to recover the 3 hours of missing data. The data recovery fraction is:

  42,430/43,824 = 0.968, or about 97 percent.

This exceeds the 90% minimum data recovery suggested in Regulatory Guide 1.23.

- Activity releases are assumed to be at ground level
- The height of the lower and upper level wind speed measurement instruments are 10 meters (33 ft) and 45.7 meters (150 ft), respectively. The upper level height is provided for information.
- Calm hours are distributed in the first wind speed category of the joint frequency distribution.
- The assumed building wake area is normal to a line drawn from the source to the receptor. Incident air flow, striking a simple block building, may move upward or sideways, depending on the position of the edges of the roof or sides. Clusters of buildings, like the Ginna site, that have a greater vertical area (and more roofs and edges) than a single structure, e.g. the facade, are effective wake producers.
- The vertical cross-section area, conservatively assumed for the building-wake correction, is $1850m^2$. This is the area of the Containment Building Facade assumed for containment leakage. Other values are also used ($1071m^2$ and $1800m^2$) to investigate sensitivity to changing wake area.
- The direction dependant Exclusion Area Boundary (EAB) χ/Q values for licensing basis case are shown on Table 2.3-21. The direction dependant Low Population Zone (LPZ) χ/Q values for the licensing basis case are shown on Table 2.3-22.
- Figure 2.3-9 shows the plant layout, including activity release points and elevations of the major structural high-points. All activity releases are not assumed into the containment wake, rather, all releases are assumed into the wake produced by the overall facility. A conservatively small wake area is assumed.
- Fourteen wind speed categories are assumed.
- Wind speed is input in meters/sec.

The off-site χ/Q values are summarized below:

Boundary	0-2 hours	0-8 hours	8-24 hours	24-96 hours	96-720 hours
EAB	2.17E-4	-	-	-	-
LPZ	4.97E-5	2.51E-5	1.78E-5	8.5E-6	2.93E-5

2.3.4.2.1 Conclusions

The atmospheric dispersion values described in Section 2.3.4.2.3 are the values that Ginna LLC will use in the future in estimating offsite radiological exposures from hypothetical accidents.

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Table 2.3-1

WIND VELOCITY SUMMARY GINNA SITE TOWER, 50 FT. TOWER (FEBRUARY 1965 - JANUARY 1967, INCLUSIVE)

wind speed	calm	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Tot	Avg	%
0	157	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	157	0	0.97
1-3	0	82	86	52	83	119	74	75	74	89	124	156	115	74	55	85	123	1466	2	9.18
4-7	0	128	100	87	154	266	191	186	229	386	611	787	537	265	244	278	161	4610	5	28.8
8-12	0	134	83	102	154	259	140	123	276	482	421	602	672	565	559	299	192	5063	9	31.6
13-18	0	68	63	97	105	91	57	40	159	287	75	129	355	598	469	283	145	3021	14	18.8
19-25	0	8	23	44	46	34	6	5	41	70	4	5	100	311	251	212	58	1218	21	7.6
26-32	0	0	13	12	10	5	0	0	5	10	0	1	23	76	94	118	21	388	28	2.5
33-40	0	0	3	2	0	0	0	0	0	0	0	0	1	18	19	36	0	79	34	0.5
40+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	3	0	7	48	0.05
TOT	157	420	371	396	552	774	468	429	784	1324	1235	1680	1803	1908	1694	1314	700	16009	10	---
AVG	0	8	9	11	9	8	7	7	9	9	7	7	9	13	13	14	10	10	---	---
%	0.97	2.65	2.35	2.50	3.50	4.86	2.94	2.69	4.92	8.32	7.74	10.3	11.2	11.8	10.6	8.22	4.39	---	---	---

Table 2.3-2

WIND VELOCITY SUMMARY GINNA SITE TOWER, 150 FT. TOWER (FEBRUARY 1965 - JANUARY 1967, INCLUSIVE)

wind speed	calm	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Tot	Avg	%
0	242	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	242	0	1.48
1-3	0	83	57	66	66	90	47	57	52	39	45	61	47	42	57	87	89	985	2	6.0
4-7	0	120	92	118	191	228	147	187	149	156	200	248	225	219	246	187	138	2851	5	17.5
8-12	0	88	67	106	183	234	242	231	335	465	391	651	741	611	516	267	122	5250	10	32.2
13-18	0	126	72	96	98	124	58	80	322	408	329	415	526	613	569	273	181	4290	15	26.3
19-25	0	65	31	57	45	35	6	18	143	169	31	68	206	342	334	214	158	1922	21	11.8
26-32	0	3	10	12	16	4	0	1	29	22	0	4	58	122	116	126	60	583	28	3.5
33-40	0	0	4	9	2	0	0	0	1	0	0	1	8	37	29	65	17	173	35	1.0
40+	0	0	1	0	0	0	1	1	0	0	1	0	1	7	6	27	7	52	50	0.3
TOT	242	485	334	464	601	715	501	575	1031	1259	997	1448	1812	1993	1873	1246	772	16348	12	---
AVG	0	10	10	11	9	9	8	8	12	12	10	10	12	14	14	16	14	12	---	---
%	1.48	2.96	2.04	2.84	3.65	4.35	3.0	3.5	6.3	7.7	6.1	8.8	11.3	12.2	11.4	7.65	4.7	---	---	---

Table 2.3-3

WIND VELOCITY SUMMARY GINNA SITE TOWER, 250 FT. TOWER (FEBRUARY 1965 - JANUARY 1967, INCLUSIVE)

wind speed	calm	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Tot	Avg	%
0	129	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	129	0	0.78
1-3	0	56	54	55	51	70	34	26	35	25	22	50	44	36	54	70	64	746	2	4.52
4-7	0	98	94	125	155	152	100	88	85	87	101	131	115	117	163	150	113	1874	5	11.3
8-12	0	86	74	127	182	208	171	152	175	220	202	311	391	380	415	283	120	3497	10	21.1
13-18	0	100	87	110	120	156	142	178	290	404	314	542	759	817	615	316	174	5124	15	31.1
19-25	0	96	59	67	65	70	31	49	249	346	237	338	401	610	482	342	161	3603	21	21.7
26-32	0	22	12	18	13	20	2	3	73	102	14	30	89	213	230	211	69	1121	28	6.7
33-40	0	1	11	8	1	5	3	1	14	19	0	0	14	81	90	107	31	386	35	2.3
40+	0	0	0	0	0	0	0	0	1	0	0	0	3	36	16	39	3	98	45	0.6
TOT	129	459	391	510	587	681	483	497	922	1203	890	1402	1816	2290	2065	1518	735	16578	15	---
AVG	0	12	12	11	10	11	10	11	16	16	14	14	15	17	17	18	15	15	---	---
%	0.78	2.75	2.32	3.05	3.5	4.1	2.9	3.0	5.55	7.30	5.35	8.5	11.0	13.8	12.5	9.2	4.4	---	---	---

Table 2.3-4

WIND VELOCITY SUMMARY (HOURS) ROCHESTER AIRPORT FIVE YEARS

wind-speed MPH	Calm	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%
calm	652	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	652	1.50
1-3	0	70	56	74	136	194	262	346	264	403	297	357	165	102	97	66	66	2955	6.7
4-7	0	216	156	185	503	624	653	716	606	1006	1248	1158	783	342	347	327	245	9115	20.8
8-14	0	548	627	662	793	862	632	456	771	1732	2879	2108	2813	1188	1363	938	777	19149	43.8
15-39	0	221	303	342	104	101	108	78	249	485	922	1081	4392	1233	1516	517	268	11920	27.2
40-49	0	0	2	0	0	0	0	0	0	0	0	2	22	0	0	0	0	26	nil
50+	0	0	0	0	0	0	0	0	0	0	0	0	5	0	0	0	0	5	nil
TOT	652	1055	1144	1263	1536	1781	1655	1596	1890	3626	5346	4706	8180	2865	3323	1848	1356	43822	---
%	1.5	2.4	2.6	2.9	3.5	4.1	3.8	3.6	4.3	8.3	12.2	10.8	18.7	6.5	7.6	4.2	3.1	---	---

Table 2.3-5

WIND VELOCITY SUMMARY (HOURS) DURING PRECIPITATION ROCHESTER AIRPORT

wind-speed MPH	Calm	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%
calm	60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	60	0.71
1-3	---	6	4	17	23	26	29	35	21	27	30	32	17	17	16	8	13	321	3.80
4-7	---	28	20	37	101	84	108	102	70	91	84	113	112	63	64	59	30	1166	13.8
8-14	---	126	126	151	261	219	166	97	148	169	268	228	552	235	299	230	192	3467	41.1
15-39	---	116	140	164	64	41	49	29	50	74	121	193	1111	393	537	178	150	3410	40.4
40-49	---	0	2	0	0	0	0	0	0	0	0	0	5	0	0	0	0	7	---
50+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	---
TOT	---	276	292	369	449	370	352	263	289	361	503	566	1797	708	916	475	385	8431	---
%	60	3.28	3.48	4.38	5.33	4.4	4.18	3.1	3.44	4.28	6.0	6.7	21.2	8.41	10.8	5.64	4.57	---	---

Table 2.3-6
WIND VELOCITY SUMMARY (HOURS) ROCHESTER COAST GUARD STATION (1951 - 1955)

Wind-Speed MPH	N	NE	E	SE	S	SW	W	NW	CALM	TOTAL	%
calm	0	0	0	0	0	0	0	0	53	53	0.9
1-3	79	92	135	122	125	299	157	75	---	1084	18.3
4-7	111	123	149	176	171	572	382	136	---	1820	30.6
8-14	93	161	183	135	104	515	470	244	---	1905	32.0
15-39	64	123	101	43	33	223	319	179	---	1085	18.2
40-49	1	3	0	0	0	0	0	0	---	4	---
50+	0	0	0	0	0	0	0	0	---	0	0
TOTAL	348	502	568	476	433	1609	1328	634	---	5951	---
%	5.9	8.4	9.5	8.0	7.3	27.0	22.3	10.7	0.9	---	---

Table 2.3-7
SUMMARY OF METEOROLOGICAL DATA GINNA SITE

Period of Record	Speed and Direction Level (ft.)	Temperature Difference Between (ft.)	Combined Percent Recovery	Comment
12/65 - 12/66	50	150 - 10	91.0	Used for 3-year composite for
1/67 - 12/67	50	150 - 10	95.0	Used for 3-year composite for
1/68 - 4/73	50	150 - 10	Not determined	Not used for analysis
5/13/73 - 5/13/74	50	150 - 10	83.3	Used for 3-year composite for
1/75 - 12/75	33	150 - 33	84.1	Used for diffusion calculations
Composite of 1/66 - 12/66 1/67 - 12/67 5/13/73 - 5/13/74	50	150 - 33	92.3	Used for comparison with 1975 data

Table 2.3-8a

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference ≤ -1.0 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	1	2	0	0	0	0	0	10	0.7	1.79
2.1 - 3.5	5	4	7	2	1	0	0	1	45	3.1	2.87
3.6 - 7.5	19	14	32	6	12	5	9	16	269	18.7	5.62
7.6 - 12.5	17	23	46	22	13	11	22	20	405	28.1	9.79
12.6 - 18.5	26	14	18	10	4	2	12	6	350	24.3	15.07
18.6 - 24.5	3	9	34	10	6	1	1	10	249	17.3	21.18
24.6+	0	0	1	9	7	0	0	3	113	7.8	30.06
TOTAL	70	65	140	59	43	19	44	56	1441	100.0	9.65
PERCENT	4.9	4.5	9.7	4.1	3.0	1.3	3.1	3.9	100.0	---	---
AVG SPEED	10.8	11.2	12.2	14.8	13.9	9.3	10.5	12.3	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 13.8											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5											

Table 2.3-8b

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference ≤ -1.0 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	0	0	1	1	2	2	1	10	0.7	1.79
2.1 - 3.5	2	1	0	2	1	4	8	7	45	3.1	2.87
3.6 - 7.5	14	12	10	7	22	46	25	20	269	18.7	5.62
7.6 - 12.5	18	23	14	23	65	23	34	31	405	28.1	9.79
12.6 - 18.5	13	18	18	27	54	37	36	55	350	24.3	15.07
18.6 - 24.5	4	3	10	18	22	38	70	10	249	17.3	21.18
24.6+	2	0	0	3	9	25	52	2	113	7.8	30.06
TOTAL	53	57	52	81	174	175	227	126	1441	100.0	9.65
PERCENT	3.7	4.0	3.6	5.6	12.1	12.1	15.8	8.7	100.0	---	---
AVG SPEED	11.3	11.0	13.1	14.4	13.6	15.5	18.7	12.1	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 13.8											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5											

Table 2.3-8c

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -1.0 BUT ≤ -0.9 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	0	0	0	1	0	0	0	1	0.2	1.70
2.1 - 3.5	2	0	3	0	1	0	1	2	19	4.5	2.86
3.6 - 7.5	3	5	8	2	2	4	5	3	81	19.3	5.79
7.6 - 12.5	5	5	12	6	4	5	18	17	143	34.0	9.70
12.6 - 18.5	7	5	5	3	4	2	4	5	102	24.3	14.44
18.6 - 24.5	1	2	3	2	0	0	0	5	49	11.7	21.84
24.6+	0	0	0	1	1	0	0	0	25	6.0	30.59
TOTAL	18	17	31	14	13	11	28	32	420	100.0	9.10
PERCENT	4.3	4.0	7.4	3.3	3.1	2.6	6.7	7.6	100.0	---	---
AVG SPEED	11.3	11.1	9.8	13.6	11.5	9.1	9.3	11.2	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 12.6											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-8d

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -1.0 BUT ≤ -0.9 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	1	0.2	1.70
2.1 - 3.5	0	0	0	0	1	2	4	2	19	4.5	2.86
3.6 - 7.5	12	12	9	3	3	4	4	2	81	19.3	5.79
7.6 - 12.5	11	13	10	12	16	3	5	1	143	34.0	9.70
12.6 - 18.5	10	4	6	24	12	6	2	3	102	24.3	14.44
18.6 - 24.5	0	2	4	4	5	11	10	0	49	11.7	21.84
24.6+	1	0	1	3	4	8	6	0	25	6.0	30.59
TOTAL	35	31	30	46	41	34	31	8	420	100.0	9.10
PERCENT	8.3	7.4	7.1	11.0	9.8	8.1	7.4	1.9	100.0	---	---
AVG SPEED	9.9	9.6	11.8	14.5	14.3	19.1	17.7	9.3	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 12.6											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-8e

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.9 but ≤ -0.8 (°F/100 FT)										
					WIND DIRECTION					
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	0	0	0	0	0	1	0	4	0.7	1.46
2.1 - 3.5	0	5	4	3	1	1	2	0	35	5.8	2.89
3.6 - 7.5	6	7	8	6	16	3	6	13	139	23.2	5.61
7.6 - 12.5	3	5	4	10	9	8	25	34	206	34.3	9.71
12.6 - 18.5	5	2	4	4	4	1	9	16	148	24.7	15.01
18.6 - 24.5	0	1	2	3	0	0	1	2	47	7.8	20.57
24.6+	0	0	0	0	2	0	0	2	21	3.5	28.69
TOTAL	14	20	22	26	32	13	44	67	600	100.0	8.14
PERCENT	2.3	3.3	3.7	4.3	5.3	2.2	7.3	11.2	100.0	---	---
AVG SPEED	10.1	8.2	9.0	10.1	9.5	8.5	10.1	11.3	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 11.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-8f

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.9 but ≤ -0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	1	0	1	1	0	0	0	4	0.7	1.46
2.1 - 3.5	2	4	0	1	2	3	3	4	35	5.8	2.89
3.6 - 7.5	17	14	10	7	10	9	4	3	139	23.2	5.61
7.6 - 12.5	15	15	17	32	19	7	0	3	206	34.3	9.71
12.6 - 18.5	8	2	11	33	33	8	6	2	148	24.7	15.01
18.6 - 24.5	1	1	1	9	13	6	7	0	47	7.8	20.57
24.6+	1	0	0	5	4	4	3	0	21	3.5	28.69
TOTAL	44	37	39	88	82	37	23	12	600	100.0	8.14
PERCENT	7.3	6.2	6.5	14.7	13.7	6.2	3.8	2.0	100.0	---	---
AVG SPEED	9.5	7.4	11.1	13.6	13.7	13.5	16.8	7.0	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 11.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-8g

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.8 but ≤ -0.3 (°F /100 FT)										
WIND DIRECTION										
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	1	0	1	0	0	1	6	0.2	0.30
CALM+ - 2.0	8	2	4	3	1	3	3	3	45	1.5	1.15
2.1 - 3.5	14	10	15	6	7	6	11	10	155	5.2	2.83
3.6 - 7.5	27	17	24	33	54	34	46	53	714	24.2	5.58
7.6 - 12.5	28	12	22	28	60	43	68	120	1011	34.2	9.71
12.6 - 18.5	20	7	9	34	19	6	22	69	669	22.6	14.94
18.6 - 24.5	17	3	10	12	4	0	3	24	235	7.9	21.00
24.6+	2	2	1	4	5	0	0	16	121	4.1	28.30
TOTAL	116	53	86	120	152	92	153	304	2956	100.0	7.28
PERCENT	3.9	1.8	2.9	4.1	5.1	3.1	5.2	10.3	100.0	---	---
AVG SPEED	10.2	8.8	8.9	11.3	9.8	8.0	9.1	12.0	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 11.3										
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 16										

Table 2.3-8h

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.8 but ≤ -0.3 (°F /100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO-MEAN SPD (MPH)
CALM	1	1	0	0	0	0	1	0	6	0.2	0.30
CALM+ - 2.0	2	3	2	4	4	0	2	1	45	1.5	1.15
2.1 - 3.5	19	17	8	8	7	6	3	8	155	5.2	2.83
3.6 - 7.5	113	117	68	36	28	21	24	19	714	24.2	5.58
7.6 - 12.5	115	102	109	132	72	22	42	28	1011	34.2	9.71
12.6 - 18.5	40	25	44	134	99	62	54	25	669	22.6	14.94
18.6 - 24.5	13	4	5	29	59	27	23	2	235	7.9	21.00
24.6+	6	0	5	27	21	18	13	0	121	4.1	28.30
TOTAL	309	269	241	370	290	156	162	83	2956	100.0	7.28
PERCENT	10.5	9.1	8.2	12.5	9.8	5.3	5.5	2.8	100.0	---	---
AVG SPEED	9.3	8.0	9.9	13.4	14.9	15.3	14.1	10.1	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 11.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 16											

Table 2.3-8i

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.3 but ≤ -0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO-MEAN SPD (MPH)
CALM	0	0	3	0	0	0	0	0	3	0.2	0.30
CALM+ - 2.0	0	2	3	2	2	2	7	5	37	2.7	1.25
2.1 - 3.5	6	7	10	8	2	3	6	11	126	9.0	2.13
3.6 - 7.5	3	3	12	12	28	20	25	51	541	38.8	5.36
7.6 - 12.5	2	3	2	10	14	22	26	90	429	30.8	9.55
12.6 - 18.5	1	2	0	4	2	3	4	38	210	15.1	14.72
18.6 - 24.5	0	0	0	0	1	0	3	10	32	2.3	21.53
24.6+	0	0	0	0	0	0	0	1	15	1.1	27.61
TOTAL	12	17	30	36	49	50	71	206	1393	100.0	5.66
PERCENT	0.9	1.2	2.2	2.6	3.5	3.6	5.1	14.8	100.0	---	---
AVG SPEED	5.2	5.7	3.9	7.1	6.9	7.6	7.4	9.8	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 8.5											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5											

Table 2.3-8j

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.3 but ≤ -0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	3	0.2	0.30
CALM+ - 2.0	3	1	5	1	1	0	2	1	37	2.7	1.25
2.1 - 3.5	18	31	9	1	4	4	4	2	126	9.0	2.13
3.6 - 7.5	140	153	57	15	5	8	5	4	541	38.8	5.36
7.6 - 12.5	67	41	56	36	25	18	12	5	429	30.8	9.55
12.6 - 18.5	36	19	36	31	16	9	5	4	210	15.1	14.72
18.6 - 24.5	4	2	1	2	4	3	2	0	32	2.3	21.53
24.6+	2	0	1	5	4	1	1	0	15	1.1	27.61
TOTAL	270	247	165	91	59	43	31	16	1393	100.0	5.66
PERCENT	19.4	17.7	11.8	6.5	4.2	3.1	2.2	1.1	100.0	---	---
AVG SPEED	8.0	6.6	9.0	12.5	12.2	10.8	10.2	8.7	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 8.5											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5											

Table 2.3-8k

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.8 but ≤ 2.2 (°F/100 FT)											
	WIND DIRECTION										
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	1	0	1	1	1	1	1	9	2.8	1.52
2.1 - 3.5	1	5	2	1	4	1	3	7	39	12.1	2.94
3.6 - 7.5	2	1	5	7	3	8	5	20	215	66.8	5.27
7.6 - 12.5	1	0	1	5	2	1	0	2	52	16.1	9.84
12.6 - 18.5	0	0	2	1	2	2	0	0	7	2.2	14.54
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	7	10	15	12	13	9	30	322	0.0	4.86
PERCENT	1.2	2.2	3.1	4.7	3.7	4.0	2.8	9.3	100.0	---	---
AVG SPEED	5.1	2.8	6.9	7.1	6.7	6.3	4.3	5.0	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 5.9											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-8l

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > -0.8 but ≤ 2.2 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	1	0	0	0	0	1	0	1	9	2.8	1.52
2.1 - 3.5	9	3	0	0	0	0	1	2	39	12.1	2.94
3.6 - 7.5	81	55	11	9	0	2	2	4	215	66.8	5.27
7.6 - 12.5	9	4	4	8	6	0	6	3	52	16.1	9.84
12.6 - 18.5	0	0	0	0	0	0	0	0	7	2.2	14.54
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	100	62	15	17	6	3	9	10	322	0.0	4.86
PERCENT	31.1	19.3	4.7	5.3	1.9	0.9	2.8	3.1	100.0	---	---
AVG SPEED	5.4	5.4	6.9	7.5	8.7	4.4	8.6	6.5	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 5.9											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-8m

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > 2.2 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	1	0	0	0	0	0	0	3	2.6	1.89
2.1 - 3.5	1	1	1	1	4	0	0	0	14	12.1	2.84
3.6 - 7.5	3	3	0	3	6	8	4	1	69	59.5	5.46
7.6 - 12.5	0	0	0	1	3	3	2	0	29	25.0	8.85
12.6 - 18.5	0	0	0	1	0	0	0	0	1	0.9	13.50
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	5	1	6	13	11	6	1	116	0.0	5.16
PERCENT	7.4	4.3	0.9	5.2	11.2	9.5	5.2	0.9	100.0	---	---
AVG SPEED	4.9	3.9	2.2	6.9	5.6	7.0	6.9	6.8	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 6.2											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-8n

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 33-FT LEVEL FOR 1975 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 33 FT)

Temperature Difference > 2.2 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 2.0	0	2	0	0	0	0	0	0	3	2.6	1.89
2.1 - 3.5	1	0	3	0	1	1	0	0	14	12.1	2.84
3.6 - 7.5	8	3	11	6	4	3	0	6	69	59.5	5.46
7.6 - 12.5	0	7	6	4	0	0	1	2	29	25.0	8.85
12.6 - 18.5	0	0	0	0	0	0	0	0	1	0.9	13.50
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	9	12	20	10	5	4	1	8	116	0.0	5.16
PERCENT	7.8	10.3	17.2	8.6	4.3	3.4	0.9	6.9	100.0	---	---
AVG SPEED	4.7	7.0	6.4	7.0	4.6	4.6	12.2	6.9	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 6.2											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-9a

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference ≤ -1.0 (°F/100 FT)										
WIND DIRECTION										
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	2	1	1	2	0	0	0	0	11	0.4	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	44	36	23	35	18	4	1	2	263	9.1	2.73
3.6 - 7.5	68	89	86	112	67	7	21	29	1012	35.1	5.20
7.6 - 12.5	13	16	57	106	60	6	11	41	887	30.8	9.46
12.6 - 18.5	2	13	40	55	26	0	1	5	501	17.4	14.82
18.6 - 24.5	0	3	38	44	9	0	0	1	192	6.7	20.31
24.6+	1	0	4	2	0	0	0	0	17	0.6	25.54
TOTAL	130	158	249	356	180	17	34	78	2883	100.0	6.56
PERCENT	4.5	5.5	8.6	12.3	6.2	0.6	1.2	2.7	100.0	---	---
AVG SPEED	4.8	6.1	10.2	9.9	8.6	5.8	7.1	8.5	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 9.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-9b

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference ≤ -1.0 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	1	0	1	0	0	0	1	2	11	0.4	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	13	6	12	2	4	5	22	36	263	9.1	2.73
3.6 - 7.5	32	53	58	25	18	80	178	89	1012	35.1	5.20
7.6 - 12.5	42	24	45	49	35	199	155	28	887	30.8	9.46
12.6 - 18.5	35	1	5	18	28	108	153	11	501	17.4	14.82
18.6 - 24.5	8	1	0	4	2	31	43	8	192	6.7	20.31
24.6+	0	0	0	0	1	7	1	1	17	0.6	25.54
TOTAL	131	85	121	98	88	430	553	175	2883	100.0	6.56
PERCENT	4.5	2.9	4.2	3.4	3.1	14.9	19.2	6.1	100.0	---	---
AVG SPEED	9.9	6.7	6.9	9.9	10.6	11.4	10.5	6.7	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 9.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2											

Table 2.3-9c

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -1.0 but ≤ -0.9 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	1	1	1	0	0	1	0	0	7	1.2	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	6	9	5	3	3	2	2	2	53	8.9	2.57
3.6 - 7.5	11	9	6	17	15	3	8	10	175	29.4	5.11
7.6 - 12.5	5	2	13	15	18	4	1	5	186	31.2	9.66
12.6 - 18.5	0	6	18	18	6	0	0	2	117	19.6	14.68
18.6 - 24.5	0	4	18	6	4	0	0	1	53	8.9	20.65
24.6+	0	0	0	4	0	0	0	0	5	0.8	27.22
TOTAL	23	31	61	63	46	10	11	20	596	100.0	6.49
PERCENT	3.9	5.2	10.2	10.6	7.7	1.7	1.8	3.4	100.0	---	---
AVG SPEED	5.4	8.1	13.4	11.9	9.6	5.9	4.8	8.0	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 9.9											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-9d

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -1.0 but ≤ -0.9 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	1	0	0	0	0	0	1	1	7	1.2	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	3	2	1	1	0	1	6	7	53	8.9	2.57
3.6 - 7.5	7	11	14	13	8	16	14	13	175	29.4	5.11
7.6 - 12.5	8	4	13	12	19	45	13	9	186	31.2	9.66
12.6 - 18.5	3	0	3	3	15	19	21	3	117	19.6	14.68
18.6 - 24.5	0	0	0	1	5	3	11	0	53	8.9	20.65
24.6+	0	0	0	0	1	0	0	0	5	0.8	27.22
TOTAL	22	17	31	30	48	84	66	33	596	100.0	6.49
PERCENT	3.7	2.9	5.2	5.0	8.1	14.1	11.1	5.5	100.0	---	---
AVG SPEED	7.7	6.5	7.9	8.7	12.2	10.6	11.6	6.6	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 9.9											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0											

Table 2.3-9e

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.9 but ≤ -0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	1	0	0	3	0.4	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	5	7	3	4	7	2	1	4	50	7.0	2.62
3.6 - 7.5	6	5	10	32	33	3	9	8	216	30.1	5.34
7.6 - 12.5	11	15	26	18	23	6	1	9	223	31.1	9.63
12.6 - 18.5	2	14	31	16	9	2	0	1	149	20.8	14.69
18.6 - 24.5	1	4	13	10	7	0	1	1	62	8.6	20.90
24.6+	0	0	1	5	0	0	0	0	14	2.0	28.01
TOTAL	25	45	84	87	79	14	12	23	717	100.0	7.22
PERCENT	3.5	6.3	11.7	12.1	11.0	2.0	1.7	3.2	100.0	---	---
AVG SPEED	8.0	10.6	13.1	11.3	8.9	7.8	7.4	8.1	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 10.4											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1											

Table 2.3-9f

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.9 but ≤ -0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	0	0	1	0	1	0	0	0	3	0.4	1.00
CALM+ - 2.0	0	0	0	0	0	0	0	0	0	0.0	0.00
2.1 - 3.5	3	1	2	1	0	4	2	4	50	7.0	2.62
3.6 - 7.5	8	15	18	8	9	18	23	11	216	30.1	5.34
7.6 - 12.5	7	6	19	19	10	27	13	13	223	31.1	9.63
12.6 - 18.5	6	0	5	4	15	25	15	2	149	20.8	14.69
18.6 - 24.5	0	0	0	1	4	10	8	2	62	8.6	20.90
24.6+	0	0	0	0	3	0	2	3	14	2.0	28.01
TOTAL	24	22	45	33	42	84	63	35	717	100.0	7.22
PERCENT	3.3	3.1	6.3	4.6	5.9	11.7	8.8	4.9	100.0	---	---
AVG SPEED	8.2	6.4	8.2	9.5	13.0	11.4	11.6	10.3	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 10.4											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1											

Table 2.3-9g

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.8 but ≤ -0.3 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	7	4	1	1	7	5	1	3	57	0.8	1.00
CALM+ - 2.0	2	0	0	0	1	0	0	1	8	0.1	1.75
2.1 - 3.5	36	47	28	42	62	49	42	50	704	10.1	2.77
3.6 - 7.5	101	66	50	79	132	167	128	105	1706	24.6	5.56
7.6 - 12.5	161	115	77	91	126	110	53	121	2349	33.8	9.65
12.6 - 18.5	29	74	74	92	42	31	9	53	1554	22.4	14.95
18.6 - 24.5	5	13	12	20	16	1	2	6	462	6.7	20.88
24.6+	2	2	3	6	0	0	0	0	103	1.5	27.85
TOTAL	343	321	245	331	386	363	235	339	6943	100.0	6.97
PERCENT	4.9	4.6	3.5	4.8	5.6	5.2	3.4	4.9	100.0	---	---
AVG SPEED	8.4	9.7	10.7	10.4	8.0	7.2	6.4	8.2	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 10.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3											

Table 2.3-9h

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.8 but ≤ -0.3 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	1	4	3	3	4	3	4	6	57	0.8	1.00
CALM+ - 2.0	1	0	1	0	1	1	0	0	8	0.1	1.75
2.1 - 3.5	33	44	62	42	40	42	45	40	704	10.1	2.77
3.6 - 7.5	96	109	166	130	115	134	60	68	1706	24.6	5.56
7.6 - 12.5	168	89	147	216	343	253	116	163	2349	33.8	9.65
12.6 - 18.5	80	21	24	86	331	267	212	129	1554	22.4	14.95
18.6 - 24.5	10	0	1	7	81	132	120	36	462	6.7	20.88
24.6+	0	0	0	2	12	17	49	10	103	1.5	27.85
TOTAL	389	267	404	486	927	849	606	452	6943	100.0	6.97
PERCENT	5.6	3.8	5.8	7.0	13.4	12.2	8.7	6.5	100.0	---	---
AVG SPEED	9.5	7.0	7.2	9.2	12.3	12.7	14.7	11.2	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 10.3											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3											

Table 2.3-9i

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.3 but ≤ 0.8 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	5	6	7	9	12	12	8	5	113	1.3	1.00
CALM+ - 2.0	7	9	6	15	18	17	12	14	181	2.0	1.93
2.1 - 3.5	28	23	27	27	54	56	79	65	963	10.9	2.72
3.6 - 7.5	42	36	23	33	106	213	171	208	3439	38.8	5.48
7.6 - 12.5	41	17	26	22	59	51	26	158	2672	30.1	9.55
12.6 - 18.5	17	4	6	6	11	7	1	47	905	10.2	14.66
18.6 - 24.5	5	1	4	2	0	0	0	6	441	5.0	21.06
24.6+	0	0	0	0	0	0	0	0	151	1.7	27.77
TOTAL	145	96	99	114	260	356	297	503	8865	100.0	5.70
PERCENT	1.6	1.1	1.1	1.3	2.9	4.0	3.4	5.7	100.0	---	---
AVG SPEED	7.5	5.5	6.7	5.5	5.8	5.4	4.8	7.5	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 8.6											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 8											

Table 2.3-9j

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > -0.3 but ≤ 0.8 (°F/100 FT)											
	WIND DIRECTION										
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	9	14	12	3	2	2	4	3	113	1.3	1.00
CALM+ - 2.0	16	22	9	13	9	6	4	4	181	2.0	1.93
2.1 - 3.5	72	127	166	82	52	30	36	39	963	10.9	2.72
3.6 - 7.5	426	473	672	439	281	169	90	57	3439	38.8	5.48
7.6 - 12.5	418	190	305	424	523	294	71	47	2672	30.1	9.55
12.6 - 18.5	149	20	25	110	270	114	74	44	905	10.2	14.66
18.6 - 24.5	25	1	2	28	76	78	157	56	441	5.0	21.06
24.6+	0	0	0	5	20	31	76	19	151	1.7	27.77
TOTAL	1115	847	1191	1104	1233	724	512	269	8865	100.0	5.70
PERCENT	12.6	9.6	13.4	12.5	13.9	8.2	5.8	3.0	100.0	---	---
AVG SPEED	8.6	6.1	6.4	8.4	10.7	11.5	15.6	12.2	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 8.6											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 8											

Table 2.3-9k

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > 0.8 but ≤ 2.2 (°F/100 FT)											
	WIND DIRECTION										
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	2	2	3	2	8	7	3	7	56	2.9	1.00
CALM+ - 2.0	6	3	1	2	1	6	4	7	77	4.0	1.92
2.1 - 3.5	7	8	6	10	38	16	27	28	428	22.1	2.74
3.6 - 7.5	0	4	5	8	32	45	27	74	1155	59.7	5.19
7.6 - 12.5	0	0	6	3	9	3	1	11	203	10.5	8.78
12.6 - 18.5	0	1	1	2	2	2	2	0	16	0.8	13.95
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	15	18	22	27	90	79	64	127	1935	0.0	3.88
PERCENT	0.8	0.9	1.1	1.4	4.7	4.1	3.3	6.6	100.0	---	---
AVG SPEED	2.2	3.6	5.7	5.0	4.4	4.5	4.0	4.6	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 5.0											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7											

Table 2.3-9l

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > 0.8 but ≤ 2.2 (°F/100 FT)											
	WIND DIRECTION										
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	4	6	1	5	1	2	2	1	56	2.9	1.00
CALM+ - 2.0	10	6	8	7	5	5	2	4	77	4.0	1.92
2.1 - 3.5	43	74	89	33	16	15	12	6	428	22.1	2.74
3.6 - 7.5	131	226	294	180	74	26	22	7	1155	59.7	5.19
7.6 - 12.5	30	39	33	27	20	14	6	1	203	10.5	8.78
12.6 - 18.5	0	0	0	2	0	2	2	0	16	0.8	13.95
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	218	351	425	254	116	64	46	19	1935	0.0	3.88
PERCENT	11.3	18.1	22.0	13.1	6.0	3.3	2.4	1.0	100.0	---	---
AVG SPEED	5.2	5.2	4.8	5.4	5.6	5.4	5.4	3.6	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 5.0											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7											

Table 2.3-9m

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > 2.2 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	N	NNE	NE	ENE	E	ESE	SE	SSE	Total	%	GEO MEAN SPD (MPH)
CALM	3	6	0	2	3	6	9	5	75	5.3	1.00
CALM+ - 2.0	4	4	1	6	4	3	3	13	81	5.8	1.93
2.1 - 3.5	3	5	8	15	28	21	21	38	406	28.9	2.74
3.6 - 7.5	2	1	6	8	28	32	27	38	798	56.9	4.96
7.6 - 12.5	0	1	0	0	6	3	1	3	42	3.0	8.58
12.6 - 18.5	0	0	0	0	1	0	0	0	1	0.1	13.16
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	12	17	15	31	70	65	61	97	1403	0.0	3.26
PERCENT	0.9	1.2	1.1	2.2	5.0	4.6	4.3	6.9	100.0	---	---
AVG SPEED	2.3	2.4	3.7	3.0	4.2	4.2	3.4	3.7	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 4.2											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6											

Table 2.3-9n

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION FROM 50-FT LEVEL FOR 1966, 1967, AND 1973-74 (TEMPERATURE DIFFERENCE BETWEEN 150 FT AND 10 FT; ADJUSTED TO 150 FT TO 33 FT. SPEED ADJUSTED TO 33 FT.)

Temperature Difference > 2.2 (°F/100 FT)											
WIND DIRECTION											
SPEED MPH	S	SSW	SW	WSW	W	WNW	NW	NNW	Total	%	GEO MEAN SPD (MPH)
CALM	5	7	5	4	10	8	1	1	75	5.3	1.00
CALM+ - 2.0	9	4	10	7	4	2	2	5	81	5.8	1.93
2.1 - 3.5	50	71	66	27	27	17	8	1	406	28.9	2.74
3.6 - 7.5	94	191	197	119	42	6	4	3	798	56.9	4.96
7.6 - 12.5	5	4	8	4	1	2	2	2	42	3.0	8.58
12.6 - 18.5	0	0	0	0	0	0	0	0	1	0.1	13.16
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6+	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	163	277	286	161	84	35	17	12	1403	0.0	3.26
PERCENT	11.6	19.7	20.4	11.5	6.0	2.5	1.2	0.9	100.0	---	---
AVG SPEED	4.2	4.4	4.5	4.8	3.6	3.1	4.0	3.7	---	---	---
AVERAGE SPEED FOR THIS TABLE EQUALS 4.2											
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6											

Table 2.3-10
GASEOUS DISCHARGE POINTS AT THE GINNA SITE

System	Vent Number
Turbine building ventilation	1
Auxiliary building ventilation system(ABVS)	2
Radwaste building ventilation	2
Containment purge vent	3
Waste gas processing vent	2
Condenser air ejector exhaust	5
Steam generator blowdown exhaust	4
Steam leakage from secondary system	1

Table 2.3-11
VENT DESIGN INFORMATION FOR GINNA

Vent Number	Location	Discharge Elevation Above Grade (m)	Height of Discharge Above Maximum Building Elevation (m)	Effective[a] Vent Diameter (m)	Velocity[a] at Point of Discharge (m/sec)
1	Turbine building roof (with hoods)	NA	Assumed ground release in building wake	NA	NA
2	Plant vent (intermediate building roof)	42.0	1.0	1.8	8.8
3	Containment purge vent (intermediate building vent)	42.0	1.0	0.91	14.4
4	Blowdown tank vent (intermediate building roof, hooded)	NA	Assumed ground release in building wake	NA	NA
5	Air ejector vent (turbine building roof)	NA	Assumed ground release in building wake	NA	NA
NOTE:— NA = Not applicable					

a. Assumed diameter of 0.91 m and velocity of 8.8 m/sec for wake-split runs.

Table 2.3-12
TABULATION OF INPUT ASSUMPTIONS FOR CALCULATIONS

Parameter	Assumed Value or Characteristic
Height of meteorological instruments for stack runs	Not applicable to Ginna
Height of meteorological instruments for ground level releases	33-ft speed and direction, delta T 150-33
Height of meteorological instruments for hourly wake split runs	33 ft and 150 ft
Height of meteorological instruments for wake split runs using joint frequency tables	33 ft
Method for determining stability and diffusion coefficients	Temperature difference using Regulatory Guide 1.23 and Pasquill curves
Calms treatment	Assumed 0.3 mph and assumed to have same direction as measured
Upper limit for σ_Z (m)	1000
Height of tallest structure for computation of Σ(m)	41.0
Vent exit conditions	From Table 2.3-11
Delta-temperature correction factor	0.56 for data prior to July 1975 only
Terrain height	See Table 2.3-13
Terrain correction factors	Figure 2 of Regulatory Guide 1.111

Table 2.3-13

TOPOGRAPHIC ELEVATIONS FEET (MSL) FOR GINNA SITE PLANT GRADE IS 270 FEET

	Distance in Miles																			
Section	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10
N	←	---------------lake---------------																		→
NNE	←	---------------lake---------------																		→
NE	←	---------------lake---------------																		→
ENE	←	---------------lake---------------																		→
E	270	265	265	265	270	270	280	280	290	280	290	280	275	300	300	300	300	300	300	280
ESE	270	330	290	300	335	350	330	360	370	375	370	370	405	405	430	420	455	500	500	430
SE	300	310	330	340	350	375	385	395	415	425	440	450	445	450	500	530	550	460	450	530
SSE	330	320	335	370	385	395	410	470	440	450	450	470	510	500	520	540	520	500	520	470
S	310	350	340	370	380	415	430	450	460	460	480	485	490	500	540	535	530	590	550	490
SSW	270	300	350	375	380	390	405	430	450	470	495	490	500	500	500	540	540	545	525	545
SW	270	315	330	360	360	380	400	405	410	430	450	470	475	450	475	480	475	480	515	490
WSW	275	305	300	330	320	330	325	330	340	340	335	340	350	345	360	365	375	365	370	360
W	280	285	270	270	270	270	250	---	---	lake	---	---	---	---	---	---	---	---	---	→
WNW	←	---	---	---	---	---	---	---	---	lake	---	---	---	---	---	---	---	---	---	→
NW	←	---	---	---	---	---	---	---	---	lake	---	---	---	---	---	---	---	---	---	→
NNW	←	---	---	---	---	---	---	---	---	lake	---	---	---	---	---	---	---	---	---	→

Table 2.3-14

ANNUAL DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS, RELEASE POINT: PLANT VENTS, WAKE-SPLIT

SOURCE: Computer Run ID: GX-3 604-65, 1976

Direction	Distance to ~~Nearest~~ Residence (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q (sec/m^3)~~	D/Q (m^{-2})	Distance to ~~Nearest~~ Vegetable Garden (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec /m^3)	D/Q (m^{-2})	Nearest ~~Site~~ Boundary (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q (sec /m^3)~~	D/Q (m^{-2})
N	lake				NA	NA	NA	NA	lake			
NNE	lake								lake			
NE	lake								lake			
ENE	lake								lake			
E	1200	1.3E-06	1.2E-06	4.4E-08					700	2.5E-06	2.2E-06	9.8E-08
ESE	950	1.1E-06	9.6E-07	4.4E-08					700	1.5E-06	1.4E-06	7.0E-08
SE	500	2.5E-06	2.3E-06	1.4E-07					650	1.8E-06	1.7E-06	9.2E-08
SSE	600	1.4E-06	1.3E-06	5.5E-08					600	1.4E-06	1.3E-06	5.5E-08
S	450	1.6E-06	1.4E-06	6.3E-08					900	1.2E-06	1.1E-06	2.5E-08
SSW	600	7.6E-07	7.0E-07	3.0E-08					500	9.4E-07	8.6E-07	3.9E-08
SW	750	9.9E-07	9.1E-07	3.9E-08					500	1.6E-06	1.5E-06	7.3E-08
WSW	1100	8.2E-07	7.4E-07	1.8E-08					1500	7.1E-07	6.4E-07	1.2E-08
W	1600	7.8E-07	7.1E-07	1.1E-08					1400	8.7E-07	7.9E-07	1.3E-08
WNW	2900	2.0E-07	1.7E-07	1.5E-09					600	8.8E-07	8.0E-07	2.1E-08
NW	lake								lake			
NNW	lake								lake			

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.

Table 2.3-15

GRAZING SEASON DIFFUSION AND DEPOSITION ESTIMATES FOR LIVESTOCK RECEPTOR LOCATIONS, RELEASE POINT: PLANT VENTS, WAKE-SPLIT

SOURCE: Computer Run ID: GX-1 604-64, 1976

Direction	Distance to Nearest Milk Cow (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec/m^3)	D/Q (m^{-2})	Distance to Nearest Meat Animal (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec/m^3)	D/Q (m^{-2})	Distance to Nearest Milk Goat (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec /m^3)	D/Q (m^{-2})
N	lake		NA		lake		NA		lake		NA	
NNE	lake				lake				lake			
NE	lake				lake				lake			
ENE	lake				lake				lake			
E	---	5.7E-08		6.6E-10	---	5.7E-08		6.6E-10	---	5.7E-08		6.6E-10
ESE	8000	4.4E-08		4.5E-10	1000	1.1E-06		3.8E-08	---	4.4E-08		4.5E-10
SE	---	4.7E-08		2.8E-10	2200	5.2E-07		4.7E-09	---	4.7E-08		2.8E-10
SSE	5500	8.1E-08		3.5E-10	4800	9.7E-08		4.3E-10	---	4.2E-08		1.8E-10
S	---	5.9E-08		2.8E-10	---	5.9E-08		2.8E-10	---	5.9E-08		2.8E-10
SSW	7000	8.4E-08		2.7E-10	2200	5.4E-07		3.1E-09	---	6.5E-08		2.1E-10
SW	---	1.0E-07		5.0E-10	2500	8.0E-07		6.4E-09	---	1.0E-07		5.0E-10
WSW	4700	4.9E-07		7.2E-10	---	6.0E-08		2.4E-10	---	6.0E-08		2.4E-10
W	---	4.0E-08		1.7E-10	---	4.0E-08		1.7E-10	---	4.0E-08		1.7E-10
WNW	---	2.8E-08		1.2E-10	---	2.9E-08		1.2E-10	---	2.9E-08		1.2E-10
NW	lake				lake				lake	lake		
NNW	lake				lake				lake	lake		

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.
NOTE:— (-) Indicates receptor distance is greater than 8000 m, diffusion values given are for 8000 m.

Table 2.3-16

GRAZING SEASON DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS, RELEASE POINT: PLANT VENTS, WAKE-SPLIT

SOURCE: Computer Run ID: GX-1 604-64, 1976

Direction	Distance to Nearest Residence (m)	χ/Q (sec/m³)	Depleted χ/Q (sec/m³)	D/Q (m⁻²)	Distance to Nearest Vegetable Garden (m)	χ/Q (sec/m³)	Depleted χ/Q (sec/m³)	D/Q (m⁻²)	Nearest Site Boundary (m)	χ/Q (sec/m³)	Depleted χ/Q (sec/m³)	D/Q (m⁻²)
N	lake		NA		lake		NA		lake		NA	
NNE	lake				lake				lake			
NE	lake				lake				lake			
ENE	lake				lake				lake			
E	1200	1.5E-06		5.1E-08	6600	8.0E-08		9.6E-10	700	2.6E-06		1.0E-07
ESE	950	1.2E-06		4.0E-08	950	1.2E-06		4.0E-08	700	1.6E-06		6.4E-08
SE	500	1.8E-06		5.3E-08	2800-3800	3.3E-07		2.6E-09	650	1.4E-06		3.8E-08
SSE	600	1.2E-06		2.6E-08	3600	1.7E-07		7.9E-10	600	1.2E-06		2.6E-08
S	450	1.7E-06		5.4E-08	2100-4200	8.0E-07		5.7E-09	900	1.7E-06		2.5E-08
SSW	600	8.4E-07		2.3E-08	2300-3600	5.3E-07		2.8E-09	500	9.5E-07		2.9E-08
SW	750	1.4E-06		5.5E-08	5300	2.1E-07		1.1E-09	500	2.3E-06		1.0E-07
WSW	1100	1.0E-06		1.6E-08	1400	9.8E-07		1.2E-08	1500	9.6E-07		1.1E-08
W	1600	6.7E-07		7.0E-09	5600	6.9E-08		3.6E-10	1400	7.4E-07		8.5E-09
WNW	2900	1.7E-07		1.3E-09	4400	7.7E-08		4.5E-10	600	7.6E-07		1.9E-08
NW	lake				lake				lake			
NNW	lake				lake				lake			

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.

Table 2.3-17

ANNUAL DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS, RELEASE POINT: GROUND RELEASE IN BUILDING WAKE

SOURCE: Computer Run ID: GX-5 604-63, 1976

Direction	Distance to ~~Nearest~~ Residence (m)	χ/Q (sec/m³)	Depleted ~~χ/Q (sec/m³)~~	D/Q (m⁻²)	Distance to ~~Nearest~~ Vegetable Garden (m)	χ/Q (sec/m³)	Depleted χ/Q (sec/m³)	D/Q (m⁻²)	Nearest ~~Site~~ Boundary (m)	χ/Q (sec/m³)	Depleted ~~χ/Q (sec /m³)~~	D/Q (m⁻²)
N	lake				NA	NA	NA	NA	lake			
NNE	lake								lake			
NE	lake								lake			
ENE	lake								lake			
E	1200	2.5E-06	2.1E-06	5.7E-08					700	6.1E-06	5.2E-06	1.4E-07
ESE	950	2.3E-06	1.9E-06	5.5E-08					700	3.8E-06	3.3E-06	9.3E-08
SE	500	7.3E-06	6.4E-06	1.8E-07					650	4.8E-06	4.1E-06	1.2E-07
SSE	600	4.2E-06	3.6E-06	7.2E-08					600	4.2E-06	3.6E-06	7.2E-08
S	450	7.8E-06	6.9E-06	1.0E-08					900	2.6E-06	2.2E-06	3.2E-08
SSW	600	4.4E-06	3.8E-06	5.1E-08					500	5.9E-06	5.2E-06	7.0E-08
SW	750	5.7E-06	4.9E-06	5.9E-08					500	1.1E-05	9.4E-06	1.2E-07
WSW	1100	2.3E-06	1.9E-06	2.7E-08					1500	1.4E-06	1.1E-06	1.7E-08
W	1600	1.8E-06	1.4E-06	1.7E-08					1400	2.2E-06	1.7E-06	2.1E-08
WNW	2900	3.7E-07	2.7E-07	2.5E-09					600	5.8E-06	5.0E-06	5.8E-08
NW	lake								lake			
NNW	lake								lake			

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.

Table 2.3-18

GRAZING SEASON DIFFUSION AND DEPOSITION ESTIMATES FOR LIVESTOCK RECEPTOR LOCATIONS, RELEASE POINT: ASSUMED GROUND RELEASE IN BUILDING WAKE

SOURCE: Computer Run ID: GX-4 604-62, 1976

Direction	Distance to ~~Nearest~~ Milk Cow (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q (sec/m^3)~~	D/Q (m^{-2})	Distance to ~~Nearest Meat~~ Animal (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec/m^3)	D/Q (m^{-2})	Distance to ~~Nearest~~ Milk Goat (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q (sec /m^3)~~	D/Q (m^{-2})
N	lake		NA		lake		NA		lake		NA	
NNE	lake				lake				lake			
NE	lake				lake				lake			
ENE	lake				lake				lake			
E	---	6.7E-08		7.9E-10	---	6.7E-08		7.9E-10	---	6.7E-08		7.9E-10
ESE	8000	4.5E-08		5.0E-10	1000	2.6E-06		5.0E-08	---	4.5E-08		5.0E-10
SE	---	4.6E-08		2.9E-10	2200	7.2E-07		6.4E-09	---	4.6E-08		2.9E-10
SSE	5500	7.7E-08		4.2E-10	4800	9.6E-08		5.3E-10	---	3.9E-08		1.8E-10
S	---	5.7E-08		2.8E-10	---	5.7E-08		2.8E-10	---	5.7E-08		2.8E-10
SSW	7000	7.9E-08		2.7E-10	2200	8.9E-07		4.7E-09	---	6.1E-08		2.1E-10
SW	---	1.0E-07		5.1E-10	2500	1.2E-06		8.2E-09	---	6.7E-08		3.2E-10
WSW	4700	1.8E-07		1.1E-09	---	6.7E-08		3.2E-10	---	6.7E-08		3.2E-10
W	---	5.8E-08		2.4E-10	---	5.8E-08		2.4E-10	---	5.8E-08		2.4E-10
WNW	---	4.4E-08		1.9E-10	---	4.4E-08		1.9E-10	---	4.4E-08		1.9E-10
NW	lake				lake				lake	lake		
NNW	lake				lake				lake	lake		

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.
NOTE:— (-) Indicates receptor distance is greater than 8000 m, diffusion values given are for 8000 m.

Table 2.3-19

GRAZING SEASON DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS, RELEASE POINT: ASSUMED GROUND RELEASE IN BUILDING WAKE

SOURCE: Computer Run ID: GX-4 604-62, 1976

Direction	Distance to ~~Nearest~~ Residence (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q~~ ~~(sec/m^3)~~	D/Q (m^{-2})	Distance to ~~Nearest~~ Vegetable Garden (m)	χ/Q (sec/m^3)	Depleted χ/Q (sec/m^3)	D/Q (m^{-2})	Nearest ~~Site~~ Boundary (m)	χ/Q (sec/m^3)	Depleted ~~χ/Q~~ ~~(sec/m^3)~~	D/Q (m^{-2})
N	lake		NA		lake		NA		lake		NA	
NNE	lake				lake				lake			
NE	lake				lake				lake			
ENE	lake				lake				lake			
E	1200	3.2E-06		6.1E-08	6600	9.7E-07		1.2E-09	700	7.7E-06		1.4E-07
ESE	950	2.9E-06		5.5E-08	950	2.9E-06		5.5E-08	700	4.8E-06		9.2E-08
SE	500	8.7E-06		9.8E-08	2800-3800	4.0E-07		3.4E-09	650	5.7E-06		6.1E-08
SSE	600	4.9E-06		4.4E-08	3600	1.8E-07		1.1E-09	600	4.9E-06		4.4E-08
S	450	1.2E-05		1.1E-07	2100-4200	9.6E-07		7.0E-09	900	4.2E-06		3.4E-08
SSW	600	7.4E-06		5.2E-08	2300-3600	8.1E-07		4.2E-09	500	9.9E-06		7.1E-08
SW	750	9.5E-06		8.4E-08	5300	2.1E-07		1.2E-09	500	1.8E-05		1.7E-07
WSW	1100	3.3E-06		2.8E-08	1400	2.3E-06		1.9E-08	1500	2.1E-06		1.7E-08
W	1600	1.6E-06		1.1E-08	5600	1.0E-07		5.1E-10	1400	2.0E-06		1.4E-08
WNW	2900	3.3E-07		2.0E-09	4400	1.3E-07		7.0E-10	600	5.4E-06		4.6E-08
NW	lake				lake				lake			
NNW	lake				lake				lake			

NOTE:— NA indicates that diffusion information for this run was not used in dose calculations for receptors in this column.

Table 2.3-20
EXCLUSION AREA BOUNDARY DISTANCES

Direction[a]	Distance (m)
N	8000[b]
NNE	8000
NE	8000
ENE	8000
E	747
ESE	640
SE	503
SSE	450
S	450
SSW	450
SW	503
WSW	915
W	945
WNW	701
NW	8000
NNW	8000

a. From plant toward exclusion area boundary.
b. For calculational purposes, exclusion area boundary distances offshore were assumed to be 8000 m.

Table 2.3-21
KRPavan DIRECTION-DEPENDENT EAB χ/Q SUMMARY

RELATIVE CONCENTRATION (χ/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0-2 HOURS	0-8 HOURS	0-24 HOURS	1-4 DAYS	4-30 DAYS	ANNUAL AVERAGE	HOURS PER YR MAX 0-2 HR χ/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR
S	450.	1.70E-04	<1.10E-04	8.86E-05	5.53E-05	2.81E-05	1.23E-05	29.3	S
SSW	450.	2.08E-04	1.30E-04	1.02E-04	6.13E-05	2.94E-05	1.19E-05	40.9	SSW
SW	503.	2.11E-04	1.27E-04	9.84E-05	5.67E-05	2.56E-05	9.72E-06	42.7	SW
WSW	915.	1.53E-04	8.91E-05	6.80E-05	3.78E-05<	1.63E-05	5.82E-06	12.8	WSW
W	945.	<1.73E-04	1.07E-04	8.41E-05	4.99E-05	2.73E-05	9.48E-06	14.8	W
WNW	701.	1.29E-04	7.86E-05	6.13E-05	3.57E-05<	1.64E-05	6.34E-06	17.2	WNW
NW	8000.	6.16E-06	2.79E-05	1.88E-06	7.98E-07	2.33E-07	5.16E-08	0.5	NW
NNW	8000.	1.44E-05	6.24E-06	4.11E-06	1.65E-06	4.49E-07	9.10E-08	0.6	NNW
N	8000.	2.25E-05	1.02E-05	6.89E-06	2.93E-06	8.56E-07	1.90E-07	<1.1	N
NNE	8000.	4.01E-05	1.81E-05	1.21E-05	5.12E-06	1.48E-06	3.24E-07	4.5	NNE
NE	8000.	2.24E-05	1.03E-05	7.03E-06	3.04E-06	9.10E-07	2.08E-07	0.3	NE
ENE	8000.	1.96E-05	8.71E-06	5.80E-06	2.40E-06	6.76E-07	1.44E-07	0.1	ENE
E	747.	1.24E-04<	8.15E-05	6.61E-05	4.20E-05	2.19E-05	9.87E-06	2.5	E
ESE	640.	1.56E-04	1.01E-04	8.18E-05	5.13E-05>	2.62E-05	1.15E-05	23.9	ESE
SE	503.	2.17E-04	1.36E-04	1.08E-04	6.48E-05	3.12E-05	1.28E-05	43.7	SE
SSE	450.	1.66E-04	9.90E-05	7.65E-05	4.36E-05	.95E-05	7.28E-06	25.0	SSE
MAX χ/Q		2.17E-04			TOTAL HOURS AROUND SITE:			269.8	
SRP 2.3.4	450.	8.00E-04	4.04E-04	2.87E-04	1.37E-04	4.71E-05	1.28E-05		
SITE LIMIT		0.00E+	0.00E+	0.00E+	0.00E+	0.00E+	1.28E-05		

RELATIVE CONCENTRATION (χ/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0-2 HOURS	0-8 HOURS	0-24 HOURS	1-4 DAYS	4-30 DAYS	ANNUAL AVERAGE	HOURS PER YR MAX 0-2 HR χ/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR

0.5 PERCENT χ/Q TO AN INDIVIDUAL IS LIMITING

NOTE: THE MAXIMUM χ/Q IS UNDERLINED ALONG WITH THE ASSOCIATED SECTOR (SE) VALUES

Table 2.3-22
KRPavan DIRECTION-DEPENDENT LPZ χ/Q SUMMARY

RELATIVE CONCENTRATION (χ/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0-2 HOURS	0-8 HOURS	0-24 HOURS	1-4 DAYS	4-30 DAYS	ANNUAL AVERAGE	HOURS PER YR MAX 0-2 HR χ/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR
S	4827.	7.39E-06	3.72E-06	2.64E-06	1.26E-06	4.32E-07	1.17E-07	6.6	S
SSW	4827.	9.76E-06	4.65E-06	3.21E-06	1.44E-06	4.54E-07	1.11E-07	179.0	SSW
SW	4827.	1.44E-05	6.43E-06	4.30E-06	1.80E-06	5.14E-07	1.11E-07	5.0	SW
WSW	4827.	3.00E-05	1.29E-05	8.47E-06	3.39E-06	9.10E-07>	1.82E-07	14.9>	WSW
W	4827.	3.48E-05	1.60E-05	1.09E-05	4.69E-06	1.40E-06	3.19E-07	15.4	W
WNW	4827.	1.30E-05>	6.07E-06	4.15E-06	1.82E-06	5.57E-07	1.31E-07	7.1	WNW
NW	4827.	1.12E-05	5.35E-06	3.71E-06	1.67E-06	5.31E-07	1.31E-07	8.3	NW
NNW	4827.	2.48E-05	1.14E-05	7.77E-06	3.36E-06	1.01E-06	2.31E-07	10.5	NNW
N	4827.	3.63E-05	1.77E-05	1.24E-05	5.70E-06	1.87E-06	4.77E-07	23.3	N
NNE	4827.	4.97E-05	2.51E-05	1.78E-05	8.50E-06	2.93E-06	7.97E-07	43.7	NNE
NE	4827.	3.52E-05	1.76E-05	1.24E-05	5.82E-06	1.97E-06	5.21E-07	20.9	NE
ENE	4827.	3.31E-05	1.57E-05	1.08E-05	4.84E-05	1.52E-06	3.68E-07	9.8	ENE
E	4827.	1.33E-05	6.68E-06	4.74E-06	2.25E-06	7.74E-07	2.09E-07	5.9	E
ESE	4827.	1.53E-05	7.39E-06	5.14E-06	2.34E-06	7.57E-07	1.90E-07	4.6	ESE
SE	4827.	1.44E-05	6.74E-06	4.60E-06	2.01E-06	6.14E-07	1.44E-07	4.1	SE
SSE	<4827.	6.68E-06	3.12E-06	2.14E-06	9.36E-07	2.86E-07	6.72E-08	2.4	SSE
MAX χ/Q		4.97E-05						361.7	
						TOTAL HOURS AROUND SITE:			
SRP 2.3.4	4827.	<5.31E-05	2.65E-05	1.87E-05	8.82E-06	2.99E-06	<7.97E-07		
SITE LIMIT		3.56E-05	1.90E-05	1.39E-05	7.02E-06	2.64E-06	7.97E-07		

RELATIVE CONCENTRATION (χ/Q) VALUES (SEC/CUBIC METER) VERSUS AVERAGING TIME

DOWNWIND SECTOR	DISTANCE (METERS)	0-2 HOURS	0-8 HOURS	0-24 HOURS	1-4 DAYS	4-30 DAYS	ANNUAL AVERAGE	HOURS PER YR MAX 0-2 HR χ/Q IS EXCEEDED IN SECTOR	DOWNWIND SECTOR

0.5 PERCENT χ/Q TO AN INDIVIDUAL IS LIMITING

NOTE: THE MAXIMUM χ/Q IS UNDERLINED ALONG WITH THE ASSOCIATED SECTOR (NNE) VALUES

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

The hydrology of the site region has been examined to provide a basis for assessing and limiting regulated radioactive liquid releases to the lake, for assessing and mitigating possible effects of accidental radioactive liquid releases on the ground or into the lake, and for establishing high and low-flow water protection criteria.

Lake Ontario, on which the site is located, is about 190 miles long, 50 miles wide, a maximum of 780 ft deep, and covers an area of about 7500 square miles. The average lake level, based on over 100 years of record, is 246 ft mean sea level (msl). The highest instantaneous stillwater level was 250.2 ft msl.

The surface of the land on the southern shore of Lake Ontario, at the site and east and west of it, is either flat or gently rolling. It slopes upward to the south from an elevation of about 255 ft msl near the edge of the lake to 440 ft msl at Ridge Road (New York State Highway 104), 3.5 miles south of the lake.

Water flows into Lake Ontario from other Great Lakes to the west of it through the Niagara River at the west end, from numerous small streams, and from four rivers along the south shore (the Genesee, Oswego, Salmon, and Black). It flows out through the St. Lawrence River at the east end of the lake. There is an annual cycle of water level variation with high water in the late spring or summer and low water in the winter as is indicated in Figure 2.4-1.

There are no perennial streams on the site except Deer Creek, an intermittent stream with a drainage area of about 13.3 square miles (Figure 2.1-2) which enters the site from the west, passes south of the plant, and empties into the lake near the northeastern corner of the site.

The predominant surface currents in Lake Ontario are from west to east and they tend to swing toward the south shore. This has been substantiated by bottle tests which were made from 1892 to 1894 and in the summer of 1957 in the vicinity of Rochester. This water movement would be expected due to the effect of prevailing winds and rotation of the earth.

2.4.2 FLOODS

2.4.2.1 Flood Design Considerations

The probable maximum Lake Ontario water level at the plant site is 250.78 ft msl based on a study conducted in 1968 for RG&E. The report of the study is included as Appendix 2A. The level was revised to 253.28 ft in 1973 based on U.S. Army Corps of Engineers projection (*Reference 1*). This would result from a design tropical storm and associated phenomena. The design-basis flood for the plant site is that resulting from the flooding of Deer Creek. There is no information available regarding major historical flood events in the site region. The probable maximum flood and flooding elevations at the plant site were developed as discussed in Section 2.4.3. The plant is protected from lake flooding by a breakwater with a top elevation of 261 ft. The plant is protected from Deer Creek flooding to an elevation of 273.8 ft to an elevation equivalent to a 26,000 cfs Deer Creek flood.

2.4.2.2 Effects of Local Intense Precipitation

In an evaluation made by the NRC staff of the flood levels which would occur at safety-related buildings assuming an occurrence of the local maximum precipitation on the immediate site area, it was concluded that flood water will pond to an elevation of about 254.5 ft msl at the north area of the site in the vicinity of the screen house. The limiting elevation for safety-related equipment is elevation 254.8 ft (screen house floor elevation of 253.5 plus 1.3 ft to diesel generator buses 17 and 18). Therefore, safety-related equipment would be unaffected by local floods, and the plant would be able to withstand immediate plant area flooding with no detrimental effects.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

2.4.3.1 Flood Evaluation Summary

The RG&E flooding evaluation (*Reference 2*) estimated Deer Creek flood flow discharges using the HEC-1 surface runoff modeling routine (*Reference 3*). This computer program uses the Soil Conservation Services Runoff Curve Number concept and a developed unit response hydrograph in combination with a selected total storm depth and a rain storm distribution (obtained from the U.S. Corps of Engineers) to estimate the watershed flood hydrograph. The 24-hour rainfall depths having return periods of 5 to 100 years were obtained from a rainfall frequency atlas and return periods of 500 years and greater were estimated from a straight line projection on Gumbel extreme probability paper. Rochester Gas and Electric then used these rainfalls in HEC-1 to predict peak discharge rates for various rainfall depths (including the probable maximum precipitation event). The estimated probable maximum flood (PMF) discharge rate is 32,500 cfs. Flooding elevations about the plant were then predicted using the HEC-2 flood routing routine (*Reference 4*).

An independent flooding evaluation was prepared by Franklin Research Center for the NRC staff (*Reference 5*). The NRC study used runoff records from eight small New York State watersheds varying in size from 1.5 to 44.4 square miles, tabulated the maximum discharge of record, and calculated the discharge per unit area and individual watershed return periods by Log Pearson III procedures. The largest discharge per unit area of 284 cfs/mile^2 was for a 13.6 square mile watershed 140 miles from the plant near the Catskill Mountains. The NRC study also predicted the probable maximum flood for Deer Creek using the same HEC-1 computer program model used by RG&E, but with variations in antecedent moisture and rainfall distribution which resulted in a maximum discharge of 38,700 cfs. Flooding depths at the plant were estimated using the same HEC-2 model with some changes in roughness coefficients.

The NRC staff concluded that further analysis should be performed (*Reference 6*). Therefore, RG&E submitted a further analysis to determine water levels across the site from Deer Creek to the screen house, for flood flows up to 38,700 cfs, the largest calculated probable maximum flood (*Reference 7*) (Table 2.4-1). The results of this analysis were a maximum elevation of Deer Creek directly south of the guardhouse of 275.7 ft for the NRC estimated probable maximum flood of 38,700 cfs, 274.8 ft for the RG&E estimated probable maximum flood of 32,500 cfs and 273.8 ft for a flow of 26,000 cfs. A maximum elevation of 262.3 ft msl at the screen house for the 38,700 cfs probable maximum flood was also calculated. Figure 2.4-2 is a north-south cross section of the site showing grade elevations sloping from Deer Creek to Lake Ontario.

The NRC staff recognized that there were inherent conservatisms in its estimate of the probable maximum flood. These conservatisms result in a flood with virtually no chance of being exceeded. The NRC staff reviewed the various conservatisms in the elements of the estimation of the probable maximum flood and made additional estimates of the probability of flooding at

Ginna Station, as described in the following section.

2.4.3.2 Derivation of Probable Maximum Flood

The construction of the probable maximum flood for an ungauged area consists of two elements: selection of the probable maximum precipitation, and development of the runoff hydrograph from this precipitation. From *Reference 8*, ANSI N170-1976, a probable maximum precipitation is defined as the estimated (precipitation) depth for a given duration, drainage area, and time of year for which there is virtually no risk of its being exceeded. The probable maximum precipitation for a given duration and drainage area approaches and approximates the maximum which is physically possible within the limits of contemporary hydrometeorological knowledge and techniques.

The selected probable maximum precipitation rainfall is then transformed into a flood hydrograph by methods that result in a probable maximum flood that is a hypothetical flood (peak discharge, volume, and hydrograph shape) considered to be the most severe reasonably possible based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snow melt.

2.4.3.3 Deleted

2.4.4 LAKE ONTARIO SURGE FLOODING

As a condition of the Full-Term Operating License, the NRC required the placement of additional shoreline erosion protection. This protection was added to ensure minimum wave overtopping of the concrete wall fronting the plant and lower water levels in the vicinity of the screen house. The NRC performed an analysis using procedures from the Shore Protection Manual, U.S. Army Coastal Engineering Research Center, 1977, of the stability and condition of the revetment fronting the plant site and concluded in April 1981 (*Reference 10*) that if the revetment fronting the plant exists as designed it would be capable of resisting surge flooding from Lake Ontario and therefore it would meet current regulatory criteria. Subsequent inspections of the revetment in November and December 1981 showed that the revetment appears to be structurally sound and stable with no evidence of major structure stability problems. Further, the inspections verified that the revetment had not degraded from the original design. Therefore, it was concluded that adequate protection from surge flooding exists at Ginna Station.

2.4.5 ICE EFFECTS

Lake Ontario seldom freezes over but ice does occur in winter, usually along the southern and northern shores and at the northeastern end of the lake.

The possibility of ice blockage of the Deer Creek discharge is considered remote. In the event of such an occurrence combined with maximum surface runoff into Deer Creek, it can be seen from Figure 2.4-4 that the site topography is such as to prevent flooding the plant.

There is a large area immediately east of the plant, where the grade levels are 225 to 260 ft, over which the discharge of Deer Creek could spill and reach the lake before the water level would rise to the 270-ft grade level of the plant. The 270-ft grade level of the plant is also interposed between the channel of Deer Creek and the screen house and the surrounding area between the plant and the lake.

2.4.6 COOLING WATER CANALS AND RESERVOIRS

The ultimate source of cooling water (ultimate heat sink) for Ginna Station is Lake Ontario. The intake structure for the plant is on the lake floor about 3000 ft offshore. Water is conveyed from the intake structure to the screen house through a buried concrete-lined tunnel. The circulating water pumps and the service water pumps are located in the screen house. The intake structure and screen house are described in Section 10.6.2.

2.4.7 FLOODING PROTECTION REQUIREMENTS

The main plant area and buildings are at grade elevation 270.0 ft msl; the north side of the turbine building and the screen house are at elevation 253.5 ft msl. The plant grade entrances to the auxiliary building are at elevation 271 ft msl. The lowest limiting elevation of safety-related equipment in the subbasement within the auxiliary building is 221.5 ft msl.

The plant is protected from lake surges and wind-driven waves by a shoreline revetment with a top elevation of 261.0 ft msl.

The equipment required for safe plant shutdown is located in the auxiliary building and the turbine building. Protection in this area (*Reference 11*) is provided to 273.8 ft msl, which is equivalent to an Ginna LLP estimated discharge flow of 26,000 cfs from Deer Creek. Because the probability of flooding beyond 273.8 ft msl is low, it is the NRC staff's judgment that the probable maximum flood accident sequence will not dominate events potentially leading to core damage. Also, Ginna LLP emergency procedures require installation of flood protection devices well before rising flood waters can jeopardize safe shutdown capability as discussed in Sections 3.4.1.1.3 and 13.5.2.2.3 (*Reference 9*).

2.4.8 LOW WATER CONSIDERATIONS

The lowest monthly average water level for Lake Ontario (at the Oswego gauge) for a 107 year period of record ending in 1967 was 242.68 ft (U.S. Coast and Geodetic Survey Datum). For a 65-year period of record, the lowest instantaneous still water level was 242.17 ft on December 23, 1934. For each year during this period, the instantaneous annual low at the Oswego gauge was not more than 1.02 ft below the corresponding annual monthly low.

For an 8-year period of record at the Rochester gauge, the lowest instantaneous level was 241.38 and the annual instantaneous low was not more than 0.59 ft below the corresponding monthly average low.

The minimum mean monthly lake level of record for Lake Ontario at the Rochester, New York, gauge is elevation 243.0 ft msl. The lowest entrance level into the intake structure is elevation 217.0 ft msl. Having 26 ft of water above the intake structure at minimum lake level is more than adequate to accommodate the maximum setdown (negative surge) for this part of the lake, which is less than 5 ft. Low water conditions for Lake Ontario are discussed in Appendix 2A.

2.4.9 DISPERSION, DILUTION, AND TRAVEL TIMES OF RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

2.4.9.1 Near-Shore Lake Currents

The character of near-shore lake currents during the spring is illustrated by measurements of a polluted mass of water which entered the lake from the Niagara River, March 15, 1933. It moved eastward along the south shore at a rate of about 5 miles per day and was measured in succession by a number of water treatment plants. It was detected at Oswego, about 130 miles

east of the Niagara River on April 11, 26 days after entering the lake.

The surface currents in Lake Ontario are generated primarily by wind stress on the water surface. The lake surface wind-driven currents have speeds which average about 1.6% to 2% of the wind speeds (measured at an elevation of about 70 ft above the lake surface), as was demonstrated by experiments described in Appendix 2B; thus, an average wind speed of 15 mph over the lake would generate an average surface current of about 0.2 to 0.3 mph, or about 5 to 7 miles/day. The flow-in from rivers and flow-out through the St. Lawrence River has a negligible effect by comparison.

Current speeds near shore may be somewhat greater or less than offshore speeds--if less, due to friction close to shore and, if greater, due to long shore currents caused by the piling up of water near the shore which is created by winds with a shoreward velocity component. Measurements near the site made in 1965 (Appendix 2B) indicate that a typical near-shore current is about 0.4 ft/sec (0.27 mph) or 6.5 miles/day toward the east.

Experiments were conducted in 1965 to measure the dispersion of liquids released to the lake under several typical conditions. These are described in Appendix 2B. The experiments involved the release of rhodamine-B dye at a constant rate of about 10 lb/day from a point about 1000 ft offshore for three 3-week periods: one in the spring, one in the summer, and one in the fall. Measurements of dye concentration were made with continuously reading instruments in an accurately navigated boat during and after each release period. The results of these experiments were used to develop estimates of dispersion discussed in Sections 2.4.9.2 and 2.4.9.3 below.

2.4.9.2 Dispersion of Regulated Radioactive Liquid Releases

2.4.9.2.1 Regulated Radioactive Liquid Releases

Regulated releases of radioactive liquids are made intermittently by metered dilution of monitored waste tank effluent into condenser and service water outflow to the lake. During power operation, condenser flow will be about 334,000 gpm and service water outflow about 15,000 gpm.

The annual average concentration of radioactive material attributable to the plant at the point where such outflow enters the lake will be limited so that it will be below the drinking water maximum permissible concentration for unrestricted areas as specified in 10 CFR 20, Appendix B, Table II.

The dose or dose commitment to an individual as calculated in the Offsite Dose Calculation Manual for radioactive materials in liquid effluents released to unrestricted areas is limited during the following items A & B.

A. Any calendar quarter to ≤1.5 mrem to the total body and to ≤5 mrem to any organ.

B. Any calendar year to ≤3 mrem to the total body and ≤10 mrem to any organ.

If the discharge were to be limited to 1/10 maximum permissible concentration, the estimated allowable long-term release rate would be about 5 mCi/sec (primarily tritium) assuming dilution in condenser flow of 334,000 gpm and isotopic composition of releases as shown in Table 11.2-5. The maximum expected long-term average release rate is about 0.05 mCi/sec or about 1/100 of the allowable rate. These estimates are illustrative. Release rate limits are contained in the Offsite Dose Calculation Manual (ODCM). Consideration of re-concentration effects in aquatic biota consumed by humans would not limit allowable release rates.

Liquid waste treatment systems are used to reduce the radioactive materials in liquid wastes prior to their discharge, if necessary, to ensure that cumulative doses due to liquid effluent releases, when averaged over 31 days, does not exceed 0.06 mrem to the total body or 0.2 mrem to any organ.

2.4.9.2.2 Liquid Dispersion

Dispersion of liquids after release into the lake from the site can be estimated by making assumptions concerning the direction and rate of drift of the receiving waters and of the rate of diffusion during injection and drift, including the effects of thermal stratification and shear currents.

For relatively long-term releases (i.e., for a number of hours) at a constant discharge rate, the peak concentration as a function of distance along the direction of mean flow can be predicted by several different theories with equations which differ only by a constant factor. In all of them the concentration is proportional to the reciprocal of distance. The simplest equation for peak concentration as a function of distance is the following one in which the boundary effect of the shore is approximated by doubling concentrations for the unconfined case. The derivation of this equation is described in Appendix 2B.

$$\frac{Sp}{q} = \frac{1}{1.77Dwx}\left(\frac{\sec}{m^3}\right)$$

(Equation 2.4-1)

where:	Sp =	peak concentration (μCi/m^3)
	q =	discharge rate (μCi/sec)
	D =	depth of mixing (m)
	w =	diffusion velocity (m/sec)
	x =	distance from release point (m)

The program of direct dispersion measurements described in Appendix 2B showed that the near-shore region of Lake Ontario near the site is characterized by an average diffusion velocity (w) of 3.3×10^{-3} m/sec. Observations in reservoirs, estuaries, and the ocean range from 2×10^{-3} to 2×10^{-2} m/sec.

Taking w = 3.3×10^{-3} and assuming that the discharged material is confined to the upper 3 m of the lake water, the resulting equation is as follows:

$$\frac{Sp}{q} = \frac{57}{x}\left(\frac{\sec}{m^3}\right)$$

(Equation 2.4-2)

However, near the discharge point this equation is not realistic for the high-volume high-momentum discharge at the site for two reasons: first, because materials will be mixed with the discharge before it is released, and second, because further dilution will occur after release due to

momentum mixing of the 2 ft/sec discharge jet with slower moving lake water. If material is mixed in the full discharge flow of 334,000 gpm, then Sp/q at entry to the lake is 5 x 10^{-2} sec/m^3 or 5 x 10^{-8} μCi/cm^3 per μCi/sec is released. Momentum mixing will cause further dilution by a factor of about 7, 1 mile from the discharge point as is discussed in Appendix 2B. Between about 1 and 8 miles, additional significant dilution of the peak concentration zone (near shore) would not be expected. At distances greater than about 8 miles along the shore, the dilutions can be predicted by the equation. Estimates made on this basis can be summarized as follows:

Distance From Site Along Shore	Sp/q (μCi/cm^3 per μCi/sec)	Dilution Relative to Concentration at Exit from Discharge Canal
In cooling water canal exit to lake	5 x 10^{-8}	1
One mile	7 x 10^{-9}	7
Five miles	< 7 x 10^{-9}	7
Fifteen miles	2.4 x 10^{-9}	15

The predicted maximum concentrations are for steady-state conditions and would occur only with persistent wind direction; therefore, at distances greater than about 20 miles, the diffusion velocity used above is not descriptive since variation in wind direction during the 50 to 70-hour travel time to these positions will produce more dispersion than predicted above.

2.4.9.2.3 Effect of Local Recirculation

Local recirculation from the discharge to the intake, which would produce significantly higher concentrations in the site region than those estimated above, is not expected. The intake for the condenser cooling water is located on the bottom at a depth of 30 ft, about 3000 ft offshore. The density difference produced by heating the condenser cooling water will usually restrict its movement to a surface layer 6 to 10-ft thick until it has mixed with ambient lake water by tenfold or so. As noted above, momentum mixing will dominate in the site region and dilution by 4 to 1 along the direct path from the discharge canal to the surface layer over the intake would be expected if the discharge plume were to be centered over the intake. If the water were then drawn into the intake along with the deeper layer, an additional dilution of approximately threefold would occur to provide a total minimum dilution of approximately twelve-fold, or a recirculation of about 8% for this case. Recirculation would be less than 8% for average conditions where the discharge plume center is not over the intake.

Lake flow reversal in front of the site results in very rapid dilution as indicated in Appendix 2B. It would be expected to cause recirculation of less than 1%.

2.4.9.2.4 Concentration of Nearest Public Water Supply Intake

If discharges average 1/10 maximum permissible concentration at entry to the lake, concentrations on the average at the intake of the nearest public water supply at Ontario 6000 ft east and 1050 ft offshore will be less than 1/10 of this (see Figure 9, Appendix 2B), or less than 1/100 of maximum permissible concentration even if thermal stratification effects are neglected.

2.4.9.2.5 Environmental Monitoring Program

As indicated in the Offsite Dose Calculation Manual (ODCM), an environmental monitoring

program is conducted including radioactivity measurements of aquatic biota and lake surface water. This program provides a check of release limits and a basis for adjusting them if necessary.

2.4.9.3 Dispersion of Accidental Radioactive Liquid Releases

2.4.9.3.1 Accidental Releases to the Lake

If accidental releases to the lake occur over relatively long times (hours), resulting concentrations can be predicted using methods similar to those in Section 2.4.9.2; however, accidental releases, if they occur, might be of relatively short duration (i.e., batch releases).

Estimates of concentration of material released in batches can be made by several theories which predict a time dependence inversely proportional to either the second or third power of time. Available data are inadequate to resolve the differences in these theories, but the use of empirical coefficients permits nearly equal statistical fitting with either of several functions; therefore, the following equation of Okudo and Pritchard is used with experimental coefficients, accommodating the boundary effect of the shoreline by doubling the concentrations for the unconfined case:

$$\frac{Sp}{q'} = \frac{2}{3.14 w^2 t^2 D}$$

(Equation 2.4-3)

where:

where:	Sp =	peak concentration ($\mu Ci/m^3$)
	q' =	activity discharged (μCi)
	w =	diffusion velocity (m/sec)
	t =	time (sec)
	D =	depth of water column (meters)

It is assumed that the equation above applies, that w = 3.3 x 10^{-3} m/sec and D = 3 m, so that

$Sp/q' = 20000/t^2$

and that the mean velocity of the water layer is 0.4 ft/sec. Then peak concentrations at various distances from the site in terms of $\mu Ci/cm^3$ per μCi released will be as follows

Distance (time) From the Site	µCi/cm3 per µCi Released
1 mile (3.66 hours)	1.1 x 10^{-10}
5 miles (18.3 hours)	4.6 x 10^{-12}
15 miles (2.3 days)	5.1 x 10^{-13}

2.4.9.3.2 Accidental Spills on the Ground

Accidental spills of radioactive liquids on the ground in the plant area, if they occur, and to the extent they do not enter the ground, will either run off on the surface into the Deer Creek channel and to the lake, or directly to the lake depending on the location of the spill. That part of a spill which enters the ground would be retained in the ground or would move slowly with the ground water northward into the lake. Ground water, bedrock, and ground surface contours are shown in Plate IIB-3 of the PSAR. As indicated in Plate IIB-3, the ground water level in the plant area generally ranges from about elevation 245 to 250 ft and slopes downward toward the lake. Ground water occurs in the overburden soils in most areas but lies beneath the rock surface in part of the southeastern sector where bedrock surface rises more steeply.

Measurements in one test indicate that the rock is almost impermeable to water flow. Soil permeability was observed in six test pits and a test well (described in Plate IB-4 of the PSAR) and ranged from 10^{-3} to 10^{-6} cm/sec. Most of the ground-water movement within the site will take place in the more permeable soils overlying the rock.

Wells are a source of drinking water in the site vicinity. The wells near the site not owned by Ginna LLC are located mostly along Lake Road east and west of the part of the road which passes through the site. A few are on Ontario Center Road which runs south from Lake Road. The nearest well is approximately 0.5 miles southwest from the center line of the reactor building.

As a result of the stratified nature of the rock, no measurable vertical permeability is indicated. Horizontal bedding limits vertical flow of water through the rock itself, and the cross-bedded nature of the rock precludes any horizontal flow over any appreciable distance. The small grain size, an argillaceous matrix, and the lack of sorting of the grains is not conducive to extensive horizontal permeability. Any movement of water through the rock would have to occur in joints and fractures. The limited extent of joints and fractures in the rock at the site would minimize circulation along these paths. The only opportunity for appreciable movement of water exists near the surface of the rock, where weathering or possible rebound may have opened small joints or fractures. The rock appears to be practically impermeable at the depths sufficient to prevent relief of stresses and consequent open joints. Inspection of the reactor excavation and the relatively dry condition of the tunnels below Lake Ontario confirm this assessment. No flow toward inland wells is expected.

2.4.10 DISPERSION, DILUTION, AND TRAVEL TIMES OF RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

2.4.10.1 Design-Basis Groundwater Level

The original groundwater studies were conducted by Dames & Moore in 1964-1965 (Reference 12). The design-basis groundwater elevation for the screen house and emergency service water structure was 253.5 ft msl, and the design basis for all other safety-related structures was elevation 250 ft msl (Reference 13).

A groundwater monitoring program was established in response to SEP Topic III-3.A to verify the original design-basis groundwater elevation of 250 ft msl (Reference 6). It consisted of three fully encased wells drilled into the groundwater table on the plant site. A liquid level detection and indication unit was installed in one well to continuously monitor and record the groundwater level. The other two wells were available if more data was needed to establish the design basis groundwater level. As a result of monitoring of groundwater levels over a 4-year period from 1983 through 1987 the new design-basis groundwater level was determined to be at elevation 265.0 ft msl. This value was based on a peak groundwater level of 264.69 ft and using a 2%

maximum expected error in the recording system. An engineering evaluation was performed of the effects of the new design-basis groundwater level on safety related structures below grade. As a conservative approach, the engineering evaluation considered a design-basis groundwater level at grade elevation 270.0 ft msl or 5 ft higher than the new design-basis level. The evaluation was based on finite element analysis of elastic plates utilizing the MacNeil Schwindler Corporation (MSC) PAL2 computer program and conventional structural engineering techniques. Pressure loads considered in the analysis consisted of hydrostatic, soil, and soil-induced earthquake forces. Four walls representative of the worst-case load conditions of all below-grade safety-related areas of the auxiliary, intermediate, and control buildings were selected for the engineering evaluation. The evaluation demonstrated that the below-grade safety-related structures were adequately designed to resist the design loads associated with groundwater levels at grade (270.0 ft msl) without requiring strengthening modifications.

2.4.10.1 Groundwater Protection Program

In 1995 a study (*Reference 14*) was initiated by RG&E and prepared by Dr. Robert Poreda (Professor of Earth and Environmental Sciences at the University of Rochester) to determine the impacts of spent fuel leakage on the environment. This study concluded:

- No significant amount of water from the spent fuel pool (SFP) was migrating off-site.
- Ginna's drain system around Containment provides an effective "capture mechanism" for potential leaks to the groundwater.
- The facility's sub-grade structure acts as a hydraulic barrier (i.e., barrier to groundwater flow); any groundwater leakage through structural imperfections enter the Containment Building's drain system which leads to the sub-basement sump from where it is processed as radwaste.

The study also confirmed that groundwater generally moves from south to north across the site, as was determined in the initial site characterization study (*Reference 12*).

As part of the 1995 study, down gradient groundwater monitoring wells installed to verify the absence of tritium in the groundwater unexpectedly identified tritium concentrations above background levels. When overboard blowdown of the secondary coolant was initiated in the spring of 1996 in preparation for refueling outage (RFO), high levels of tritium were detected in the down gradient wells. Engineering determined that the underground blowdown canal was degraded and introducing secondary coolant to the groundwater under the turbine building. Chemistry determined that leakage from the intermediate building north wall contained tritium and short-lived radioiodine, confirming the reactor coolant system (RCS) rather than the SFP as the source. Secondary coolant system radioactivity was elevated at that time due to significant primary to secondary leakage. The release of tritium to the groundwater was stopped when the underground blowdown canal was repaired and abandoned and the primary to secondary leakage was eliminated with steam generator replacement. The total tritium released from the site to Lake Ontario via groundwater for the 1996 event was calculated at approximately 1.2E-3 Ci. Since that time, down gradient groundwater monitoring well tritium concentrations have decreased due to diffusion, dispersion, and decay.

In 2006, the industry began a voluntary groundwater protection program involving all nuclear power plant operators to improve the management of situations involving radiological releases to groundwater. Nuclear Energy Institute's industry guideline for the groundwater protection program, "Industry Ground Water Protection Initiative: Final Guidance Document," NEI 07-07, was released in August 2007. New wells (Screenhouse East wells) were installed in 2007 in an effort to enhance the groundwater monitoring program. These wells are located down gradient of

the previous monitoring wells to serve as additional and redundant down gradient monitoring points for any potential plant releases. The site's existing groundwater monitoring wells were upgraded to include locking caps and bollards in August 2010. As part of this project, a new groundwater control well was installed approximately 2,000 feet southwest of the reactor. The groundwater protection program was further enhanced in September 2013, when 5 new wells were installed outside of the protected area for increased monitoring. As of September 2013, the site maintained 13 onsite groundwater monitoring wells, and 2 control groundwater monitoring wells. Additionally, water from four onsite storm water catch basins is periodically collected and analyzed as part of the site's Groundwater Protection Program.

2.4.10.2 Water Use

Lake Ontario water is used for industrial and domestic water supplies, recreation, domestic and international shipping, and a limited amount of commercial fishing. A description of historical water intakes on the southern shore of the lake is given in Table 2.4-2.

The town of Ontario has a domestic water intake 1.1 miles east of the site extending approximately 4,000 ft from shore. The demand on this system is approximately 2,200,000 gallons per 24 hour period.

The capacity of the Ontario water treatment system is 3.0 million gallons per day. The Ontario water system supplies water to the towns of Walworth and Macedon to the south of Ontario, and to the town of Marion to the southeast of Ontario. Marion also purchases water from Williamson to its north. Macedon is partly supplied by a metered connection with the Monroe County Water Authority. The town of Walworth has emergency connections with the Monroe County Water Authority. The town of Ontario has emergency connections with the towns of Williamson and Webster to its east and west, respectively.

During the construction phase of Ginna in the 1960's, a tracer release and dilution study was conducted to determine dilution factors at the Ontario Water District Intake, which was then located 1,100 ft offshore and at a depth of 11 ft. Dye was released continuously for more than 10 days in the spring, summer, and fall of 1965 and concentrations were measured in the study area. Contours of tracer concentrations were mapped and various diffusion analysis were conducted based on the observed tracer concentration and currents under prevailing wind conditions during the study. The 1965 Study projected dilution factors at the Ontario Water District Intake to be 1:20 at the bottom under complete vertically mixed conditions.

In 2002, the Ontario Water District relocated its water intake structure to its current location, which is about 4,000 ft from the shoreline and at about 40 ft water depth. In 2010, a more comprehensive, computer-based hydrodynamic and thermal model was developed to quantify the dilution factors between the Ginna Station and the Ontario Water District Intake. This model incorporated environmental processes that affect dilution, including lake water elevation, three dimensional current, diffusion, and temperature variations. Annual average dilution factors at the Ontario Water District Intake were estimated to be approximately 1:360 at the bottom. The monthly average dilution factors at the bottom of the lake at the intake were determined to be greater than or equal to 1:200 (*Reference 15*).

In 2013, the Monroe County Water Authority finished construction on a water treatment plant in Webster, New York. The point of intake for this facility is 6,025 feet from the shoreline of Lake Ontario. The demand on the Webster plant in a 24-hour period is approximately 20,000,000 gallons. The maximum daily capacity of the site is 50 MGD.

REFERENCES FOR SECTION 2.4

1. Letter from K. W. Amish, RG&E, to D. I. Skovholt, NRC, Subject: Armor Stone Addition to Ginna Breakwall, dated May 15, 1973.

2. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topics II-3.A, II-3.B, II-3.B.1, III-3.A - R. E. Ginna Nuclear Power Plant (with Attachment - Ginna Station Design Basis Flooding Study for Rochester Gas and Electric Corporation, August 1981, NUS Corporation), dated August 18, 1981.

3. U.S. Department of the Army, Corps of Engineers, "HEC-1 Flood Hydrograph Package," Hydrologic Engineering Center, Users Manual, (February 1981 version), July 1981.

4. U.S. Department of the Army, Corps of Engineers, "HEC-2 Water Surface Profiles," Hydrologic Engineering Center, Users Manual, (April 1980 version), January 1981.

5. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Ginna Nuclear Power Plant - Final Evaluation of SEP Topics II-3.A, II-3.B, II-3.C, and II-4.D, dated May 27, 1982.

6. U.S. Nuclear Regulatory Commission, Integrated Plant Safety Assessment, Systematic Evaluation Program, R. E. Ginna Nuclear Power Plant, Final Report, NUREG 0821, December 1982.

7. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic II-3.B, Deer Creek Flooding, dated January 31, 1983.

8. American Nuclear Society, "Standards for Determining Design Basis Flooding at Power Reactor Sites," ANSI N170-1976, ANS-2.8, La Grange Park, Illinois.

9. Letter from D. M. Crutchfield, NRC, to J. E. Maier, NRC, Subject: Integrated Plant Safety Assessment (IPSAR) Section 4.5, Plant Flooding by Deer Creek - R. E. Ginna Nuclear Power Plant, dated August 19, 1983.

10. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Evaluation of SEP Topics II-3.A, 3.B, 3.B.1, and 3.C, dated April 10, 1981.

11. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic II-3.B, Deer Creek Flooding, dated May 20, 1983.

12. Dames & Moore, Site Evaluation Study, Proposed Brookwood (Ginna) Nuclear Power Plant, June 14, 1965.

13. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Ginna Nuclear Power Plant - Final Evaluation of SEP Topics II-3.A, II-3.B, and II-3.C, dated January 28, 1981.

14. Robert J. Poreda, "Hydrology of the Ginna Power Station," dated May 31, 1996.

15. HydroQual, Inc., "R. E. Ginna Nuclear Power Plant Tracer Dilution Study for the Town of Ontario Municipal Drinking Water Intake - Final Report," dated May 28, 2010.

Table 2.4-1
DEER CREEK OVERFLOW SUMMARY TABLE

Total Flood Flow (cfs)	Elevation at Screen House (ft)	Elevation at Deer Creek Section 2380[a] (ft)
14,600[b]	253.5	270.0
15,000[c]	253.55	270.1
16,000	253.7	270.6
17,300[d]	254.0	271.1
18,000	254.2	271.4
20,000	254.8	272.1
20,600	255.0	272.3
22,000	255.4	272.8
24,000	256.0	273.3
26,000[f]	256.0	273.8
28,000	257.8	274.2
30,000	259.0	274.5
35,000	261.6	275.1
38,700[e]	262.3	275.7

a. About 100 ft. west of bridge over Deer Creek leading to plant
b. Channel capacity
c. Standard project flood
d. Standard project flood plus 1 ft.
e. Probable maximum flood
f. NRC staff approved design flood level

Table 2.4-2
INDUSTRIAL AND MUNICIPAL WATER SUPPLIES

Name and Location	Type of Water Use	Quantity Used	Treatment Before Use	Location With Respect to Ginna Site (miles)	Comments
Ontario Water District, Ontario	Domestic	1,800,000 gpd	Filtration chlorination	1.1 east	Intake 1050 ft from shore; Serves 3 adjacent towns
Williamson Water District, Williamson	Domestic	149,000 gpd	Filtration chlorination	5.25 east	Serves 4 adjacent water districts
Sodus Point Water District, Sodus	Domestic	84,000 gpd	Filtration chlorination	15 east	Serves South Shore water district
Wolcott	Domestic	240,000 gpd	Filtration chlorination	24 east	Auxiliary source
Comstock Foods, Incorporated, Red Creek	Industrail cooling	100 gpm	Chlorination	25 east	Operates during months of October and November
Marathon Corporation, Oswego	Industrial process	3 to 4 mgd	Rapid sand filtration, chlorination	41 east	Water treatment plant has 5 mgd capacity. Intake point about 250 ft from shore
Niagara Mohawk Power Corporation, Oswego	Cooling	500 mgd	None	41 east	---
Oswego City	Domestic	5.0 mgd	Chlorination	41 east	New intake under construction; Serves 4 adjacent water districts
Queensboro Farm Products, Incorporated, Lycoming	Boiler and cooling water	Not known	None	46 east	---
RG&E Russell Station, Greece	Condenser cooling	166 mgd	None	16 west	Intake extends 3660 ft from shore
Eastman Kodak Company Waterworks, Greece	Industrial processing	17.3 mgd	Filtration chlorination	16 west	Two intakes 700 ft apart and extending 7800 ft from shore
Rochester Public Water Supply, Greece	Domestic	34 mgd	Filtration chlorination	16 west	Pumped fom Eastman Kodak Company intake pipe
New York Water Service Corporation, Rochester Plant, Greece	Domestic	13.5 mgd	Filtration chlorination	16 west	Two intakes 100 ft apart and extending 4000 ft from shore

Name and Location	Type of Water Use	Quantity Used	Treatment Before Use	Location With Respect to Ginna Site (miles)	Comments
Hilton Public Water Supply, Parma	Domestic	0.2 mgd	Filtration chlorination	24 west	Intake extends 350 ft from shore
Brockport Public Water Supply, Hamlin	Domestic	1.3 mgd	Filtration chlorination	30 west	Intake extends 2600 ft from shore
Lyndonville Public Water Supply, Yates	Domestic	68,000 gpd	Filtration chlorination	53 west	Intake extends 530 ft from shore
Barker Public Water Supply, Somerset	Domestic	0.1 mgd	Filtration chlorination	62 west	Intake extends 600 ft from shore
Newfane Water District No. 1, Newfane	Domestic	147,000 gpd	Filtration chlorination	68 west	Intake extends about 600 ft from shore
Wilson Public Water Supply, Wilson	Domestic	175,000 gpd	Filtration chlorination	76 west	Intake extends 450 ft from shore

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

A geological program involving a regional geological survey, borings, and other tests at the site was conducted to provide information needed to assess foundation conditions, seismic activity, and ground-water conditions. The details of these investigations are reported in detail in Appendix 2C (and in the PSAR, Volume 1, Appendix D). Additional studies were performed in 1973 as part of the Sterling alternative site evaluation. This is described in Section 2.5.2.3.

These results and subsequent information discussed below indicate that the rock and compact granular soil on the site provide a suitable foundation for plant structures with allowable bearing pressures in the range of 3 to 5 tons/ft^2 for spread or mat foundations on the compact granular soils and 35 tons/ft^2 on bedrock.

2.5.1.1 Regional Geology

The site is located on the southern shore of Lake Ontario in the eastern portion of the Erie-Ontario Lowlands Physiographic Province (Fenneman, 1938). The regional topography is of low relief and rises gradually from an elevation of +250 mean sea level (msl) at the lake to +500 ft msl at the Portage Escarpment, which is the northern boundary of the Appalachian Plateau Province to the south. A beach ridge 10- to 25-ft high parallels the shoreline of Lake Ontario 4 miles to the south. North of the ridge is the lake plain of former glacial Lake Iroquois. The site lies on this plain.

The southern margin of Lake Ontario is characterized by many promontories which seem to reflect prominent joint directions in bedrock. The site is located near one such promontory called Smokey Point. Major joint directions are north 75° to 85° east and north 10° east to 30° west. Erosional bluffs along the lake range from 15- to 30-ft high. Smokey Point is located at the eastern end of a 5-mile-long ridge, the crest of which is about +310 ft. Relief in the site area is low, with elevations ranging from +350 to +300 ft. The site is underlain by 20 to 60 ft of glacial deposits and approximately 2700 ft of Paleozoic (570 million years to 225 million years before present) sedimentary rocks over crystalline basement. The uppermost Paleozoic unit is sandstone of Upper Ordovician (455 to 430 million years before present) Queenston formation. The Queenston is roughly 1000-ft thick in this area and overlays approximately 80 ft of Oswego sandstone, approximately 600 ft of Lorraine shales, and probably less than 30 ft of Potsdam sandstone. The pre-Cambrian surface is roughly 2600- to 2700-ft deep at the site.

The glacial deposits include at least two till horizons. The lower unit overlies bedrock and varies in thickness from 6 to 25 ft. This unit consists of grayish-red, calcareous, silty clay. The unit is poorly sorted and contains numerous striated and faceted pebbles, cobbles, and boulders. The upper till unit is at or near the ground surface and ranges from 7 to 30 ft in thickness. This unit is composed of relatively uniform olive-gray to yellow-brown silty,

sandy clay, with large boulders several feet in diameter. Between the two till horizons is a zone of lakebed deposits consisting of gray, very plastic clay.

Rochester Gas and Electric Corporation has determined by regional correlation that the lower till unit is associated with the Woodfordian glacial advance, a substage of the Wisconsinan Stage, which took place about 22,000 years ago. The lakebed deposit is believed to have been deposited in the bed of Lake Iroquois. The upper till is related to a minor glacial readvancement that occurred about 12,000 years ago.

2.5.1.2 Site Geology

The major Ginna Station structures are supported in the Queenston formation or atop a thin layer of natural or compacted granular soils immediately above the bedrock. The Queenston formation, which is generally found at depths of 30 to 40 ft, is composed of alternating strata of thinly to thickly bedded, dense, fine-grained sandstone, silty and sandy siltstone, with occasional thin beds of fissile shale. Bedding is essentially horizontal with occasional cross-bedding and shaly partings. The color is predominately red, but random green blotches and layers occur throughout the depths explored. Occasional continuous vertical joints were noted in the borings and during site inspections.

Subsequent to the initial environmental studies, seven additional borings were drilled to depths between 35 and 90 ft in the reactor area for a supplementary foundation study. The locations of these borings are shown in Figure 2.5-1. The soil and rock encountered in the seven borings were similar in all respects to the onsite materials described in the PSAR.

Nine borings were drilled for the proposed intake and discharge tunnels. As shown in Figure 2.5-1, these borings extended from the shore to a distance of about 3000 ft into Lake Ontario.

Prior to construction of the plant foundations, the soil overburden (30 to 40 ft of glacial drift) was removed. The exposed rock surface was observed to be similar to that examined in nearby outcrops. Bedding was horizontal and occasional cross-bedding and shaly partings were evident. A pattern of vertical joints of limited vertical extent was evident in the outcropping rock, particularly along the lakeshore side of the excavation. The observed joints continued to depths of from 20 to 30 ft from the top of the rock, but no evidence of movement along the joints was found. The major joint systems were found to be in accordance with those trends reported in the PSAR. Some minor exfoliation noted in the bottom of the excavation is believed to have been caused primarily by the heavy equipment traffic on the excavation floor and the drying effects of exposure to air.

The cores extracted in the nine borings drilled for the intake structure investigation were compared with the cores of the previous borings drilled at the site. As expected, the rock encountered below the lake was consistent with the rock encountered in onshore borings.

The onshore shaft and tunnels were inspected during construction as well as after completion of the tunneling. Examination of the exposed rock revealed conditions consistent with those encountered during the previous studies. No zones of defective rock were found and no weathered rock was evident in the tunnels. The rock in both tunnels is sound. Water flow was practically nonexistent, being essentially limited to scattered areas of minor moisture

infiltration. The actual conditions found in the tunnel excavations were in agreement with those encountered in all previous borings drilled during the initial subsurface investigation and the other supplementary investigations.

2.5.2 VIBRATORY GROUND MOTION

A seismological program was carried out to provide information for predicting possible seismic effects at the site. Estimates of such effects which are described in this section indicate that the seismic design criteria set forth in Section 3.7 are conservative. Field investigations and predictions are described in the PSAR, Volume 1, Appendix D.

The site is within 150 miles of the St. Lawrence valley area where earthquakes of Richter magnitude 7.0 have been experienced. It is within 50 miles of the area around Buffalo which has experienced moderate earthquake activity of a smaller magnitude, and within 35 miles of the fault system near Attica. Historical and physical evidence described in Appendix D, Volume 1, of the PSAR indicates that the site is seismologically quiet.

2.5.2.1 Seismicity

The following explorations were made to evaluate the seismological characteristics of the Ginna site.

A. An investigation of the earthquake history of the northeastern United States and eastern Canada was used to develop estimates of the maximum expected and maximum credible earthquake which could affect the site. All recorded earthquakes in this region with Modified Mercalli Intensity of V or greater were plotted and considered. Figure 2.5-2 is an updated epicentral map. Table 2.5-1 lists nearby earthquake activity in the mid-1960s.

B. Investigations were made on the site and in the surrounding area to search for any evidence of seismic activity such as would be indicated by faulting. This involved examination of outcrops, including dip and strike measurements, and the development of a bedrock surface profile from onsite borings, probings, and a shallow and deep refraction survey.

C. Microtremor measurements of ground motion and deep refraction surveys to measure the elastic properties of bedrock were made to provide a basis for estimating effects at the site of the maximum expected and maximum potential earthquakes.

The northeastern United States and eastern Canada are moderately active earthquake areas as indicated in Figure 2.5-2. However, there is no instrumental or verifiable record of extremely large magnitude shocks (above Richter 8) and as indicated on Figure 2.5-2, there is no record of damaging earthquakes with epicenters within 50 miles of the site.

2.5.2.2 Maximum Earthquake Potential

The historical record indicates the maximum earthquakes to be expected in the site region are the following:

A. A shock of epicentral intensity VIII (Modified Mercalli Scale) at a distance of about 60 miles (similar to the 1929 Attica shock, which is judged to be less than Richter magnitude 6).

B. A shock of epicentral intensity VIII (Richter magnitude 5.5) at a distance of 110 miles (similar to the 1914 Lanark shock).

C. A major shock (Richter magnitude 7.0) far to the east, near Montreal, 200 or more miles away.

These maximum expected earthquakes would not result in significant ground motion at the site. Ground acceleration at the site is estimated to be less than 1% of gravity. It is judged that the maximum credible earthquake would be one of Richter magnitude 6.0 with an epicenter 60 miles from the site or one of magnitude 7.0 at a 90-mile epicentral distance.

A procedure developed by Dames & Moore, using the results of research at the Earthquake Institute in Tokyo, was used to estimate ground motion at a given location if the earthquake magnitude, epicentral distance, and elastic properties of foundation soils and rock are known. Using this method and the assumed maximum credible earthquakes discussed above, maximum acceleration on the site was calculated to be 8% of gravity for soil surface and 7% for bedrock surface. Plant structures, systems, and components designated as Seismic Category I (see Section 3.7) are designed to remain within applicable stress limits for the operating-basis earthquake (0.08g) and the safe shutdown earthquake (0.20g). The ground motion spectrum used in the design are shown in Figures 3.7-1 and 3.7-2.

In 1980, the NRC developed site-specific ground response spectra for the eastern United States. The spectra established ground motion acceleration values to be used in structural analyses to determine seismic loads at those eastern power plants that were a part of the NRC's Systematic Evaluation Program. The ground response spectrum for the Ginna site is shown in Figures 2.5-3 and 3.7-3.

2.5.2.3 Surface Faulting

2.5.2.3.1 Nearby Regional Faulting

Within the Ontario lowlands, the nearest regional faulting is the Clarendon-Linden structure near Batavia, New York. The structure trends north-south and is about 35 miles west of Ginna Station. The fault is described as a complex faulted zone with a major north-south set of subparallel normal and reverse faults that have a cumulative displacement of approximately 100 m with east-side up (*Reference 1*). Data suggest that the zone is continuous to the north across Lake Ontario for a total length of as much as 180 km.

No unequivocal evidence of postglacial faulting was found among 36 faults, 6716 joints, and 87 pop-ups studied around the Clarendon-Linden fault system (*Reference 1*). However, numerous earthquakes, including the 1929 Modified Mercalli Intensity VIII earthquake, have occurred within the fault system near Attica. A number of seismologists have concluded that these events are probably related to solution mining of salt.

The presence of faults has been documented at the Nine Mile Point and FitzPatrick nuclear sites approximately 50 miles east of Ginna. The structures are three west-northwest striking high-angle faults, and several north-south striking thrust faults and folds. Displacements range from inches to several feet. Several of the faults mapped at Nine Mile Point Unit 2 have been shown to have undergone some movement during the last 10,000 years. The most

recent displacements are most likely associated with the complex phenomena caused by glacial loading and unloading. However, no such post-Pleistocene (less than 10 million years before present) faults have been identified at Ginna Station.

A structural complex was also discovered at the proposed New Haven site located a few miles east of Nine Mile Point. These structures consist of a large northeast striking anticline with several associated faults. The folds and faults were demonstrated by the applicant to be non-capable (*Reference 2*).

Several minor normal faults with 2 to 15 ft of displacements have been identified between the site and northward projection of the Clarendon-Linden fault. There is no evidence that indicates post-Pleistocene movement along these faults.

2.5.2.3.2 Ginna Site Vicinity Faulting

During an investigation conducted by RG&E in 1973 for an alternate nuclear site adjacent to the Ginna site (proposed Sterling Power Project), evidence of faults was found in core borings. An extensive investigation program was carried out. The investigations included a large trench excavated across the fault zone, additional borings, petrographic and mineralogical analyses, testing of samples from the fault zones, geophysical explorations, and surface geological mapping.

The studies revealed that the fault zone was comprised of three down-to-the-northeast faults that trended north 65° west. The maximum offset is about 26 ft which decreases to about 6 ft to the southeast near the plant. The fault zone passes about 30 ft southwest of the reactor complex. Three geological reconnaissances were made by a staff geologist at the site to review progress of the investigations and examine features exposed in trenches across the fault zone.

A large trench across the fault revealed extensive deformation of glacially deposited horizons but there was no deformation that was directly attributable to tectonic movement along the faults. The strongest evidence that these deformations are not related to tectonic displacement on the bedrock faults is the presence of a horizontal unit at the base of the lower till that lies undisturbed across the southernmost fault and stacking planes (imbricate thrust sheets caused by the southward advancement of the glacier) that cut across the faults without displacements.

Rochester Gas and Electric Corporation also attempted to determine the age of the fault gouge by radiometric techniques but the results were unreliable. However, other lines of evidence indicate a much older age of last movement than Pleistocene. This evidence includes the following:

A. The observation that the contemporary stress field is different from that in which the fault originated. According to Sbar and Sykes, (*Reference 3*) the contemporary stress picture in western New York is one of nearly horizontal compression oriented in an east-west direction. Evidence for this is local squeeze and pop-up features and in situ stress measurements in the region. The existing stress field is not consistent either in orientation or type of stress

field in which the faults were formed; and the stress regime in which the faults were formed was essentially northeast-southwest and tensional.

B. The presence of unsheared hydrothermal crystals within the fault zone demonstrate that faulting predates the hydrothermal event which deposited the crystals and this event probably occurred no later than the Cretaceous (65 million years ago). Analyses carried out by consultants to RG&E show that the mineralization of fluid inclusions in calcite crystals along with sulfide mineralization, particularly pyrrhotite and molybdenite, more than likely reflect hydrothermal mineralization at temperatures of at least 225°C to 300°C. The last known tectonic environment within which such conditions could have developed in the area was about 65 million years ago.

C. No recorded historic earthquake has occurred which could be associated with the faults.

It is concluded that the faults at least predate the latest major glacial advance which occurred about 22,000 years ago. The weight of all the available information indicates that the faults are more than 65 million years old.

Additional information pertaining to the evaluation discussed above can be found in the *Additional References for Section 2.5*.

2.5.2.3.3 Ginna Excavation

Construction photographs of the Ginna excavation were also examined by the NRC staff. There were ample fair-quality photos to cover most of the walls of the major excavation. Bedrock bedding could be clearly seen in many of the photographs and, although there are numerous joints, there was no indication of displacement. It is concluded that there is no faulting directly beneath the major Seismic Category I structures of the plant.

2.5.3 STABILITY OF SLOPES

2.5.3.1 General

This topic pertains to the stability of all slopes, whose failure could adversely affect the safety of the plant. The scope of the topic discusses the following subjects: (1) slope characteristics, (2) design criteria and analyses, (3) results of field and laboratory tests, (4) excavation, backfill, and earthwork in slopes, (5) liquefaction potential affecting slopes, and (6) instrumentation and performance monitoring.

The applicable rules and basic acceptance criteria pertinent to this topic are the following:

A. 10 CFR 50, Appendix A: General Design Criteria 1, 2, and 4.

B. 10 CFR 100, Appendix A.

C. Regulatory Guides.

1. Regulatory Guide 1.132, Site Investigations for Foundations of Nuclear Power Plants.
2. Regulatory Guide 1.138, Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants.

2.5.3.2 Onsite Slopes

Two onsite slopes, whose failures may be of safety concern, were identified by RG&E (*Reference 4*). The first slope is located about 200 ft northwest of the turbine building while the second slope is located east of the screen house. Both slopes were excavated from the original ground elevation of about 270 ft down to elevation 255 ft in silty clay soil and were graded at approximately 7.5 horizontal to 1 vertical.

The subsurface exploration program of 1964 revealed that the bedrock of red siltstone was at depths ranging from 30 to 40 ft below the original ground surface (*Reference 5*). The overburden soils consisted of reddish-brown clayey silt, silty clay, and sand and gravel layers. The thicknesses and the engineering properties of those soils varied considerably throughout the site.

One boring (No. 1) was drilled at the first slope, and two borings (No. 3 and No. 119) were drilled at the second slope. The laboratory tests performed in 1964 were very limited and the shear strengths of the soft clayey soil varied in a wide range. In order to assess the stability of those slopes, assumptions have been made about the subsurface conditions and the soil parameters. The sectional profile of the first slope was assumed to be represented by boring No. 1, the second slope by boring No. 3. Conservative soil parameters obtained from the 1964 investigation were used in the slope stability analyses.

2.5.3.3 Stability Analyses

Stability analyses, both static and pseudostatic with earthquake load, were performed by the NRC staff using a commercially available computer program, MCAUTO's "Slope" program. Material properties which controlled the stability analyses are shown in Table 2.5-2.

The results of the slope analyses performed by the NRC staff during the Systematic Evaluation Program show that the factors of safety against slope failure under both static and earthquake loading conditions are less than unity, indicating that these slopes are not stable and that failure would take place along an arc of radius about 175 ft. The NRC staff believes that the shear strength of the in situ silty clay soil should have gained strength because of consolidation of the clayey soil, but there is no new data about the in situ soil conditions and strengths, so reasonably conservative soil data has been used by the staff in the analyses.

2.5.3.4 Failure Evaluation

Since the slopes were not determined to be stable, the impact of their failures was further evaluated by the NRC staff. The most critical failure arc, as calculated, would intercept the slope at elevation 276 ft, adjacent to the crest and at elevation 257 ft, adjacent to the toe. The lateral spread of the slope failure adjacent to the toe is estimated by the staff to be somewhere around 8 ft, based on postfailure equilibrium.

At the first slope, northwest of the turbine building, there is no structure nor equipment located within or adjacent to the slope except a roadway. Therefore, the failure of that slope would not pose any safety concern but might close the road. The second slope, east of the screen house, is sufficiently removed from any required safety-related equipment. Thus, its failure would not be of safety concern.

REFERENCES FOR SECTION 2.5

1. R. H. Fakundiny, P. W. Pomeroy, J. W. Pferd, T. A. Nowak, Jr., and J. C. Meyer, Structural Instability Features in the Vicinity of the Clarendon Linden Fault System, Western New York and Lake Ontario, New York State Museum, 1978.

2. New York State Electric and Gas Corporation, Preliminary Safety Analysis Report New Haven Nuclear Site, Appendix 2.5, 1979.

3. M. L. Sbar and L. R. Sykes, "Contemporary Compressive Stress and Seismicity in Eastern North America: An Example of Intra-plate Tectonics," Geological Society of America Bulletin, Vol. 84, pp. 1861-1882, 1973.

4. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic II-4.F, Settlement of Foundations and Buried Equipment, dated June 30, 1981.

5. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic II-4.D, Stability of Slopes and SEP II-4.F, Settlement of Foundations and Buried Equipment, dated January 15, 1982.

ADDITIONAL REFERENCES FOR SECTION 2.5

1. Dames & Moore, Nine Mile Point Nuclear Station, Unit 2 Geologic Investigations, for Niagara Mohawk Power Corporation, 1978.

2. Dames & Moore, Geologic and Geophysical Investigations Ginna Site, Ontario, New York, for Rochester Gas and Electric Corporation, 1974.

3. Dames & Moore, Site Evaluation Study, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation, 1965.

4. Letter from J. C. Tilson, Environmental Science Services Administration, to H. L. Price, AEC, Subject: 1966 Report on the Seismicity of the Rochester, New York Area, dated February 16, 1966.

5. N. M. Fenneman, Physiography of Eastern United States, McGraw-Hill Book Co., New York, 1938.

6. R. F. Flint, Glacial and Quaternary Geology, John Wiley and Sons, Inc., New York, 1971.

7. Power Authority State of New York, James A. FitzPatrick Nuclear Power Plant Final Safety Analysis Report, 1972.

8. Stone and Webster, Report of Fault Investigations at FitzPatrick Nuclear Power Plant, for Power Authority of the State of New York, 1978.

9. Letter from the Acting Director, U.S. Geological Survey, to H. L. Price, AEC, Subject: Geology and Hydrology of the Proposed

10. Brookwood Nuclear Station No. 1 Site, Wayne County, New York, dated February 28, 1966.

Table 2.5-1
EARTHQUAKE ACTIVITY NEAR ATTICA, NEW YORK

Year	Date	Time	Maximum Intensity
1965	July 16	06:00	IV
1965	August 27	20:57	IV
1966	January 1	08:23	V-VI
1967	July 13	14:08	IV-V

Table 2.5-2

MATERIAL PROPERTIES USED IN THE NRC STAFF ANALYSIS OF SLOPE STABILITY

Soil Layer Number	Soil Type	Thickness Below Top of Slope (ft)	Total Unit Weight (pcf)	Cohesion (psf)	Angle of Interal Friction (degrees)
1	Reddish-brown clay silt	12	107	130	20
2	Brownish-clay silty clay	24	108	120 - 250	0
3	Red fine sand and gravel	8	130	0	38
4	Bedrock (siltstone)	NA	NA	NA	NA

NOTE:—Ground-water level was assumed at elevation 245 ft above sea level (10 ft below the top of the slopes). The earthquake load used in the analysis is equal to the safe shutdown earthquake, 0.2g, for Ginna Station.

APPENDIX 2A

PROBABLE MAXIMUM FLOOD AND LOW WATER CONDITIONS

R. E. Ginna Plant

Reports by R.O. Eaton

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RICHARO 0. EATON. P. E.
CONSULTING ENGINEER

August 13, 1968

Subject: Robert Emmett Ginna
Nuclea Power Plant
Rochester, New York
Wave Runup Analysis

Mr.Wm. W, Lowe
Pickard, Lowe and Associates
Suite 1104, 1730 - M. St., N.W.
Washington, D.C. 20036

Dear Mr, Lowe:

Pursuant to your request I have conducted a study of wave runup under Probable Maximum Hurricane conditions (as established in my prior report, March 28, 1968).

The enclosed report by my Associate, Mr. T.E. Haeussner, presents this study in detail. I have carefully checked Mr. Haeussner's analysis and I concur in his conclusions, i.e., that there will be essentially no wave runup on the vertical plant wall. The rubble mound breakwall adequately intercepts nearly all of the wave energy and the small amount of overtopping will be almost entirely attenuated in the canal between the rubble structure and the plant wall. During winter months ice accumulation along the breakwall will probably entirely eliminate overtopping.

In summary, it is my considered opinion that there will be no hazard to plant operation due to wave runup.

Sincerely yours,

Richard O. Eaton

ROE:w
Encl. Haeussner Report

2A.1 ESTIMATE OF WAVE RUNUP ON VERTICAL PLANT WALL ROBERT EMMETT GINNA NUCLEAR POWER PLANT ROCHESTER, NEW YORK

2A.1.1 GENERAL

Accurate determination of the magnitude of wave runup to be expected on the vertical plant wall during an occurrence of the maximum probable Tropical Storm requires consideration of the following factors:

a. Maximum water level at the plant site.

b. Design wave conditions (height, period, etc.).

c. Near-shore topography.

d. Site conditions, i.e., ground elevations, structural measures and detail.

Waves approaching the plant site will be affected by water depths in the vicinity of the plant site, will break and runup on existing (or proposed) shoreline structures, and will overtop and move forward toward the plant wall. An evaluation was made, as described below, of each of the above factors, insofar as it will affect the mechanics of the height of wave runup on the vertical plant wall.

2A.1.2 DISCUSSION OF FACTORS

The Maximum Water Level to be expected in Lake Ontario at the plant site is 250.78 ft MSL, as indicated in *Reference 1*. That level would result from an occurrence of the probable maximum Tropical Storm over Lake Ontario. This factor together with near-shore topography, determine the breaking depth of water fronting the plant site. The various components comprising that elevation are outlined in that reference report.

Design Wave Conditions. The significant wave height and period resulting from an occurrence of the design storm as determined in *Reference 1*, would be 19 ft and 9.7 seconds, respectively. That wave, and its characteristics, would be affected by near-shore depth conditions as it approached the plant site.

Near-Shore Topography. An offshore bottom profile extending northward in Lake Ontario from the plant site is shown on Enclosure 1. Data were obtained from U. S. Lake Survey Map No. 23, dated 1962. The peak water level at shore is also shown on that sketch together with a pertinent portion of the setup water surface profile extending lakeward.

Site Conditions were obtained from Construction Print, Drawing No. 33013-171 c, printed June 12, 1968, R. G. & E. Eng. Dept. The plant is fronted by an armor stone breakwall with an approximate 1 on 1 slope from lake bottom to elevation 254 ft MSL. Concrete paving (@ elev. 253 ft MSL) will extend shoreward from the breakwall a variable distance (20-25 ft) to the discharge channel. That channel has a 1 on 1 sideslope with 30 ft bottom width at elevation 238 ft MSL. A concrete overhang deck (@ elev. 253.5 ft MSL) extends from the south channel wall 100 ft to the vertical plant wall. Waves overtopping the armor stone breakwall

will move across the paved area to the discharge channel and, conditions permitting, across the overhang deck to the plant wall.

2A.1.3 ANALYSIS

Standard procedures, described in *Reference 2*, were used to evaluate the effective water height, runup, and overtopping relationships involved in this problem. The breaking depth (d_b) affecting the design wave was taken from Enclosure 1 at a 100-ft distance lakeward of the armor stone breakwall, and is estimated to be 9.4 ft (250.8 ft-241.4 ft). The breaking wave height (H_b) is equal to 0.78 of that depth, or 7.3 ft. According to Equation 1-37 of *Reference 2* the equivalent deep water wave height

$H'_o = (1.837/T)\,(d_b)^{3/2} = (1.837/\,9.7)\times(9.4)^{3/2} = \underline{5.3\text{ ft}}$

Parametric relationships relating to wave runup on rubble-mound slopes to the wave height period ratio are given on Figure 3-12 of *Reference 2*. The latter ratio

$$\frac{H_0'}{T^2} = \frac{5.3}{94} = \underline{0.563}$$

(Equation 1)

The slope of the armor stone breakwall is approximately 1 on 1, requiring interpolation of the runup curves on Figure 3-12. Entering that figure with the ratio 0.563, an R/H'_o value of 0.63 was obtained, giving a runup value of R = 3.34 ft. Adding that value to the peak wind tide elevation results in a wave runup elevation of 254.12 ft.

2A.1.4 DISCUSSION

The elevation of the armor stone breakwall fronting the plant site is 254.0 ft. Based on the computed wave runup the depth of overflow over the breakwall and onto the paved area fronting the discharge channel would be 0.12 ft per wave. That overflow would be blown into the discharge channel; as the water level in the channel exceeds the peak tide level outflow will occur to the lake. The water surface elevation in the discharge channel is therefore not expected to exceed the peak tide elevation by about 1 ft. Since the width of the discharge channel fronting the plant site is less than 30 ft width there will be no possibility of wave generation across that channel. Any wave action entering the channel from the lake will be dampened somewhat by friction and will run up the side slope of the channel and pond in the area east of the plant site.

2A.1.5 CONCLUSIONS

Based on the above analysis the following conclusions are drawn:

1. There will be no wave runup against the vertical plant wall during an occurrence of the design Tropical Storm.

2. Wave action entering the discharge channel will runup the channel side slope east of the plant and pond in the area indicated on Construction Print Drawing No. 33013-69 as the site for "Future Screen House", but will not affect the vertical plant wall.

Submitted by,
/s/ Theodore E. Haeussner
Hydraulic Engineer, Consultant
Jacksonville, Florida
August 3, 1968

Enclosures

1. Lake Ontario Bottom Profile
2. References.

REFERENCES

1. Haeussner, Theodore E., "Maximum Probable Water Levels for Robert Emmett Ginna Nuclear Power Plant, Lake Ontario". March 26, 1968.

2. U. S. Army Coastal Engineering Research Center, "Shore Protection, Planning and Design", Technical Report No. 4, Third Edition 1966. Department of the Army, Corps of Engineers.

2A.2 MAXIMUM PROBABLE WATER LEVELS IN LAKE ONTARIO AT THE ROBERT EMMETT GINNA NUCLEAR POWER PLANT SITE

Report dated
March 28, 1968

2A.2.1 INTRODUCTION

OBJECTIVE The basic objective of this report is to establish the "probable maximum" and "minimum" water levels to be expected in Lake Ontario at the Robert Emmett Ginna Nuclear Power Plant site near Rochester, New York; those levels to be based upon a set of conditions whose individual and collective occurrence frequency are sufficiently rare so as to provide a very high degree of plant safety.

PROBLEM The combination of conditions ultimately selected requires a detailed evaluation of the various hydrological and meteorological factors and events which can reasonably be expected to occur in the area. Hydrologically it involves the normal seasonal regulation criteria and levels prescribed for Lake Ontario, with due cognizance of unusual events or circumstances which could affect those levels and/or operating criteria. Meteorologically it involves consideration of both extratropical and tropical cyclonic storm occurrence, their paths, intensity, and frequency with respect to Lake Ontario, as well as their overall effect in terms of accompanying rainfall, winds, seiches or tides generated, pressure effect, and associated wave action. The element of time is also involved as it relates to lake stage, i.e., the most likely time of the year when deep intense extratropical cyclones occur as compared with the seasonal limitation on the occurrences of tropical storms, or hurricanes. Each of these meteorological factors and elements are examined below the evaluated in terms of their probable occurrence and effects in relation to the hydrologic conditions involved.

2A.3 ANALYSES OF ALTERNATIVE HYDROLOGIC AND METEOROLOGIC CRITERIA

A. LAKE ONTARIO REGULATION

GENERAL Water level records have been maintained for Lake Ontario since 1860 providing some 108 years of record. During that period the average lake stage has been about 246 feet above MSL (USC&GS 1935 Datum). Extreme ranges in monthly average stage have been from a low of about 242+ feet in 1934 to a high of about 249+ feet in 1952. The normal annual variation in lake levels is seasonal, ranging from low levels in November February (as a result of freezing temperature and more solid forms of precipitation), rising to maximums in May and June from Spring snowmelt and rainfall. Local short period extremes in stage have been observed around the lake perimeter and have resulted from wind action creating seiches at the extreme east and west ends of the lake, with minimal or negligible effect in the central north and south shore areas of the lake, including the plant site area.

REGULATION of Lake Ontario water levels is under the International St. Lawrence River Board of Control with supervision and direction from the International Joint Commission of the United States and Canada. Operation and regulation criteria have been developed by the Board and its staff and are contained in *References 1* and *2*. The initial regulation plan, 1958-A, was placed in effect April 20, 1960. Subsequent to that date several other modified plans have been initiated; the present plan 1958-D, was adopted July 1963. That plan has two sets of basic rule curves for discharge utilizing a basic "storage equation" and supply indicators for adjusting outflows from the lake. Seasonal adjustments to the outflow curves permit storage of water in winter, spring and early summer and the opposite in the late summer and fall, resulting in a high operating efficiency for maximum benefits to all water users. Approximately 85 percent of the annual inflow to Lake Ontario comes from the upper Great Lakes with the remaining 15 percent from local drainage. Thus the basic water supply to the lake changes very slowly permitting reasonably accurate forecasts and operating actions to maintain desired levels. Because of this only minor concern is given to "short term" supply changes, such as ice jams on the Niagara River or local winter floods.

During late winter and early spring these exceptions to the normal inflow and supply are not considered critical because of the large storage volume available in the lake. The storage increment per foot of stage is about 4.8 million acre feet. The discharge requirement to lower the lake one foot at relatively high stages is 348,000 c.f.s. for one week. The lake is to be regulated seasonally over a 5-ft range in elevation, between 243 ft and 248 ft. Lake regulation stages follow the "normal" high in summer low in winter levels. Period-of-record monthly routings (1860-1954) were made by the Board of Control to test the effectiveness of the present plan (1958-D) in maintaining desired levels and flows over a wide range of conditions to insure meeting all of the established criteria. Monthly mean adjusted stages and those resulting from application of Plan 1958-D are contained on Plates 6-1 through 6-10 of *Reference 1* for the period 1860-1954. Stage duration curves, based on those routing results can be found on Plates 12 through 23 for each month of the year. The 1 percent stage (percent of time monthly average stage is equalled or exceeded) taken from those curves is tabulated below for each month.

Month	1 Percent Stage (ft MSL)
January	246.45
February	246.75
March	246.90
April	247.60
May	248.00
June	248.00
July	248.05
August	247.70
September	247.10
October	246.55
November	246.25
December	246.05

Those stages are plotted graphically on Exhibit 3 of this report. Also plotted on that graph are the monthly mean stages for the period-of-record 1860-1966, and the monthly maximums and minimums of record with year of occurrence noted. Although the regulated 1 percent occurrence stage graph provides a limiting 2-ft range, as does the mean monthly record stage graph, there is an evident shift in peak month from June to July. This is believed due to the routing procedures and the rule curves employed in regulating periods of unusually heavy runoff.

REGULATION EXCEPTIONS As noted above, the week-to-week changes in inflow to Lake Ontario are highly predictable and can be compensated for by adjustments in outflow criteria. Because of the large storage volume per foot on the lake proper, day-to-day fluctuations in overall lake stage and storage (excluding wind effects along shore) must be related primarily to direct rainfall on the 7,500 square mile lake area and, to a lesser extent, from resultant local runoff. The occurrence of ice jams in the International Rapids Section is rather remote and limited to late winter or early spring months. Their effect in reducing outflow and overall resultant effect on lake stage would be small however. For example, if an ice jam occurred reducing normal winter outflow from the lake 50 percent, i.e., say from 300,000 c.f.s. to 150,000 c.f.s., the cumulative effect on lake stage per day would be 150,000 x 2 = 300,000 acre feet + inflow. If inflow = required outflow (i.e., 300,000 c.f.s.) the total effect would be 900,000 acre feet per day accumulated storage, or about 0.2 ft per day increase in overall lake stage. It is assumed some action would be taken by the Commission to eliminate such a condition before the cumulative effect became critical and endangered property around the lake. By far and large, the occurrence of unusually heavy and widespread rainfall on the lake proper is much more significant as to sudden short-period rises in both stage and storage. That parameter is evaluated in the following paragraph.

B. RAINFALL

GENERAL CLIMATOLOGY The occurrence of heavy widespread rainfall over much or all of the 7,500 square mile surface area of Lake Ontario is a significant factor in short-term rises in lake stage. Heavy concentrated rains, of the type which could raise lake levels half-a-foot or more within a matter of 24 to 48 hours, are associated with large-scale cyclonicity over central and northern latitudes and with hurricanes moving inland and overland from the Gulf of Mexico and the Atlantic Ocean. Various areas of the United States have experienced intense widespread rains from both sources; e.g., in the Hallett, Oklahoma storm of September 4, 1940 more than 6 inches of rain fell on 8,600 square miles in 11 hours (*Reference 3*); in hurricane Diane of August 17-20, 1955 nearly 11 inches of rain fell on 10,000 square miles in portions of New York, Vermont, Massachusetts and New Hampshire within 48 hours (*Reference 4*). Similarly, in the tropical storm of August 19, 1939 the city of Manahawkin, New Jersey recorded 17.8 inches of rainfall in 15 hours, with about 6 inches occurring over a 7,500 square mile area. In *Reference 5*, Figure 9, it was shown that the prime source of moisture for 140 winter-spring cases of heavy precipitation over central and northern U.S. was northward flow aloft from the Gulf of Mexico into a system of cold and warm fronts. Heavy rainfall of this type is usually associated with deep cyclonic lows having such attendant cold and warm fronts accompanied by overrunning and often occlusion. In *Reference 3* the total volume of precipitation in such type events was found to be a large fraction of the volume of atmospheric moisture flowing into the converging cyclonic area (from 50 up to 100 percent). In contrast to this type of storm-rainfall condition, hurricane rainfall such as that noted along the eastern seaboard in hurricane Diane, is primarily the result of orographic lifting of moist air, brought inland from the ocean by cyclonic circulation, over the coastal mountain ranges. As such, intense widespread rains of the type experienced in hurricane Diane are essentially limited to about a 100-150 mile distance inland from the Atlantic coast in the New York-New England area. The mechanism responsible for this limitation and a fairly reliable basis for predicting hurricane rainfall, intensity, and distribution, can be found in *Reference 3*.

LAKE ONTARIO RAINFALL The average monthly rainfall for the lake, based on an analysis of records contained in *Reference 7* for Rochester, New York and other stations on or near the lake does not vary widely - averages about 2 to 3 inches, with an average annual rainfall total of about 32 inches. Variations in annual totals are about ± 6 inches of that value. Extremes in monthly totals vary from 0.2 inch to around 6 inch. The extreme monthly rain-fall noted in *Reference 7* for Rochester, New York is 9.70 inches. No reference is given as to month or year of occurrence. In general the highest monthly amounts of rain occur in August; however, in terms of snowfall and equivalent water content, amounts of 3 to 4 inches can be found to occur in the months October to February. Analysis of maximum 24 hour rainfalls for Rochester, Buffalo, and Syracuse, from *Reference 7*, for the period 1921-1960 indicates values of 1.19 inches, 4.28 inches, and 4.79 inches, respectively, for those stations. In *Reference 6* the month of highest seasonal probability of occurrence of intense 24-hour rainfall with a return period in excess of 100 years is given as September for the Lake Ontario area.

WINDS The prevailing wind direction at Rochester, New York, from *Reference 7*, is west southwest. Analysis of wind directions associated with the "fastest mile" of wind at that station for the period 1956-1968 indicates that in only 20 months of the 144 months checked, or 14 percent, was the direction from a NE to NW quadrant. The remainder of the time the

fastest mile of wind during the month was predominantly from either the west or southwest. The fastest mile of wind recorded at Rochester, New York in the period 1931-1968 was 73 miles per hour. The months of highest winds from the west-southwest appear to be from January-June with values averaging from 55-60 mph.

C. EXTRATROPICAL CYCLONES

Numerous studies have been made relating to extratropical cyclones (termed Northeasters when over the ocean), their origin, paths, frequency, intensity, general monthly distribution, and effects as they relate to the Great Lakes (*References 8*, *9*, *10*, and *11*). Possibly the earliest study of this kind by Garriott, published in 1903, *Reference 12*, lists 238 cases covering a 25-year period. A more recent study by Irish and Platzman (*Reference 13*) investigated 76 such storms in the 20-year period 1940-1960 which caused setup in excess of 6 feet on Lake Erie. Those storms occurred from September through April; the maximum number - 26, occurred in November, 16 occurred in January, 13 in December, and 10 in March. The second highest observed set-up on Lake Erie was caused by a severe March storm, the highest by a January storm. The former storm, that of March 22, 1955 was an intense cyclone that began in east Texas and deepened rapidly as it moved up the Mississippi Valley. Near Lake Michigan it deepened to 975 mb. as it occluded. It then moved NE across northern Lake Ontario. Gusts up to 74 mph were reported. Perhaps the most all inclusive study of extratropical storms is contained in *Reference 14*. Nearly all of the destructive storms studied occurred in the months of November through April. Of 160 incidents of gale force winds recorded at Boston in the 75-year period 1870-1945, half were classified as northeast gales. Some 51 of these cyclonic storms studied in that report consisted of a single low pressure cell moving eastward from the central and upper United States across the New England area. The low pressure cell was usually associated with only one cold front and one warm front, although multiple lows and fronts were observed in many other storms. Exhibits 1 and 2 from *Reference 14* show such simple and complex pressure systems for April 2, 1958 and March 1, 1914, comprising of 972 mb. cell and two low centers of 960 mb. and 956 mb., respectively. The latter was the lowest pressure (28.25 inches) ever recorded at New Haven, Connecticut from such extratropical storms. Other notable cyclones listed in that report are those of December 2, 1942, 959 mb.; March 4, 1931, 961 mb.; and March 4, 1960, 961 mb. The average low pressure for the 51 storms was 983 mb. Observed wind speeds of 65-75 miles an hour are not unusual in these deep mature lows. In the March 1914 cyclone, wind speeds approached 80 m.p.h. Maximum winds in these storms are a function of pressure gradient; for the 51 storms the gradients ranged from 11 mb. per 150 naut. miles up to 25 mb. per 150 naut. miles (maximum). The origin of 73 percent of the 51 storms was found to be primarily the Texas-East Gulf and South Atlantic regions. Maximum cyclongenesis takes place in these source regions during the colder months because of the marked temperature contrast between maritime and continental air masses along the southern coasts. The forward speed of the 51 storms averaged 22 knots over the 12 hour period of their path prior to approaching the coast or passing into the Atlantic Ocean. Final deepening of those storms during that 12 hour period ranged from 6 mb. for storms moving in an eastward direction up to 11 mb. for storms moving more northward.

D. TROPICAL CYCLONES

The three principal areas of tropical cyclone, or hurricane, formation are the Gulf of Mexico, the Caribbean Sea and the north Atlantic Ocean. Literally hundreds of these storms have formed in these areas and, affected by largescale meteorological factors and sea surface temperatures, have moved on a wide variety of paths. Those paths have been chronicled by numerous authors, viz. *References 15* and *16*, as well as by the U.S. Weather Bureau in Monthly Weather Review and Climatological Data publications. Various authors have attempted to correlate the paths of tropical storms, areas of formation, and month of the year of occurrence. A study of the seasonal variation in the frequency of topical cyclones for various geographic areas along the eastern U.S. coast, in terms of the effect of general atmospheric circulation on storm path, was presented by Ballenzweig in *Reference 17*. Ballenzweig concluded that varying seasonal circulation patterns form the framework for steering tropical cyclones after their generation; and that recurrent positive and negative anamolies of the 700 mb. height in terms of departure from normal for the hurricane months could form the basis for predicting hurricane movement.

CYCLONES AFFECTING LAKE ONTARIO A study was made by the author of hurricane paths since 1888 using *References 15*, *16*, *18*, and U.S.W.B. Climatological Data publications to determine the relative occurrence of hurricanes and tropical disturbances moving inland, overland, and passing over or near Lake Ontario from their various areas of formation. Of especial interest were storms moving northward along the Atlantic seaboard whose movement was blocked and which ultimately recurved westward and/or northward toward or over Lake Ontario. In the 36 year period covered in *Reference 15*, 1888-1924, some 21 hurricanes and tropical depressions passed over or near Lake Ontario-17 from the Gulf of Mexico and 4 from the Atlantic Ocean. Of the latter, three passed directly over the lake in 1893, 1903, and 1923. Storm paths shown in *Reference 16* for the period 1924-1937 indicate 4 storms fell into that category. Since 1937, *Reference 18*, some 5 tropical storms have passed over or near the lake. All totalled, about 30 hurricanes and tropical disturbances in the last 80 years have affected Lake Ontario to some degree. Those storms occurred in the months of June through November, with a prevalence of occurrence in July, August and September. In the 31-year period, 1903-1933, four major Atlantic hurricanes recurved inland along the coasts of Maryland, Delaware, and New Jersey passing over Lake Ontario. Those storms had the shortest overland trajectory, some 200-300 miles, from the ocean to the lake. This fact is of prime importance in regard to the filling and deintensification process that occurs within the storm system in its overland trajectory.

2A.4 DESIGN STORM ANALYSIS FOR MAXIMUM PROBABLE WATER LEVEL

GENERAL As indicated earlier in this report the problem of evaluating the maximum probable water level to be expected at the plant site involves selection of a design storm, its time of occurrence in coincidence with lake level, and the cumulative effects of that storm in terms of wind setup at shore, pressure effect, antecedent or associated rainfall, and wave effect.

From the data presented and discussed above two types of cyclonic storms affect the area - extratropical cyclones and tropical cyclones. Each has its own set of characteristics, probable time of maximum occurrence and intensity, and its resulting effect on Lake Ontario in terms of maximum water elevation at the site area. Because of the basic differences associated with these two storm types, two separate analyses were made to determine the most critical combination of conditions for each and resultant maximum probable water levels. Those analyses are presented below.

2A.4.1 EXTRATOPICAL STORM ANALYSIS

CRITERIA The combination of conditions selected to represent an occurrence of this event is as follows:

a. Central Pressure. The minimum central pressure was based on the lowest storm pressure observed at New Haven, Conn. modified by the mean rate of deepening for storms moving eastward across the upper United States for the 12 hr period prior to reaching the coast. Po = 957 mb. + 9 mb. = 966 mb. (28.53 inches)

b. Path. The storm center would move eastward just south of Lake Ontario so that winds in the western part of the storm would be from the north, moving progressively from NNE-N-NNW over a period of about 6 hours.

c. Forward Speed. The storm would move at a rate of about 20 mph, an average speed for storms of this nature.

d. Wind Speeds. Average wind speeds over the lake for a North-South fetch to the plant site would be about 60 mph, based on a pressure gradient of 25 mb. per 150 naut. miles.

e. Lake Stage. The storm was assumed to occur in April; the 1 percent frequency stage from Exhibit 1 or 247.60 ft msl. was used as prestorm lake stage.

f. Antecedent rainfall of 4.2 inches (0.35 ft.), associated with frontal passage during the 24 hours preceding maximum wind setup was assumed to occur as an average value over the lake.

RETURN FREQUENCY The relative frequencies of the various criteria, in combination, represent a rate event, on the order of a once in 10,000 year occurrence. The return frequency of the selected storm is in excess of a 50-year event, the selected lake stage has a 1 percent return period; and the lakewide average rainfall of 4.2 inches is believed to be on the order of a 25-30 year event for the area.

PEAK WATER LEVEL Wind setup computations were made for a lake stage of 247.95 ft MSL (247.60 ft + 0.35 ft rainfall). Lake bottom elevations were averaged for 3 fetches, 22.5 degrees east and west of north, and a N-S fetch. An average wind speed of 60 mph was used

over the fetch and setup computations made beginning at a node line approximately 20 miles north of the plant site area. (The selection of that location was based on trail computations. Since the lake is extremely deep, over 500 ft for almost half the fetch, it was found that the difference in final wind setup would be on the order of .01 - .02 ft for a shift of a mile or more in either direction in node point location.) The peak computed wind setup was 0.45 ft at the plant site. Pressure effect on the lake was determined using a 4 mb. departure from storm center to the area of interest. The variation from normal pressure, converted to feet of additional rise in lake level at the site area, was determined as follows:

[977 mb. - (966 mb. + 4 mb.)/ 33.8 (conversion to inches)] x(1.14) = 0.91 ft

Wave effect was considered to add an additional foot of rise in water level at shore. Deepwater wave forecasting procedures, using V_{av} = 60 mph, Fetch = 45 miles, gave a significant wave height H_s = 16 ft, a wave period T_s = 9 seconds, for a required duration of 3+ hours.
The breaking depth for a 16 ft wave is about 20.5 ft. That wave would break about one-half mile from shore; successive wave trains would add to the depth of water near shore. The 1-ft value is believed to be reasonable.

SUMMARY The total maximum probable water level at the plant site from the design extratropical storm and associated phenomena would be:

Lake Stage	247.60 ft MSL
Rainfall	0.35 ft
Wind Setup	0.45 ft
Pressure effect	0.91 ft
Wave Effect	1.00 ft
Max. Water Level	**250.31 ft MSL**

2A.4.2 TROPICAL STORM ANALYSIS

CRITERIA The combination of conditions selected to represent an occurrence of this event is as follows:

a. Central Pressure. A maximum probable hurricane, derived by the author for the Barnegat Bay, New Jersey area, is considered applicable for transportation to the Lake Ontario area. The central pressure of that storm at the coast is 917.7 mb. (27.10 inches). Filling of that storm in its path from the coast to the lake would change its central pressure (+20 mb.) based on a study of filling in hurricanes, *Reference 19*. The central pressure at Oswego, New York would be 938 mb. (27.7 inches).

b. Path. The path of the storm was assumed to be similar to those of the major hurricanes of 1903, 1923, 1928, and 1933, all of which entered the east coast along the Maryland-New Jersey shoreline, curving northward and over or near Lake Ontario. The storm center was

assumed to pass close to Oswego, New York in order to obtain winds from the north over the lake.

c. Forward Speed. The forward speed of the hurricane would average about 25 mph in its overland trajectory to the lake.

d. Wind Speeds. Maximum wind speeds in the eastern semi-circle of the hurricane would be reduced from 120 mph at the open coast to about 105 mph at the lake. Winds in the western portion of the storm would be reduced from 90 mph to about 75 mph. An average wind speed of 70 mph was used on the lake over the fetch in computing setup at the plant site.

e. Lake Stage. The hurricane was assumed to occur in July; the 1 percent frequency stage from Exhibit 1 of 248.05 ft MSL was used as pre-storm lake stage.

f. Antecedent Rainfall. Analysis of past record hurricanes entering the east-coastal area (*Reference 3*) indicates that extreme convergence plus the orographic effect of coastal mountain ranges will precipitate a high percentage of moisture in the storm within the first hundred miles of its inland movement. Consequently, associated rainfall in the design hurricane over Lake Ontario was assumed to be nominal and estimated to average 2+ inches 0.17 ft) over the lake at the time of peak wind setup.

PEAK WATER LEVEL Wind setup computations were made for a lake stage of 248.22 ft MSL (248.05 ft + 0.17 ft rainfall). The same bottom elevations and fetch conditions noted above in the extratropical storm analysis were used. Using a wind speed of 70 mph over the average fetch a peak setup of 0.53 ft was computed at the plant site. Pressure effect was determined in the same manner as for the extratropical storm, using about a 30 mb. change in pressure in the 55 mile distance between Oswego and the fetch area. Pressure effect was computed to be 1.03 ft. Wave effect was considered to add an additional foot of rise in water level at shore. The significant wave height, H_S = 19 ft for a 70 mph average wind speed; T_S = 9.7 seconds, for a required duration of 3+ hours. The breaking depth for a 19 foot wave is about 24+ ft. That wave would break about 3,000 ft offshore; successive wave trains would add to the depth of water near shore.

SUMMARY The total maximum probable water level at the plant site from the design tropical storm and associated phenomena would be:

Lake Stage	243.05 ft MSL
Rainfall	0.17 ft
Wind Setup	0.53 ft
Pressure effect	1.03 ft
Wave effect	1.00 ft
Max. Water Level	250.78 ft MSL

2A.4.3 EXTREME LOW WATER LEVEL

GENERAL Several factors affect and, to a large extent, control the value of extreme low tide elevation to be expected at the Robert Emmett Ginna Plant site. They are essentially as follows:

1. Hurricane wind speed and direction in storms passing west of the plant, so as to have the zone of maximum winds directed offshore to the lake.
2. Offshore depths, both nearshore and with respect to the overall depth of the lake.
3. The general orientation of the bay axis with respect to hurricane wind direction.
4. Initial water level of the lake prior to storm occurrence.

For the project plant site area tide generating conditions on a north-south oriented fetch are maximum in comparison with the east-west tide generating potential. The site is located at or near the nodal point for the latter condition and would experience negligible setup or setdown for east-west oriented winds. As estimate of the maximum anticipated setdown to be expected at the plant site was based on an assumed occurrence of the Maximum Probable Hurricane transposed to the lake on a path from the south with the center passing some 40± miles west of the plant site. Peak hourly average winds from the south-southeast, blowing offshore, would be on the order of 90-95 mph during passage of the storm across the lake. The assumed lake level at the time of hurricane passage would be the lowest future lake level under the International Commission regulatory plan 1958D - 243.07 ft MSL. It is probable that this stage would occur as a result of a prolonged drought, extending over a period of a year or more, so that the low stage could occur during mid-summer and the hurricane season. Wind tide computations were made with that lake stage and the above wind and fetch criteria. The maximum setdown elevation at the plant site was determined to be 0.83 ft. This would result in an Extreme Low Water Elevation of 242.23 ft MSL (243.07 - 0.83 ft).

CONCLUSIONS

Based on the above analyses the undersigned has drawn the following conclusions:

1. That Lake Ontario is subject to the repeated occurrence of both extratropical and tropical cyclonic storms and their effects.
2. That hydrologic analyses of regulatory criteria established for the lake provide a sound and highly reliable basis for predicting the probable range in future stages.
3. That available meteorological analyses for both type storms are sufficiently detailed and accurate to permit derivation of events of rare frequency and their transportation to the lake area.
4. That the critical combination of assumed meteorological and hydrological conditions for a design tropical storm would result in a slightly higher maximum probable water level on the lake than would occur from a design extratropical storm.
5. That the Maximum Probable Water Level to be expected at the Robert Emmett Ginna Nuclear Plant Site is 250.78 ft MSL.
6. That the Extreme Low Water Level to be expected at the plant site is 242.23 ft MSL.

Submitted by,
/s/ Theodore E. Haeussner
Hydraulic Engineer, Consultant
Jacksonville, Florida
March 26, 1968

EXHIBITS

1. A typical single-cell single-front extratropical storm on April 2, 1958.
2. A complex double-cell double-front extratropical storm on March 1, 1914.
3. Mean monthly observed and regulated normal and extreme water levels for Lake Ontario.

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APPENDIX 2B

DRIFT AND DISPERSION CHARACTERISTICS OF LAKE ONTARIO NEARSHORE WATERS ROCHESTER, NEW YORK TO SODUS BAY, NEW YORK

A Specialized Limnological Study Sponsored by Rochester Gas and Electric Corporation Rochester, New York

Conducted by
Pritchard-Carpenter

208 MacAlpine Road
Ellicott City, Maryland

Including

2B Appendix A	OBSERVED TRACER DISTRIBUTIONS (PARTS PER BILLION)
2B Appendix B	WIND SPEED AND DIRECTION OBSERVATIONS METEOROLOGICAL TOWER ON BROOKWOOD SITE ANEMOMETER - ELEVATION 150 FEET
2B Attachment I	THE EFFECT ON LAKE DILUTION OF MOMENT MIXING

2B.1 DRIFT AND DISPERSION CHARACTERISTICS OF LAKE ONTARIO NEARSHORE WATERS ROCHESTER, NEW YORK TO SODUS BAY, NEW YORK

2B.1.1 SUMMARY

Drift and dispersion studies were conducted in Lake Ontario during April-May, July and October, 1965. The study area was along the south shore of the lake between Rochester and Sodus Point. The drift was found to be primarily wind induced, with speeds and directions correlated to wind speed and direction. A steady drift to the east of 0.05 knots was present during calm periods.

Tracer material was released continuously at the Brookwood site during the study periods. The observed distributions of released material were fitted to theoretical equations. The probable distribution of heat and materials released with the condenser cooling water flow under different discharge structure designs were computed from the observed diffusion data. These computations show that the use of a horizontal jet minimizes the thermal effect and produces the most rapid dilution of discharge constituents, so that a jet (approximately 2 ft/sec) discharge should be considered as optimum. With horizontal discharge, significant heating would not be present along the lake shore beyond the site boundary and the area with temperatures elevated by 5° would extend out into the lake approximately 3000 feet and have an average width of 200 feet.

The study showed that a twenty-fold or greater dilution would occur before the discharge reaches the area of the nearest public water intake (town of Ontario). This intake is located on the bottom at a depth of 11 feet. Thirty-fold dilution would be expected before the discharge could be drawn into the plant intake located on the bottom at a depth of 28 feet.

As a result of implementing the Extended Power Uprate (EPU) there will be an increase in the thermal discharge. In accordance with the Federal Clean Water Act (CWA) Section 316(a) and 6NYCRR Part 704 of the New York State Water Quality Standards Constellation Energy assessed the effect of the increased thermal discharge to assure the protection and propagation of a balanced indigenous population of shellfish, fish, and wildlife in and on Lake Ontario. The analyses consisted of three separate studies designed to evaluate the size of the thermal plume under the planned EPU conditions and to assess thermal impacts to indigenous species of fish. The three reports, listed below, were submitted to the NYSDEC on March 8, 2005. Thermal Plume Study (Ocean Surveys, Inc. (OSI)), Near-Field and Far-Field Modeling Studies for the R.E. Ginna Power Plant (HyroQual Environmental Engineers & Scientists (Hydro Qual)), Biological Assessment: Near-Field and Far-Field Modeling (Northern Ecological) Associates, Inc. (NEA), along with a New York Form 2C (Attachments I-VII), Supplemental Form A to Form 2C, a SEQR LEAF, 401 Water Quality Application, and a State Coastal Zone management form.

OSI performed an insitu thermal plume study that was used as validation input for the near-field and far-field modeling studies. The extent of the thermal plume is a product of the velocity of the discharge water and wind influence, and is confined to a narrow stream in the lake. As expected, vertical profile data from the study showed a drop in temperatures as the

plume expands into the lake, limiting the thermal impact to the near-field. In the near-field study area (within 600-700 feet of the plant discharge), the thermal plume mapping survey showed complete vertical mixing from surface to bottom, while the plume was limited to the surface (upper 5 feet) in the far-field study area (10,000 feet north of the discharge and 6,000 feet both east and west of the centerline of the plume).

HydroQual modeled the thermal plume under existing operating conditions, SPDES permit conditions, and EPU conditions to determine the aerial extent of the 3°F isotherm. Modeling simulations were used to assess the plant's compliance against the SPDES permit Additional Requirement Number 5, which limits the allowable mixing zone, as defined above, to an area of no more than 320 acres. Under planned EPU conditions, the modeling results indicate that, during the summer and winter critical periods, the predicted plume sizes under all operating conditions occasionally will exceed the permit limit of 320 acres. More specifically, the modeled thermal plume is predicted to exceed 320 acres by approximately 12 percent approximately 2.5 days over a 30-day period. This results in a modeled plume size of 360 acres.

NEA assessed the thermal tolerance of ten selected Representative Important Species (RIS) under conditions expected in the Ginna Station thermal discharge under planned EPU conditions using actual discharge temperature data and literature values for species' thermal tolerance. Monitoring results from 2000 to 2004 indicate that during the summer months, the monthly average hourly discharge temperature in the discharge canal could potentially reach or exceed the upper thermal tolerance for most of the fish species evaluated. Fish are highly mobile species and would be able to seek ambient ideal waters to avoid impacts. During periods of adverse conditions, residency of any fish species in the discharge canal would be highly unlikely. During summer months, cold and coolwater species would avoid the near-shore waters of the lake area, thus avoiding impact. Warmwater species have a higher probability of being in the nearshore waters during summer months. Although the average temperature in the Ginna Station discharge canal would exceed the lower range of the upper thermal tolerance of warm water species, the majority of all fish species seek cooler waters for shelter, thus minimal impacts are expected.

In summary, the thermal plume currently affects only a small region of the southern shoreline of the Lake and the planned Extended Power Uprate (EPU) could only occasionally (i.e., during extreme summer conditions) result in a small increase in the area of Lake Ontario impacted by thermal discharges from Ginna Station.

2B.1.2 INTRODUCTION

The Rochester Gas and Electric Corporation is undertaking the installation and operation of a nuclear electric power station at the Brookwood site located about 18 miles east of Rochester. As a part of the preliminary environmental analysis, calculations of the distribution of materials released to Lake Ontario in the Brookwood region were undertaken. These calculations were based on experience in other bodies of water and the general characteristics of Lake Ontario.

During 1965, an observational program in Lake Ontario was undertaken. The purpose of the program was to obtain direct information on the drift and dispersion characteristics of the Lake waters, and to use these characteristics to predict the distribution of materials and heat

released at the site. The field study has provided a basis for estimates in which considerably greater confidence can be placed than was the case for the estimates made in the preliminary analysis. However, it is noted that the estimates are in general agreement with the preliminary analysis. In addition, the observational program was desirable to furnish evidence that no unusual (unexpected) features are present in the drift and dispersive characteristics of the area. This report summarizes the field observations made during the program and translates the data into forms useful in making predictions of the concentrations to be expected at various positions along the lake shore.

2B.1.3 GENERAL COMMENT ON EFFECT OF DISCHARGES ON LAKE ONTARIO

Lake Ontario is approximately 190 miles long and 60 miles wide and has a surface area of about 7,500 square miles.

Depths of 40 to 100 feet are found within one to two miles of shore and the maximum depth is 778 feet. The mean sectional depth is roughly 300 feet, so that the volume is approximately 6 x 10^{13} cubic feet.

The mean total flow through Lake Ontario corresponds to the discharge through the St. Lawrence River of 241,000 cfs, of which 85% is contributed by the Niagara River flowing from Lake Erie. The volume of Lake Ontario, therefore, represents discharge at the mean rate for

2.5 x 1.0^8 seconds or 8.2 years. Changes in the bulk composition of Lake Ontario as a result of alteration in Lake Erie would be expected to take place with a time scale of ten to twenty years.

The ultimate concentration of materials discharged into Lake Ontario may be estimated from the volume of "new" water available. For a nuclear power plant discharging condenser water at 600-700 cfs, constituents of this effluent would be diluted 350 fold when mixed into the total "new" water. Even with partial mixing throughout only one-third of the Lake width, dilutions of 1:100 would be expected. In view of the flow-through time scale of eight years, it seems certain that mixing over much of the Lake volume will occur, since horizontal motions transport water from one end of the Lake to the other in a few months and complete vertical mixing takes place annually.

2B.1.4 CURRENTS IN LAKE ONTARIO

The predominant surface currents in Lake Ontario would be expected to move from west to east, since the predominant wind direction is from west to east and wind stress on the water surface appears to be the strongest current generating force. The currents associated with inflow from the Niagara and other rivers and outflow by the St. Lawrence are not strong. Even if it is assumed that this flow-through is intermittently confined to an upper 30 foot layer (the summer mixed layer), speeds of 0.04 feet per second would result.

Currents in the Brookwood region were measured in several ways during the course of the observational program. Continuous measurements at a position 800 feet offshore of the Brookwood site were obtained during May 1965. The current meter was suspended from a frame which rested on the Lake bottom, so that the meter was six feet below the water surface. The current meter was a direction resolving, time integrating device built by W. H.

Johnstone Laboratories of Baltimore, Maryland. Speed was sensed by the tilt of the suspended instrument case, which contained two compass cards with collimated beams of radioactivity. The radioactivity was detected with two ionization chambers that were shielded with absorbers that were machined to the function relating tilt angle to current speed (i.e., square foot of the tangent of the angle). The count rates were directly proportional to the North- South and East-West current vectors. The signals from the two ionization chambers were recorded on a two channel digital, integrating printer. The integrated currents were recorded each thirty minutes.

The results of these current measurements are shown in Figure 1. The East-West component of the wind is also shown in Figure 1. The wind speeds are taken from the hourly readings of the 50 foot anemometer at the Brookwood site, except for those days[(2)] when recording malfunction had occurred and on those days the wind speed that would have been recorded is inferred from the record for the 150 foot anemometer with a conversion factor of 0.7.

Covariance of lake currents with wind speed and direction are visually obvious in Figure 1. The relationship is particularly clear on the 27th and 30th of May, 1965. Lag in changes in speed in the same direction does not seem to be more than one hour; however, change in direction from east to west may take four or five hours for the moderate winds from the east observed during the recorded period.

The wind-driven currents are superimposed on an easterly drift of approximately 0.1 knots, as shown quite well on the 19th and 21st of May, 1965. This current was present throughout the period of the record, but is weaker (0.05 knots) during the later portion of the record (29-31, May 1965). The observed horizontal temperature distribution was in accord with a geostrophic current of the magnitude observed. Decreasing temperature with increasing distance offshore was found with a gradient of approximately 1° Celsius per 1500 feet. In considering a geostrophic current, cause and effect are not resolvable and it can only be stated that the observed density distribution would be in equilibrium with a current of the direction and magnitude observed. The observed density distribution could be produced by more rapid temperature increase in the shallow nearshore waters by solar heating and by the supply of warmer water from the Genesee River. Both of these processes were certainly occurring during the May period. These processes will not be as important during other seasons of the year and weaker geostrophic currents would be expected.

The relationship between wind speed and water current speed has been observed by several authors and values ranging from 1.6 to 2.3 percent reported for open water. Since observed wind speed is a function of elevation (anemometer height) and surface roughness, the observed relationship is dependent on the particulars of the wind observations. For our observations, the lake current (mph) was approximately 0.023 times the wind speed (mph) observed at the 50 foot anemometer on the Brookwood site. If the 150 foot anemometer record bad been used, the relationship would have been 0.016. The observed relationship may be used with the statistics of the wind speed observations to produce reliable statistics of Lake speed, with uncertainties of not more than 1.5 percent. Frequencies of particular directions and speeds may be derived after an adequate length of wind record at the site has been developed.

In addition to the observations with the fixed current meter, current measurements from the survey boat were made during all three study periods. On those days when wave height permitted anchoring without excessive swinging on the two anchor lines, currents were measured with a confined drag. When anchoring was impractical, a free drifting drogue was used and the time to travel known distances recorded. These observations were in general agreement with the results from the fixed meter. During October 1965, the persistent eastward current was 0.05 knots and the relationship between wind current and wind speed was 0.02.

Currents were measured during periods of increasing wind speed and time lags of less than one hour were observed.

Vertical current and temperature profiles were observed at three positions off the point on the site. These stations were located 1000, 3700 and 6000 feet north of the point. Essentially uniform speeds were found in the upper ten feet, except during periods (2-4 hours) of direction reversal to the west when the surface layer was found to move downwind, while the deeper (below 5 feet) water was still flowing to the east. Considerable horizontal shear was found, except during periods when the currents were 0.15 knots and less. Speeds offshore were frequently double those nearshore, when the nearshore speeds were in the range 0.2-0.4 knots.

The only observed currents that are not accounted for by wind stress and the density distribution are those on 25 July, 1965. Winds had been 7-11 mph from the northwest for the previous 18 hours. Currents at the nearshore station were 0.25 knots to the east. At the offshore stations, speeds of 0.63 knots in the upper 25 feet were found. This transitory current was perhaps the result of internal wave motion associated with the strong thermocline present during the July period. While no systematic series of observations were made for the purpose of detecting internal waves, the temperatures shown in Table 1 show large temperature changes at depths of 18 to 25 meters that can only be accounted for by internal wave motion with amplitudes of 8-10 meters.

Our interest was not in the details of the velocity field per se, but rather as confirmation of the tracer studies (below) which show strong dispersion of the released material. The time and space variations found in the current system would be expected to produce rapid dispersion.

2B.1.5 TRACER RELEASES

1. Technique.

   A tracer material, rhodamine B, was released continuously off the site at the rate of 8.3 pounds per day. Dye solution was discharged with a metering pump through a pipe line to an outfall 1000 feet north of the point on the site. A diffuser distributed the solution through the upper eight feet of the water at the discharge point. The periods of pumping were:

   1500, 29 April, 1965—2230, 16 May, 1965

   1730, 8 July, 1965—1000, 25 July, 1965

   1530, 6 Oct., 1965—1510, 16 October, 1965

Clogging of the discharge system with decaying, floating algae was a continuing problem during the study. Algae growth began in early May and extensive beds were present during July and October. Underwater observation showed the beds extended offshore to depths of 12-15 feet. Each period of strong winds broke the algae loose and produced dense mats along the shore line.

The concentration of the tracer material was measured with a fluorometer operated on the survey boat. An underway sampling system permitted continuous recording of tracer concentrations along horizontal transects. Vertical profiles were measured by lowering the intake of a hone through which the sample water flowed to the instrument. Temperatures were also monitored on the same sample stream and noted on the fluorometer record.

At distances greater than 3000 feet from the outfall, the vertical distribution of tracer was uniform during July and October, with the exception of the offshore edge of the dye plume. Higher concentration with depth was found on the outer edge, suggesting offshore movement in the deeper (10-20 feet) layers. During the first week of May, vertical mixing was incomplete due to the rapid heating of the upper ten feet and the dye was confined to this surface layer.

Horizontal transects were taken perpendicular to the shore at intervals of approximately 2500 feet from the discharge point to the area where the tracer was undetectable. These records have been used to construct charts of the horizontal distribution of the tracer, as shown in Appendix A. The wind speeds and directions during the observations are shown in Appendix B.

2. Results.

   a. April-May.

      The chart for 29 April shows the distribution four hours after the release was begun. The wind had been from WNW at approximately 13 mph (50 feet anemometer) during the four hours. The drift of the material over a distance of one mile during the four hours is an average rate of 0.25 knots. The ratio between tracer drift and wind speed was 0.02 in close agreement with the current measurements described above. The onshore set of the drift should be noted. Tracer material discharged 2000 feet off the proposed location of the cooling water outlet from the plant reached the shore area approximately 5000 feet down current. Discharge at greater distances offshore would not radically alter this pattern, and it would be expected that material released 4000 feet offshore would approach the shore at distances of 10,000 feet down current.

      The tracer release rate was not uniform on the 30 April due to interference by the algae noted above. The resulting distribution on 1 May with high concentrations in a patch off Bear Creek Harbor is a result of this artifact. The discharge was maintained constant throughout the remainder of the May test.

      The patterns observed during the two weeks of steady release show a slow drift to the west on 4 days and to the east on the other 13 days, which is a frequency that corresponds to average conditions as suggested by the wind record analyses described in the preliminary hazards evaluation. The distribution on 14 May is particularly significant. Drift to the east had begun on 9 May and had been persistent through the five days. During the afternoon of the 13th and again on the 14th, the winds were from the

NE and reversal of the drift to the west was being initiated. The reversal is associated with the development of a confused (turbulent) current pattern with large eddies (several thousand feet in diameter). This motion produces much more extensive dilution than that which occurs with persistent drift along the shore. It was striking that when drift direction reversed, return passage of the large quantity of water containing the previously released material could not be observed to any great extent. The observed rapid diluting process apparent in the 14 May chart is the probable reason for this effect. General accumulation of released material in the area was not observed and it is implied that the exchange rate of the nearshore waters with the bulk of the Lake proceeds so rapidly (weeks) that "new" (i.e., water whose tracer content corresponds to a dilution of 1:100 for the proposed rate of plant discharge) water is available for the development of the plumes that form with persistent winds from either the west or east.

b. July.

The July test period was dominated by drift to the east produced by west winds. The only drift to the west was found on 13 July. The effect of southwest winds are well represented in the July results. Both the 9th and 17th show the offshore drift resulting from southerly components in the wind.

c. October.

The October results are also dominated by drift to the east. However, the 7, 8, and 9 October distribution show the movement to the west developed by southeast winds. The 8 October distribution is the maximum excursion to the west observed during all three study periods. The return to east drift may be seen on 9 October and the rapid dilution due to large-scale turbulence is similar to that observed with reversal from east to west drift.

3. Interpretation.

The observed distribution of tracer material may be used to compute the comparable distribution of other materials discharged at the site on the basis that the ratios of concentration to the quantity discharged are identical. The tracer distributions are shown in units of parts per billion and, for the tracer discharge rate used, 1 ppb corresponds to 2.66×10^{-13} unit per cc per unit per day discharged. For example, for those areas where the dye concentration was 0.1 ppb, it would be predicted that, with a radioactive isotopes discharge rate of 1 millicurie per day, the concentration of radioactive isotopes would be 2.66×10^{-13} millicuries per cc in those areas. Similar scale factors may be derived from any other units chosen for expressing discharge rates.

If the tracer had been injected into a cooling water flow of 290,000 gpm (647 cfs), the concentration for our injection rate would have been 2.4 ppb. An alternate way of viewing the observed distributions would be on the basis of dilution from the base concentration of 2.4 ppb. In this case, 0.24 ppb would correspond to a dilution of 1 to 10, etc.

Inspection of the observed distributions show that concentrations greater than 2.4 ppb were observed several thousand feet down current from the tracer outfall. Natural turbulent dispersion does not furnish as rapid dilution as may be achieved in the cooling water discharge canal. Release of materials from the plant should be by way of the discharge

canal to eliminate these high concentrations present close to a single point discharge out in the lake. At distances greater than roughly four thousand feet, the manner of discharge will not modify the concentrations significantly, except for the effect of momentum mixing as discussed below.

In using the observed tracer distributions to anticipate the distribution of plant discharges, direct scaling can be used as outlined above. An alternate approach to developing predictions is the description of observations with theoretical equations, which may then be used to compute distributions for various assumed conditions. An example of this type is the simple peak concentration equation used in the preliminary environmental analysis to predict peak concentrations on the basis of an assumed diffusion velocity and a point source discharge.

It seems clear that release of materials from the site with condenser cooling water flow has distinct advantages and it is assumed that this will be the manner of release. This cooling water flow is not equivalent to a mathematical point (vertical line) source. The relationship proposed by Okubo and Pritchard (Okubo, Akira. 1962. A Review of Theoretical Models of Turbulent Diffusion in the Sea. Chesapeake Bay Institute, The Johns Hopkins University, Technical Report 30, Reference 62-20) for horizontal diffusion from a vertical line source at times so that steady state has been achieved is *equation (1)*.

$$S_c(x, y - y_0, t_s) = \frac{q}{2\sqrt{\pi} \cdot D \cdot w[x^2 + (y - y_0)^2]^{1/2}} \exp\left[-\frac{U(y - y_0)^2}{x^2 + (y - y_0)^2}\right]\left[1 + erf\,\frac{(xU)}{w(x^2(y - y_0)^2)}\right] \quad \text{(Equation 1)}$$

where: $S_c(x, y - y_0, t_s)_l$ =concentration of material in mass per unit volume from a continuous vertical line source located at x = 0, y = y_0.

w = diffusion velocity.

U = velocity in x-direction (It is assumed to be constant in this model; in addition V = W = 0).

q = rate of discharge of material uniformly over a depth, D.

and the x axis points along the plume and the y axis across the plume.

Equation (1) may be applied to a vertical plane source of length b and depth D running from x = 0,y = 0 to x = 0, y = b by integrating as shown in *equation (2)*. The boundary effect is incorporated as a virtual source running from y = 0 to y = -b.

$$S_c(x, y, t_s)_p = \frac{1}{b}\int_{-b}^{+b} S_c(x, y - y_0, t_s)_l \, dy \quad \text{(Equation 2)}$$

where: $S_c(x, y, t_s)_p$ =concentration of material in mass per unit volume from a continuous plane source of length b and depth D.

Conditions described by *equation (2)* are shown schematically in Figure 1.

Equation (2) may be non-dimensionalized as follows:

Let:

$$x = bx' \quad y = by' \quad y_0 = by_0' \quad \frac{U}{w} = U' \qquad \text{(Equation 3)}$$

Equation (2) then becomes

$$\frac{S_c(x',y',t)_y \cdot 2\sqrt{\pi} \cdot D \cdot w \cdot b}{q} \equiv S_c' = \int_{-1}^{+1} \frac{1}{(x'^2 + (y'-y_0')^2)^{1/2}}$$

$$\exp\left[-\frac{U'^2(y'-y_0')^2}{(x'^2+(y'-y_0')^2)}\right]\left[1+erf\left(\frac{x'U'}{(x'^2+(y'-y_0')^2)^{1/2}}\right)\right] \qquad \text{(Equation 4)}$$

The right-hand side of *equation (4)* is not integrable except by machine methods. It has been evaluated for representative values of U ′ as a function of x ′ and y ′ and may be considered as known from this point on.

Using U ′ = U/w and *equation (4)* we obtain

$$S_c' = \frac{S_c\sqrt{\pi}\, DUb}{U'q} \qquad \text{(Equation 5)}$$

Multiplying both aides of *equation (5)* by Q, the discharge rate of condensor-cooling water into the lake, we obtain

$$S_c'Q = \frac{S_c\sqrt{\pi}}{U'} \cdot \frac{DUb}{q/Q} \qquad \text{(Equation 6)}$$

but q/Q is the initial concentration of material, so on making this substitution and rearranging *(6)* we obtain

$$Dilution \equiv \frac{S_0}{S_c} = \left[\frac{DUb}{Q}\right]_{1st\ stage\ dilution} \left[\frac{\sqrt{\pi}}{U'S_c'}\right]_{2nd\ stage\ dilution} \qquad \text{(Equation 7)}$$

Equation (7) describes the dilution of introduced material as proceeding in two stages. The first stage is the dilution that occurs between injection and formation of the vertical plane source and the second stage is the dilution produced by natural dispersion as the material moves with the lake current.

The variation in dilution with distance down the plume for various values of U ′ is shown in Figure 2. These dilutions are the minimum to be expected at each distance, since they are for y - 0, i.e., the shore line of the lake.

The lateral distribution of material is shown in the form of second-stage dilution for a representative value of U ′ in Figure 3. The value of U ′ is derived from the observational data in the following way. The simplest characteristic of the observed plumes is the maximum concentration found along the plume. In considering *equation (1)*, if we define the x coordinate as running along the center of the plume and y = 0 along this line, the peak concentration, s_p, is given by the following relationship:

$$S_p = \frac{q}{2\sqrt{\pi}\,D \cdot w \cdot x}$$

The values of w are most readily found from a plot of peak concentration versus distance. In considering the observations, it must be remembered that the theoretical equations apply to steady-state conditions, which are only present in the lake after a persistent wind. Also, steady- state conditions do not exist along the entire length of the plume and the one-third of the plume farthest down current from the source is not at steady state. The equations apply to the mean concentration at each position and the observations are essentially instantaneous concentrations, so that considerable scatter about the theoretical functions must be expected.

Data under conditions approximating steady state have been selected from the complete set of observations for use in estimating the diffusion velocity, w. Plots of peak concentration versus distance are shown in Figures 4, 5, 6, 7 and 8. Figures 4, 5 and 6 are for drift to the east and Figures 7 and 8 are for data during drift to the west. During the July study period, significant drift to the west did not occur and no west plot is made for July.

For drift to the east, the data for May, July and October are in close correspondence, being described by a diffusion velocity of 0.33 cm/sec during all three periods. Mean drift speed during the intervals when near steady state was approached was approximately 0.2 knots (10 cm/sec). The corresponding value of U ′ is 30, which is the value used in constructing Figure 3.

Drift to the west did not occur for a sufficient length of time to produce steady state at distances of greater than 4000 meters. The May results in Figure 7 suggest that a diffusion velocity of 0.33 cm/sec is descriptive of dispersion during drift to the west. The October results appear to be better fitted by larger values of w, but steady state was not established and 0.33 cm/sec may be taken for prediction purposes with conservatism.

2B.1.6 DISCUSSION

Predictions of the distribution of released materials may be based on the above data and the characteristics of the discharge as it leaves the site. As noted above for *equation 7*, the dilution process may be viewed in two distinct steps for which the first stage is controlled by the geometry and momentum of the discharge and the second stage results from the natural turbulent dispersive motions in the lake.

2B.1.7 POINT SOURCE

If the condenser cooling water were released to the lake through a relatively wide and deep canal so that the discharge had velocities of a few tenths of feet per second (negligible momentum) or through a single outlet on the bottom a few thousand feet offshore, the point source equation would be applicable and the first stage dilution quite small. In this case decrease in concentration requires travel over relatively great distances and 6600 feet would be required for dilution of 1 to 2 and 13,000 feet required for a dilution of 1 to 4. These dilutions correspond to temperatures of 11 and 5.5°F in excess of natural temperatures, neglecting heat transfer to the atmosphere. Since further dilution would be beneficial, other modes of discharge were examined as indicated below.

2B.1.8 LINE SOURCE

The first stage dilution may be increased by distributing the discharge along a line running perpendicular to the shore. This distribution could be provided by a multiple outlet pipe (diffuser) or by moderate (0.5 ft/sec) velocity canal discharge to produce a plume which moves an equal distance offshore before losing its momentum.

2B.1.9 DIFFUSER SOURCE

Optimum diffuser design with a large (50-100) number of ports in a 1500 ′ length could provide first stage dilution with all the water flowing across the diffuser length. The second stage dilution would be as described by *equation 7* and Figure 2. Dilution under the various lake current speeds is estimated by considering three examples.

1. Average lake current speed. With a speed of 0.2 knots (10 mph wind), the first stage dilution would be 1 to 4.1 for 700 cfs and the surface excess temperature along the distributor would be 5.4°F for a condenser temperature rise of 22°. Having generated this line (vertical plane) source of excess heat or material, significant second stage dilution requires travel over great distances, as shown in Figure 3. The computed distance for a second stage dilution of 1 to 2 is 13.5 miles, with a travel time of 2.8 days. Heat loss to the atmosphere would be significant and this minimum dilution would not be observed at 13.5 miles unless the flow persisted for approximately five days. Excess temperatures of less than one degree may be anticipated at a distance of 13.5 miles, due to cooling but dilution of conservative (stable) materials would be 1 to 8.2.

   At a distance of six miles (for example, off Pultneyville to the east of the site), the second stage dilution would be 1 to 1.1 and the total dilution 1 to 4.5. Excess temperature would be 4.9° without considering heat loss to the atmosphere and using a heat loss coefficient of 0.1 ft/hr as typical of summer conditions, a temperature elevation of 2.9° would be expected. The travel time is 1.25 days so that steady state would be approached after approximately two days, which would frequently occur. These calculations are in agreement with the observed tracer distributions, as for example, on 13 October, 1965 with persistent east drift for the previous four days, tracer concentrations off Pultneyville were 0.5 ppb and the computed concentration with the dilution of 1 to 4.5 derived above is 0.53 ppb.

2. Minimum lake current speed. The minimum speed that persists for more than a few hours is 0.05 knots. The first stage dilution with 1500 feet of distributor length would be 1 to 2.3.

Further dilution (second stage) of 1 to 2 would be expected at a distance of 5 miles down current. The total dilution of 1 to 4.6 at five miles for these conditions is not greatly different from that computed above under average conditions, which was 1 to 4.5 at six miles. Temperature elevation at the surface over the distributor area would be 9.6°. For this minimum lake current speed case, atmospheric cooling would be more important than second stage dilution. Drift over the five mile distance would provide dilution of 1 to 4.6 or a temperature excess of 4.7°, but heat loss to the atmosphere would have reduced the temperature excess to 1.1° during the 4.2 days required to travel the five miles.

It should be noted that the distributor would produce an area approximately 1500 by 2000 feet with an excess temperature of greater than 9° during periods of minimum lake current speeds.

3. High lake current speeds. The maximum observed current speed was 0.5 knots under the influence of 20 mph winds. For this speed, the first stage dilution would be 1 to 10.5. Further dilution by natural turbulence (second stage), even if it is assumed that the diffusion velocity is 0.6 cm/sec (the maximum observed), would occur only after drift for large distances. Second stage dilution of 1 to 2 is computed to occur 25 miles down current. The distributor system would be quite effective near the site under conditions of high lake current speeds, which are not frequent, and would not greatly change the concentrations several miles from the site.

2B.1.10 JET SOURCE

Another manner of cooling water discharge is release of the flow through a restricted opening so that the discharge has velocities considerably greater than the lake velocity. Literature review and model studies of warm-water jet behavior are described by Yuan Jen, R. L. Wiegel and Ismail Mobarek (1966, Surface discharge of horizontal warm-water jet, Journal of the Power Division, ASCE, No. PO2, 4801). A more extensive literature is available for gases (smoke stacks, wind tunnels, etc.) released at right angles to moving air streams. The model studies of Jen, et al were run at very high densiometric Froude numbers, which will not be present in many jet discharge situations and the effects of heated discharge do not appear to be adequately scaled. Observations of discharges with the desirable velocity and volume are not available. Observations on gases and model studies are extrapolated to the conditions possible at the Brookwood site using conservative choices of assumptions to produce a conservative prediction.

Jet discharge into a stationary fluid produces a plume which consists of an initial mixing zone that has a length that is 3 to 5 times the original discharge width and beyond this region the velocity decreases as the volume flux increases due to entrainment of the receiving fluid. For an unbounded jet, the velocity decreases to 0.1 of the initial velocity at distances approximately 60 times the nozzle diameter and the velocity decreases in proportion to the reciprocal of the distance from the nozzle. For large volume flows like condenser cooling water discharged at the surface, the jet is bounded by the free water surface and, due to its buoyancy from heating, may be assumed not to mix vertically and, therefore, is bounded by a lower surface represented by the abrupt density change with depth. Some vertical mixing will occur, but it is neglected here since there are no reliable estimates of this mixing and this procedure

is conservative. For the bounded jet, the velocity decreases with the reciprocal of the square root of the distance from the nozzle.

Observations of jets have shown angles of spread ranging from 1 to 5 through 1 to 8. With wider angles of spread, the entrainment processes is proceeding rapidly with distance. We assume an angle of spread of 1 to 6 as being conservative. From this assumption and the distance dependence assumed above, the shape and concentrations in the jet may be calculated. The effect of the horizontal movement in the lake is taken into account as momentum contributed to the jet by entrainment to produce a deflection of the jet. This momentum mixing process is described in Attachment I to this Appendix.

Figure 9 shows the calculated jet pattern for a 50 by 6 feet nozzle (canal) discharging 700 cfs into water with 0.5 feet per second flow at right angles to the initial jet axis. This pattern is translated into a predicted temperature distribution as shown in Figure 10, where the effect of recirculation on the downstream (wake) side of the jet is taken into account to produce an accumulation of heat in this area. As may be seen by comparing Figures 9 and 10, recirculation along streamlines corresponding to rough semicircles with a radius of 1000 feet is assumed. The probable pattern along this downstream edge has not been described in the studies available in the literature on jets. However, if the recirculation is by way of streamline patterns of smaller or greater radius, lower temperatures will be present in this area and Figure 10 seems to be a conservative estimate. The most likely pattern is recirculation along streamlines with a radius of a few hundred feet, which will produce lower temperatures than shown in the wake region.

These predicted concentrations of heat and material have been made without considering processes other than dilution. Heat loss to the atmosphere and radioactive decay would be significant if such rapid dilution were not available.

The dilutions have been calculated assuming the discharge will not be mixed deeper than six feet; an assumption that produces higher concentrations than would be calculated if greater vertical mixing occurs. If the cooling water intake is located off shore on the bottom, possible recirculation could occur only with extensive vertical mixing (high speed winds). With an intake depth of 28 feet, complete mixing would produce a dilution of 1:4.7 in addition to that shown above and the water drawn into the intake would be a 30 fold dilution of discharged water.

The public water intake nearest to the Brookwood site is the town of Ontario pumping station, which draw water from an inlet about 1100 feet off shore at a depth of 11 feet. Momentum mixing would produce a dilution of 1:10 in the upper six feet, and complete vertical mixing would produce a total dilution of approximately 1:20 for water drawn into this intake.

The anticipated temperature distribution of the surface waters off the Brookwood site is not expected to produce significant effects. Fish may not prefer the limited area (1000 by 100 feet) immediately adjacent to the discharge canal. There is no evidence in the form of fishing activity in the area now that suggests significant fish populations. The larger area (6000 by 2000 feet) with temperatures a few degrees above ambient may attract fish, as has been observed in other localities where discharge of similar quantities and temperatures of heated water has been studied.

Table 1
Temperature (°C) at a station 6000 feet offshore of Brookwood

Depth (meters)	9 July	10 July	12 July	16 July	17 July	25 July
S	19.02	18.10	17.69	18.97	19.09	20.36
2	18.84	17.27	17.59	18.80	18.59	20.15
4	18.19	17.20	17.45	18.61	18.56	20.00
6	16.85	17.20	17.26	18.58	18.55	19.97
8	16.50	17.20	17.14	18.55	18.55	19.97
10	16.44	17.13	17.03	18.53	18.55	19.97
12	15.66	17.06	16.91	18.10	18.50	19.80
14	14.23	16.95	16.84	17.80	18.41	19.70
16	10.44	16.84	16.83	17.71	18.38	18.58
18	5.35	15.52	16.82	17.65	18.37	17.73
20	4.99	12.22	16.80	17.60	18.30	16.81
22	4.94	10.09	16.80	17.31	16.08	12.50
25	4.94	6.70	16.71	11.76	6.25	7.00

APPENDIX A TO APPENDIX 2B

OBSERVED TRACER DISTRIBUTIONS (PARTS PER BILLION)

APPENDIX B TO APPENDIX 2B

WIND SPEED AND DIRECTION OBSERVATIONS
METEOROLOGICAL TOWER ON BROOKWOOD SITE
ANEMOMETER - ELEVATION 150 FEET

Table 1

	29 April	30 April	1 May	2 May	3 May
0100	W/12	SW/17	NNW/4	SSW/13	SE/2
0200	WNW/12	SW/16	NNE/5	SSW/9	ESE/5
0300	WNW/11	SW/16	NE/3	S/7	SSE/5
0400	WNW/13	SW/17	SSW/2	S/2	S/5
0500	W/12	SW/16	S/5	ENE/5	SSE/3
0600	W/12	WSW/16	SW/8	ESE/8	SSW/3
0700	WSW/11	WSW/19	SW/12	E/7	SSW/1
0800	W/13	WSW/20	SW/6	ESE/4	SW/3
0900	W/13	WSW/17	SSW/6	ENE/5	WSW/6
1000	WNW/11	W/19	ENE/7	ENE/7	W/8
1100	WNW/14	W/20	ENE/5	ENE/7	W/3
1200	WNW/10	WNW/18	NE/7	ENE/7	WNW/6
1300	NW/11	WNW/21	ENE/6	ENE/6	W/13
1400	WNW/10	WNW/22	ENE/7	E/9	W/17
1500	NW/16	WNW/23	ENE/6	E/8	WSW/10
1600	WNW/18	WNW/25	E/6	ENE/9	W/25
1700	W/15	WNW/24	NE/4	ENE/8	WSW/23
1800	W/17	WNW/24	E/2	ENE/9	WSW/26
1900	W/19	WNW/22	E/3	E/16	WSW/17
2000	WSW/22	W/20	E/4	E/15	WSW/20
2100	WSW/20	WNW/17	SSE/4	E/12	WSW/20
2200	WSW/21	NW/13	S/3	E/12	WSW/21
2300	WSW/19	WNW/5	SSE/7	SE/8	WSW/21
2400	WSW/17	WNW/4	S/12	SSE/7	WSW/21

	4 May	5 May	6 May	7 May	8 May
0100	W/23	SW/3	S/12	ESE/9	SSE/14
0200	WSW/22	SW/0	SW/5	ESE/0	SSE/12
0300	WNW/18	NE/2	NNE/5	ENE/5	SSE/14
0400	W/14	ENE/3	SSE/0	SSE/6	SSE/13
0500	W/12	E/3	ENE/0	SE/12	SSE/15
0600	WNW/15	E/1	WNW/2	SE/16	SSE/11
0700	NW/17	ENE/3	NNW/0	SE/10	SSE/11
0800	W/9	E/5	SSE/3	SE/11	SSE/10
0900	WSW/5	E/5	ENE/0	ESE/8	SSE/15
1000	WNW/8	E/3	ENE/3	ESE/9	SSE/13
1100	WNW/10	ENE/6	ENE/5	SE/11	S/9
1200	NNW/3	ENE/7	ENE/10	SSE/18	SSE/8
1300	ESE/5	NE/9	ENE/5	SSE/21	SSE/10
1400	E/6	NE/11	ENE/11	SSE/21	SSE/11
1500	ESE/5	NE/7	ENE/9	SSE/21	SE/8
1600	E/4	ENE/8	ENE/5	SSE/23	SSE/7
1700	E/3	ENE/9	ENE/6	SSE/16	SE/9
1800	ESE/1	ENE/6	E/9	SSE/16	SE/9
1900	SSE/2	E/6	E/11	SSE/15	SSE/18
2000	SSE/5	E/8	E/12	SSE/16	SSE/16
2100	SW/5	ESE/6	E/12	SSE/17	S/16
2200	WSW/4	SE/7	E/12	SSE/17	SSE/18
2300	WSW/2	SSE/11	ESE/10	SSE/16	S/15
2400	SSW/2	S/12	ESE/5	SSE/14	S/15

	9 May	10 May	11 May	12 May	13 May
0100	SSW/14	WNW/11	SW/15	W/9	W/11
0200	SSW/16	SSW/6	WSW/17	W/10	WNW/11
0300	W/15	WSW/8	W/17	SW/5	WNW/14
0400	SSW/3	SW/11	W/16	WSW/9	NNW/14
0500	SSW/9	WSW/10	W/22	WSW/11	N/19
0600	SW/6	SW/8	W/15	SW/12	NNE/15
0700	WSW/7	SW/9	W/15	SW/11	N/18
0800	WSW/8	WSW/8	W/14	SW/9	N/15
0900	W/11	WSW/11	W/14	SW/9	N/14
1000	WNW/17	SW/10	WNW/11	WSW/8	N/19
1100	WSW/6	W/7	WNW/16	WNW/12	NNW/14
1200	SW/8	N/5	WNW/16	WNW/13	NNW/15
1300	SSW/17	NE/5	WNW/13	NW/15	NNW/12
1400	SW/15	NE/5	W/15	NW/15	NN@/11
1500	W/12	NNE/4	WNW/17	NW/9	NW/9
1600	WNW/17	ENE/2	WNW/17	WNW/5	WNW/11
1700	NW/5	E/5	W/17	NW/7	WNW/8
1800	WNW/1	ENE/0	WNW/21	W/17	WNW/6
1900	WSW/8	WNW/2	WNW/18	W/12	WNW/9
2000	WSW/7	SW/5	W/12	WSW/15	WNW/10
2100	WSW/9	SW/5	W/8	WSW/15	WNW/9
2200	WSW/9	SSW/8	SW/11	WSW/17	S/0
2300	SW/11	SSW/10	WSW/11	WSW/17	S/0
2400	SSW/13	WSW/14	WSW/11	W/14	SSW/3

	14 May	15 May	16 May
0100	S/5	S/12	S/13
0200	ESE/7	S/13	S/12
0300	SSE/8	SSW/13	S/14
0400	S/7	S/11	SSW/14
0500	SSW/4	S/5	SSW/13
0600	WSW/8	S/5	SSW/13
0700	W/4	SE/5	SSW/11
0800	WSW/0	SSW/2	SSW/10
0900	N/2	S/4	SSW/11
1000	N/3	SSE/5	SSW/12
1100	NNE/3	SE/2	SW/14
1200	NE/5	ENE/6	SW/13
1300	ENE/4	ENE/8	SW/12
1400	ENE/5	ENE/7	SSW/13
1500	ENE/5	ENE/8	SSW/16
1600	E/6	ENE/7	W/9
1700	ENE/6	E/13	NE/0
1800	E/6	E/11	SW/11
1900	E/10	E/8	SSW/14
2000	E/8	E/9	S/14
2100	ESE/6	ESE/10	SSW/16
2200	SE/7	SE/11	SSW/15
2300	SSE/9	SE/11	WNW/11
2400	SSE/10	SSE/13	W/8

	9 July	10 July	11 July	12 July	13 July
0100	S/12	WSW/12	WNW/13	WNW/8	N/7
0200	S/12	W/13	WNW/10	WNW/9	N/7
0300	S/12	W/15	WSW/9	WNW/8	N/5
0400	S/9	WNW/12	WSW/8	W/6	N/1
0500	S/11	WNW/16	W/10	SW/10	N/2
0600	S/11	W/12	W/10	WSW/11	N/5
0700	S/11	W/14	WSW/6	WSW/9	N/1
0800	SSW/9	WNW/16	WNW/12	W/7	N/4
0900	SSW/5	WNW/12	WNW/9	WNW/10	N/5
1000	SSW/6	WNW/14	WNW/8	WNW/11	N/8
1100	SSW/5	WNW/13	WNW/9	WNW/9	N/2
1200	SSW/9	NW/9	WNW/11	NW/7	N/2
1300	SSW/14	NW/7	WNW/9	NNW/4	N/3
1400	SSW/14	NNW/3	WNW/11	NNW/4	N/4
1500	SSW/13	NNW/1	WNW/9	NNE/8	N/4
1600	S/14	NNW/1	WNW/8	NNE/4	N/6
1700	S/14	NNW/3	WNW/9	NNE/2	N/5
1800	S/12	WNW/14	W/10	N/3	N/6
1900	S/11	WNW/6	WNW/10	ENE/0	N/2
2000	SW/23	WNW/5	W/7	SE/0	N/2
2100	WSW/18	W/9	W/10	SE/0	N/3
2200	WSW/13	WNW/15	W/10	N/2	N/7
2300	SW/17	WNW/15	W/10	N/3	N/12
2400	WSW/12	WNN/13	WNW/8	N/5	N/12

	14 July	15 July	16 July	17 July	18 July
0100	N/14	WNW/7	W/9	WSW/10	N/3
0200	N/13	WNW/10	W/9	SW/9	N/2
0300	N/13	NW/16	WSW/10	W/10	ENE/4
0400	N/12	W/12	SW/9	WSW/4	NE/6
0500	N/14	W/13	WSW/10	SW/7	NE/16
0600	N/16	W/11	SW/11	SW/2	ENE/10
0700	N/18	W/13	SW/9	S/3	NE/11
0800	N/15	WNW/13	SSW/7	E/0	ENE/7
0900	N/16	WNW/14	SSW/7	SE/3	ENE/6
1000	N/15	NNW/16	WSW/4	SE/1	NE/5
1100	N/11	NW/15	NW/2	E/0	NE/4
1200	N/13	WNW/12	NE/5	ENE/4	NNE/8
1300	W/14	NW/10	NE/7	ENE/4	NNE/18
1400	W/16	NW/7	ENE/7	ENE/5	NNE/11
1500	W/15	NW/5	ENE/7	NE/7	N/10
1600	W/20	NW/10	E/6	ENE/6	NNW/10
1700	W/8	NW/9	E/9	ENE/5	NNW/9
1800	W/5	NNW/7	ESE/5	E/5	NNW/11
1900	NW/5	NW/3	ESE/3	NE/4	NNW/7
2000	NW/8	WNW/5	SE/5	NE/4	NNW/6
2100	WSW/12	W/11	SE/5	ENE/1	NNE/4
2200	WSW/10	W/10	SSW/6	SE/3	NNW/7
2300	W/8	W/9	SW/8	NNE/1	NNE/9
2400	W/8	W/9	SW/9	NNW/1	NNE/15

	19 July	20 July	21 July	22 July
0100	NNE/13	NNE/14	WSW/10	S/10
0200	N/12	NNE/11	WSW/9	S/11
0300	N/12	NNE/8	WSW/10	S/10
0400	N/11	N/7	WSW/11	S/9
0500	NNW/9	NW/2	SW/11	S/8
0600	N/12	NW/3	WSW/11	S/8
0700	NNW/9	WNW/6	WSW/10	S/7
0800	NNW/9	W/5	WSW/6	WSW/3
0900	NW/11	NW/14	WNW/6	SW/4
1000	NW/16	NW/15	WNW/7	WNW/7
1100	NW/18	NW/14	WNW/7	WNW/5
1200	NW/17	NW/11	NW/4	WNW/5
1300	NW/20	WNW/11	NNW/4	NNW/3
1400	WNW/17	NW/10	NNW/6	N/0
1500	WNW/16	NW/11	NNW/5	NE/1
1600	WNW/17	WNW/10	NNW/4	ENE/0
1700	WNW/18	WNW/8	NNW/3	NE/1
1800	WNW/21	WNW/7	N/3	ESE/6
1900	N/5	WNW/5	N/2	S/1
2000	NW/6	W/3	E/0	SSE/3
2100	WNW/13	WNW/5	SSE/3	SE/6
2200	NNW/11	W/9	SSW/2	SSE/9
2300	N/11	W/11	SSW/5	SSE/10
2400	N/14	W/12	S/9	S/7

	23 July	24 July	25 July
0100	S/8	WNW/6	WNW/16
0200	SSW/8	WSW/7	WNW/13
0300	SSW/9	WSW/6	W/12
0400	SSW/6	SW/7	WNN/12
0500	SSW/7	SW/9	W/9
0600	S/5	SSW/9	WSW/7
0700	SSW/5	N/0	WSW/7
0800	SSW/4	SSW/6	W/9
0900	SW/1	SSW/8	WNW/10
1000	NNW/2	SW/6	WNW/8
1100	NNE/3	WNW/12	NW/9
1200	N/2	NW/5	NW/7
1300	NNE/2	NE/3	NNW/6
1400	NNE/5	ENE/3	N/4
1500	NE/4	E/2	NNW/3
1600	NE/5	WNW/10	NNW/9
1700	NE/4	W/17	NW/6
1800	NE/3	WNW/12	WNW/5
1900	N/0	W/15	WNW/8
2000	WNW/2	W/14	W/5
2100	WNW/3	NNW/11	W/8
2200	W/5	WNW/12	WSW/10
2300	W/6	WNW/10	SW/10
2400	WNW/5	WNW/11	SW/10

	7 Oct.	8 Oct.	9 Oct.	10 Oct.	11 Oct.
0100	S/17	ESE/8	SSW/8	SW/10	WSW/8
0200	SSE/17	SSE/4	SW/13	WNW/14	WSW/8
0300	S/15	SSW/1	SW/14	WNW/14	W/11
0400	S/15	S/4	SW/15	WNW/15	SW/6
0500	SSE/16	SSW/10	SW/16	NW/17	WSW/7
0600	SSE/15	SW/12	SW/15	W/11	SW/9
0700	S/15	SSW/14	SSW/16	WNW/14	SSW/7
0800	S/16	SSW/10	SW/15	WSW/10	SSW/7
0900	SSE/18	SSW/12	SW/18	W/9	SSW/7
1000	S/16	SW/12	SW/16	NW/15	SSW/8
1100	SSE/18	SW/19	SW/17	WNW/12	S/8
1200	S/19	SW/13	WSW/18	WNW/11	S/11
1300	SSE/18	SSW/15	WSW/22	WNW/14	S/6
1400	SSE/18	SSW/17	WSW/21	WNW/10	SSW/5
1500	SSE/16	SSW/18	WSW/13	NW/15	SW/9
1600	SE/17	WSW/15	SW/16	WNW/18	WSW/16
1700	SE/13	W/7	WSW/16	WNW/12	NW/15
1800	SE/11	SSW/7	SW/9	W/8	W/10
1900	SE/13	WSW/11	SW/9	W/9	WSW/9
2000	SE/12	W/11	SW/7	W/11	WSW/11
2100	SE/15	SW/15	SW/8	WSW/8	SW/9
2200	SE/17	SW/11	SSW/9	W/8	W/11
2300	SE/15	SSW/11	SSW/9	WSW/9	W/8
2400	ESE/15	SSW/11	SSW/8	WSW/10	W/7

	12 Oct.	13 Oct.	14 Oct.	15 Oct.	16 Oct.
0100	WSW/6	WSW/13	S/11	S/14	NW/14
0200	WSW/7	WSW/12	S/11	S/16	WNW/10
0300	WSW/9	WSW/11	SSE/11	S/16	W/10
0400	SW/9	WSW/11	S/9	S/15	W/12
0500	WSW/10	WSW/10	S/9	S/14	W/12
0600	SW/10	SW/10	SSE/7	S/15	WNW/11
0700	SSW/11	SW/10	W/1	S/15	W/9
0800	SSW/10	SW/9	SW/2	SSW/13	NNW/16
0900	SW/10	WSW/11	SSW/2	SSW/14	N/12
1000	SW/14	N/-0	SSE/1	SW/21	N/13
1100	SW/14	WNW/11	ENE/4	SW/24	N/13
1200	SSW/11	WNW/14	ENE/7	WSW/26	NNE/10
1300	WSW/25	WNW/18	ENE/7	WSW/23	NNE/8
1400	SW/26	NW/17	ENE/8	WSW/19	NNE/9
1500	W/14	NW/18	ENE/9	W/18	NNE/7
1600	W/21	NW/17	E/8	W/21	N/5
1700	W/23	W/10	E/6	W/14	N/7
1800	W. 20	W/4	E/8	WNW/15	NE/1
1900	WNW/14	S/5	E/10	WNW/21	ESE/0
2000	WSW/10	S/10	ESE/11	WNW/17	SW/0
2100	WSW/11	S/8	SE/11	NNW/18	SW/1
2200	SW/10	SSW/9	SSE/9	NW/15	WSW/2
2300	SW/12	S/9	S/13	WNW/15	NNW/0
2400	N/0	S/11	S/15	NW/17	WNW/1

APPENDIX 2B ATTACHMENT I

THE EFFECT ON LAKE DILUTION OF MOMENT MIXING

At distances greater than about 8 miles from the site, the equation used for the computation is given on page 2.4-10. The limnological data derived from the continuous tracer release tests are used in the form of the empirical diffusion velocity. The best fit to the observed data was given by a diffusion velocity of 3.3 x 10 m/sec and this value was used in the computations.

At distances less than about 8 miles from the site, the dilution is produced primarily by the momentum mixing resulting from the horizontal discharge of the circulating water system. This process may be evaluated from the following considerations.

Our coordinate system is as follows:

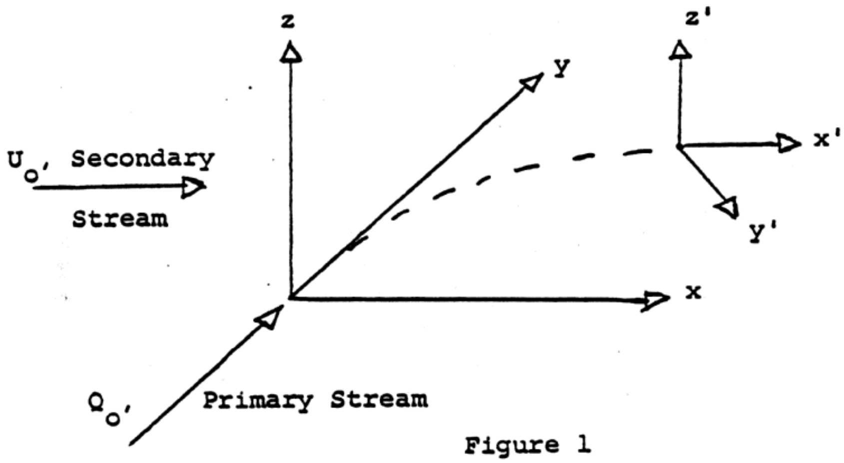

Figure 1

Subject to the following assumptions:

1. That longitudinal diffusion may be neglected,
2. That momentum and material spread at the same rate, and
3. That profiles of mean fluid attached properties such as concentration, velocity, etc. along an x ′ = constant plane, when scaled by their peak or centerline values, are expressable as a universal function of $z'/1_o$ and y'/l_o where 1_o (x ′) is a length scale. In our case we have chosen a simple top-hatted distribution function, i.e., at a particular x, values along y ′ are constant throughout the plume and zero outside the plume.

$$\bar{\sigma}(x') = \bar{\sigma}_M(x')\, f(y'/b(x')) \quad \text{where } f = 1 \text{ for } |y'| \leq b(x') \text{ and } f = 0 \text{ for } |y'| > b(x')$$

and where $\bar{\sigma}_M(x')$ is the fluid attached property and b(x ′) is the width of the top hat. the conservation of material for 3 dimensions may be written as

$$\frac{d}{dx'}\{D(x')\, b(x')\, Q_M(x')\, \bar{\sigma}_M(x')\} = 0 \quad \text{(Equation 1)}$$

or for a system bounded in the z ′ direction by the sea surface and by a level bottom we have

$$\frac{d}{d(x')}\{b(x')\,Q_M(x')\,\bar{\sigma}_M(x')\} = 0 \quad \text{(Equation 2)}$$

where Q_M (x ′) is the centerline velocity and $\bar{\sigma}_M$(x ′) is the centerline concentration of material.

Integrating (2) with respect to x ′ from x ′ = 0 to x ′ we obtain

$$\frac{\bar{\sigma}_M(x')}{\sigma_0}\cdot\frac{Q_M(x')}{Q_0}\cdot\frac{b(x')}{b_0} = 1 \quad \text{(Equation 3)}$$

In general it may be shown that assumption 3) requires that the lateral spread of the fluid attached properties be linear or that

$$\frac{db(x')}{dx'} = constant$$

Therefore, we write

$$\frac{b(x')}{b_0} = 1 + x'/x'_v,\ for\ x' \geq 0 \quad \text{(Equation 4)}$$

where x'_v is the distance from the orifice to the boundary between the zone of establishment and the zone of established flow. See Figure 2.

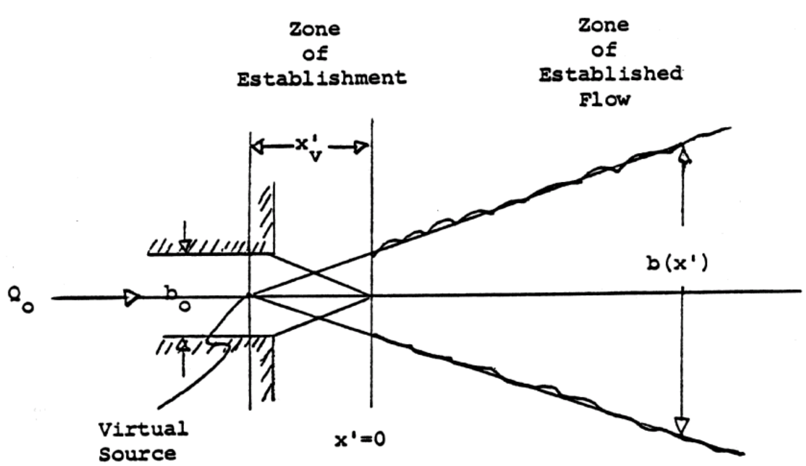

Figure 2

From (3) and (4) we obtain

$$\frac{\bar{\sigma}_M(x')}{\sigma_0}\cdot\frac{Q_M(x')}{Q_0} = \frac{1}{1 + x'/x'_v} \quad \text{(Equation 5)}$$

It now remains to relate $\frac{\bar{\sigma}_M(x')}{\sigma_0}$ and $\frac{Q_M(x')}{Q_0}$. To show this we proceed as follows.

It should be remembered that we are mixing two streams of water. The jet, or primary stream, initially possesses no x-momentum. By entrainment of the surrounding fluid, or secondary stream, it gains X-momentum and gives up Y-momentum and material. The jet continues to spread by entrainment into the secondary stream until such time as it has given up all of its material and Y-momentum. This occurs theoretically at infinity. We may express the foregoing concepts more formally as follows.

Consider a mixture of n parts of the secondary stream and m parts of the jet, or primary stream. We may write for concentration

$$\bar{\sigma}(x') = \frac{n \cdot 0 + m \cdot \sigma_0}{n+m} = \frac{m}{n+m} \cdot \sigma_0$$

or

$$\frac{\bar{\sigma}_M(x')}{\sigma_0} = \frac{m}{n+m} \equiv dilution$$ (Equation 6)

Similarly for X-momentum we have

$$U(x') = \frac{n \cdot U_0 + m \cdot 0}{n+m} = \frac{n}{n+m} \cdot U_0$$

or

$$\frac{U(x')}{Q_0} = \frac{n}{n+m} \cdot \frac{U_0}{Q_0} = \frac{n}{n+m} \cdot R$$ (Equation 7)

where $R \equiv U_0 / Q_0$ (Equation 8)

and for Y-momentum $\frac{V(x')}{Q_0} = \frac{m}{n+m} = \bar{\sigma}_M(x')/\sigma_0$ (Equation 9)

Combining equations (5), (6), (7), (8), (9) and remembering that

$$\frac{Q_M(x')}{Q_0} = \sqrt{\left(\frac{U(x')}{Q_0}\right)^2 + \left(\frac{V(x')}{Q_0}\right)^2}$$ (Equation 10)

on the centerline, we obtain

$$\frac{\bar{\sigma}_M(x')}{\sigma_0} = \left\{\frac{1}{1 + x'/x'_v}\right\}^{1/2} \qquad \left\{\frac{m^2}{n^2R^2 + m^2}\right\}^{1/4} \quad \text{(Equation 11)}$$

In order to compute $\bar{\sigma}_M(x')/\sigma_o'$, the dilution, as a function of x'/x'_v we rearrange (11) and obtain

$$x'/x'_v = \frac{1 - \left(\frac{\sigma_M(x')}{\sigma_0}\right)^2 \sqrt{\frac{n^2}{m^2} \cdot R^2 + 1}}{\left(\frac{\bar{\sigma}_M(x')}{\sigma_0}\right)^2 \sqrt{\frac{n^2}{m^2} \cdot R^2 + 1}} \quad \text{(Equation 12)}$$

and tabulate as follows for R = 1/5 remembering that

$$\frac{n}{n+m} + \frac{m}{n+m} = 1$$

Dilution or $\bar{\sigma}_M(x')/\sigma_o$ is plotted as a function of $1 + x'/x'_v$ in Figure 3 from Table 1. It shows a decrease to the -1/2 power at early time and a decrease to the -l power at late time.

The centerline trajectory may be defined by the following equation

$$\frac{d(x'/x'_v)}{Q_M(x')} = \frac{d(x/x'_v)}{U_M(x')} = \frac{d(y/x'_v)}{V_M(x')} \quad \text{(Equation 13)}$$

or

$$\Delta(x/x'_v) = \frac{U_M(x')}{Q_M(x')}\Delta(x'/x'_v) \quad \text{(Equation 14)}$$

and

$$\Delta(y/x'_v) = \frac{V_M(x')}{Q_M(x')}\Delta(x'/x'_v) \quad \text{(Equation 15)}$$

That is, the trajectory may be computed by computing $\sigma_M(x')$, $V_M(x')$, and $Q_M(x')$ from Figure 3 and *equations (7)*, *(9)*, and *(10)* and substituting the values thus obtained in *equations (14)* and *(15)* to obtain x/x'_v and y/x'_v. This computation has been made for $R = \frac{U_0}{Q_0} = 1/5$ and is shown in Table 2.

The results of this computation are shown as Figure 9 of Appendix 2B.

Table 1

$\frac{\bar{\sigma}_M(x')}{\sigma_0}$	$\frac{m}{n+m}$	$\frac{n}{n+m}$	$\left(\frac{n}{m}\right)^2$	R^2	$\sqrt{\frac{n^2}{m^2} - R^2 + 1}$	x'/x'_v
1	1	0	0	1/25 = 0.04	1	0
0.9	0.9	0.1	0.0124	"	1	0.235
0.8	0.8	0.2	0.0625	"	1	0.562
0.7	0.7	0.3	0.184	"	1.003	1.003
0.6	0.6	0.4	0.445	"	1.009	1.756
0.5	0.5	0.5	1	"	1.02	2.920
0.4	0.4	0.6	2.250	"	1.043	5.00
0.3	0.3	0.7	5.44	"	1.101	9.10
0.2	0.2	0.8	16	"	1.280	18.50
0.1	0.1	0.9	81	"	2.06	47.5
0.05	0.05	0.95	361	"	3.92	101.0
0.02	0.02	0.98	2401	"	9.88	252

Table 2

TABLE 2

$\frac{x'}{x'_v}$	$\Delta\frac{x'}{x'_v}$	$\frac{\bar{\sigma}_M(x')}{\sigma_o}$	$\frac{V_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_o}$	$\frac{Q_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_M(x')}$	$\overline{\frac{U_M(x')}{Q_M(x')}}$	$\Delta\frac{x}{x'_v}$	$\frac{x}{x'_v}$	$\frac{V_M(x')}{Q_M(x')}$	$\overline{\frac{V_M(x')}{Q_M(x')}}$	$\Delta\frac{y}{x'_v}$	$\frac{y}{x'_v}$
0		1	1	0	1	0			0	1			0
	1						0.043	0.043			1.000	1.000	
1		0.70	0.70	0.060	0.70	0.086			0.043	1			1.000
	1						0.116	0.116			0.992	0.992	
2		0.575	0.575	0.085	0.584	0.146			0.159	0.984			1.992
	1						0.171	0.171			0.982	0.982	
3		0.500	0.500	0.100	0.51	0.196			0.330	0.980			2.974
	1						0.222	0.222			0.975	0.975	
4		0.44	0.44	0.112	0.454	0.247			0.552	0.970			3.949
	1						0.268	0.268			0.964	0.964	
5		0.40	0.40	0.120	0.418	0.288			0.820	0.958			4.913
	1						0.308	0.308			0.952	0.952	
6		0.366	0.366	0.127	0.388	0.328			1.128	0.945			5.865
	1						0.344	0.344			0.938	0.938	
7		0.341	0.341	0.132	0.366	0.360			1.472	0.931			6.803
	1						0.376	0.376			0.926	0.926	
8		0.320	0.320	0.136	0.348	0.391			1.848	0.920			7.729
	1						0.405	0.405			0.915	0.915	
9		0.304	0.304	0.139	0.334	0.418			2.253	0.910			8.644
	1						0.434	0.434			0.901	0.901	

TABLE 2 (Cont 'd)

$\frac{x'}{x'_v}$	$\Delta\frac{x'}{x'_v}$	$\frac{\bar{\sigma}_M(x')}{\sigma_o}$	$\frac{V_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_o}$	$\frac{Q_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_M(x')}$	$\overline{\frac{U_M(x')}{Q_M(x')}}$	$\Delta\frac{x}{x'_v}$	$\frac{x}{x'_v}$	$\frac{V_M(x')}{Q_M(x')}$	$\overline{\frac{V_M(x')}{Q_M(x')}}$	$\Delta\frac{y}{x'_v}$	$\frac{y}{x'_v}$
10		0.285	0.285	0.143	0.319	0.450			2.687	0.893			9.545
	2						0.473	0.946			0.880	1.760	
12		0.258	0.258	0.148	0.298	0.496			3.633	0.866			11.305
	2						0.520	1.040			0.853	1.706	
14		0.236	0.236	0.153	0.281	0.543			4.673	0.840			13.011
	2						0.562	1.124			0.828	1.656	
16		0.219	0.219	0.156	0.269	0.580			5.797	0.815			14.667
	2						0.598	1.196			0.802	1.604	
18		0.203	0.203	0.159	0.258	0.616			6.993	0.788			16.271
	2						0.632	1.232			0.774	1.548	
20		0.190	0.190	0.162	0.250	0.648			8.225	0.760			17.819
	5						0.684	3.420			0.728	3.640	
25		0.162	0.162	0.168	0.233	0.720			11.645	0.696			21.459
	5						0.743	3.715			0.668	3.340	
30		0.143	0.143	0.171	0.223	0.766			15.360	0.640			24.799
	5						0.784	3.920			0.616	3.080	
35		0.129	0.129	0.174	0.217	0.801			19.280	0.593			27.879
	5						0.825	4.125			0.563	2.815	
40		0.112	0.112	0.178	0.210	0.848			23.405	0.532			30.694
	10						0.869	8.690			0.495	4.950	

TABLE 2 (Cont'd)

$\frac{x'}{x'_v}$	$\Delta\frac{x'}{x'_v}$	$\frac{\bar{\sigma}_M(x')}{\sigma_o}$	$\frac{V_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_o}$	$\frac{Q_M(x')}{Q_o}$	$\frac{U_M(x')}{Q_M(x')}$	$\overline{\frac{U_M(x')}{Q_M(x')}}$	$\Delta\frac{x}{x'_v}$	$\frac{x}{x'_v}$	$\frac{V_M(x')}{Q_M(x')}$	$\frac{V_M(x')}{Q_M(x')}$	$\Delta\frac{y}{x'_v}$	$\frac{y}{x'_v}$
50		0.093	0.093	0.181	0.208	0.890			32.095	0.457			35.644
	10						0.903	9.030			0.424	4.240	
60		0.079	0.079	0.184	0.201	0.916			41.125	0.391			39.884
	20						0.933	18.660			0.347	6.980	
80		0.061	0.061	0.188	0.198	0.950			59.785	0.307			46.864
	20						0.960	19.200			0.281	5.620	
100		0.050	0.050	0.190	0.196	0.970			78.985	0.255			52.484
	50						0.977	48.850			0.238	11.900	
150		0.0335	0.0335	0.193	0.196	0.984			127.835	0.171			64.384
	50						0.987	49.350			0.150	7.500	71.884
200		0.0251	0.0251	0.195	0.197	0.990			177.185	0.128			
	50						0.993	49.650			0.115	5.750	77.634
250		0.0201	0.0201	0.196	0.197	0.995			226.835	0.102			
	50						0.995	49.750			0.094	4.700	82.334
300		0.0169	0.0169	0.197	0.198	0.995			276.585	0.085			

APPENDIX 2C

REPORT, SUPPLEMENTARY FOUNDATION STUDIES, PROPOSED BROOKWOOD NUCLEAR POWER PLANT (R. E. GINNA NUCLEAR POWER PLANT), ONTARIO, NEW YORK

EARTH SCIENCES

DAMES & MOORE
CONSULTANTS IN APPLIED EARTH SCIENCES
SOIL MECHANICS · ENGINEERING GEOLOGY · GEOPHYSICS

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ASSOCIATE: FRANCIS E. RANFT

June 2, 1966

Gilbert Associates, Incorporated
Engineers and Consultants
525 Lancaster Avenue
Reading, Pennsylvania 19603

Attention: Mr. Hans Lorenz

Gentlemen:

We submit herewith ten copies of our "Report, Supplementary Foundation Studies, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation."

The scope of our studies was planned in cooperation with Mr. D. K. Croneberger of Gilbert Associates, Incorporated. Our preliminary conclusions were transmitted verbally to Messrs. Croneberger and H. Lorenz during the course of our studies.

Yours very truly,

DAMES & MOORE

Robert M Perry

RMP:ts

Robert M. Perry, P.E.

2C.1 INTRODUCTION

2C.1.1 GENERAL

This report presents the results of our supplementary foundation studies for the proposed Brookwood Nuclear Power Plant presently under construction near Ontario, New York, for the Rochester Gas and Electric Corporation. Detailed information relative to environmental conditions, site and subsurface features, and general foundation recommendations are presented in our report[a] dated June 14, 1965.

2C.1.2 PURPOSE

The purpose of our supplementary studies was to:

1. recommend specific bearing pressures for use in the design of foundations supported by the natural compact granular soils, compacted granular fill and sound bedrock;
2. present more detailed information on the depths at which the compact natural granular soils and the bedrock are encountered;
3. further explore the condition of the bedrock in the reactor area; and
4. evaluate the effects of the dynamic load imposed by the turbine-generator on the soil-foundation system.

2C.1.3 SCOPE OF WORK

The field phase of our supplementary studies consisted of drilling seven test borings. Two of the borings were drilled in the reactor area and extended 50 feet into the bedrock. The remaining five borings were terminated when bedrock was encountered. Undisturbed soil samples, suitable for laboratory testing, were extracted from each test boring. Rock cores were recovered from the two borings in the reactor area.

The locations of the borings drilled for these studies are shown in relation to the proposed construction and previously drilled borings on the Plot Plan, Plate 1. The field explorations were performed under the technical direction of a Dames & Moore Engineering Geologist.

The results of the field explorations and laboratory tests, which provide the basis for our engineering analyses and recommendations, are presented in the Appendix to this report.

2C.1.4 SITE CONDITIONS

The plant will be located in a relatively level meadow area with surface elevations[b] on the order of +275 feet. Grading operations were underway during our field explorations.

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a. "Report, Site Evaluation Study, Proposed Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation"

b. All elevations presented in this report refer to United States Coast and Geodetic Survey Datum.

The subsurface conditions encountered in the borings drilled during this investigation are similar to those previously encountered in the plant area. In general, the plant area is underlain by four basically different types of material. These are, in order of increasing depth:

1. firm brown surficial silty and clayey soils;
2. soft gray silty clay;
3. compact sandy and gravelly soils; and
4. bedrock.

Detailed descriptions of the materials encountered in the plant area are shown on the boring logs presented in the Appendix. In general, the compact granular soils were encountered at depths ranging from about five feet to 35 feet below the original ground surface. Bedrock generally was observed at depths ranging from about 34 feet to 40 feet below the surface. The southwest corner of the proposed plant revealed bedrock at somewhat shallower depths.

Contours of the surface of the compact granular soils and the underlying bedrock are presented on the Plot Plan. This contour map was prepared by interpolation between borings. Consequently, local variations may occur between the boring locations which are not indicated by the contours.

2C.2 DISCUSSION AND RECOMMENDATIONS

2C.2.1 GENERAL

It is understood that foundations for the major plant facilities will be installed at depths of 25 or more feet below the original ground surface. In our prior report, we recommended that spread or mat foundations be installed on the natural compact granular soil, compacted granular backfill or sound bedrock.

Spread and mat foundation installation and design criteria are presented in subsequent sections of this report. The results of our analyses evaluating the effects of the turbine generator on the soil foundation system are presented in the final section of this report.

2C.2.2 FOUNDATION INSTALLATION PROCEDURES

Natural Soils: Spread or mat foundations can be installed directly on the compact granular soils at elevations below those indicated by the contours on the Plot Plan. We recommend that the sand and gravel at foundation depth be proof rolled with heavy pneumatic-tired equipment. The proof rolling will recompact soils which are disturbed during excavation operations. Any local pockets of loose or soft material requiring additional excavation also will be revealed by the proof rolling operations. Soils removed below proposed foundation grade should be replaced with compacted structural fill or lean concrete.

Compacted Backfill: Foundations which are to be installed above the elevation of the surface of the natural granular soils should be supported by compacted granular backfill placed after the clayey soils are removed. Prior to placing the backfill, the exposed underlying natural granular soil should be proof rolled. The structural fill then should be placed in layers approximately eight inches in thickness. Each layer should be compacted to a density of at least 95 percent of the maximum density obtainable by the Modified AASHO[a] Method of Compaction, Test Designation T180-57. We suggest that large vibratory or heavy pneumatic tired equipment be used to compact the granular backfill soils.

We believe that most of the natural granular soils excavated in the plant area below the elevations indicated on Plate 1 can be reused as back fill. The upper silty and clayey soils should not be used as structural fill.

It will be necessary to dewater all deep excavations. Information regarding ground water levels and soil permeability was presented in our previous report. We recommend that adequate dewatering measures be taken prior to final excavation and that the dewatering be continuously maintained during:

1. final excavation;
2. proof rolling operations;
3. placement of structural backfill;
4. foundation installation; and

---

a. American Association of State Highway Officials

5. general backfilling operations.

We recommend that an experienced Soils Engineer be present during site preparation in order to inspect the excavation and proof rolling operations and to technically supervise the placement of structural backfill.

2C.2.3 FOUNDATION DESIGN CRITERIA

Soil: Based upon the results of our field explorations and laboratory tests, we recommend that spread and mat foundations be designed utilizing the net bearing pressures presented on Plate 2, Foundation Design Data. The bearing pressures presented on Plate 2 are applicable for the compact natural granular soil and structural granular fill compacted in accordance with our aforementioned recommendations. The recommended bearing pressures apply to the total of all design loads, dead and live. The term "net bearing pressures" refers to the foundation pressure that can be imposed in excess of the lowest adjacent overburden pressure. The recommended bearing pressures apply to foundations at least ten feet in width.

We recommend that the maximum net bearing pressures imposed on the natural compact soils and the compacted structural fill should be limited to 10,000 and 8,000 pounds per square foot, respectively. Although, from a stability standpoint, greater bearing pressures could be used in the design of large spread and mat foundations, we recommend that these limiting vales be maintained in order to restrict foundation movements to small elastic deformations.

Rock: We recommend that foundations installed on the underlying sound rock be designed utilizing a bearing pressure not in excess of 35 tons per square foot. This pressure applies to the total of all design loads, dead and live. It is possible that weathered rock may be encountered at the soil-rock interface. Our field explorations indicate that the weathered zone is relatively thin, generally less than one to two feet in thickness.

We understand that the bedrock in the reactor area will be required to provide resistance to lateral forces. We believe that a lateral resistance of 25,000 pounds per square foot of vertical contact area can be relied upon in the sound rock. This lateral resistance applies only to foundations poured in "neat" excavations directly against the exposed rock faces. The 25,000 pounds per square foot value does not take into account the additional resistance which would be provided by any adjacent overburden above the surface of the bedrock.

The exposed bedrock should be inspected by a qualified Engineering Geologist in order to examine the condition of the foundation material and to check for any unusual or unanticipated joint patterns.

2C.2.4 TURBINE-GENERATOR FOUNDATION

The turbine generator will be supported on a mat foundation approximately 40 feet by 150 feet in plan dimensions. The base of the mat will be installed at approximately Elevation +243 feet, some four to seven feet above the rock surface. The center-line of the turbine generator will be approximately 50 feet above the base of the mat foundation. The dead weight of the equipment and the foundation will impose a pressure of about 4,000 pounds per square foot on the foundation soils.

We understand that the turbine generator will operate at approximately 1,800 revolutions per minute. During start-up and operation, an unbalanced moment on the order of 2,000,000 foot-pounds will be transmitted to the soils at the base of the mat. This moment is a steady-state condition and does not vary with the operating speed. Unbalanced dynamic forces will be negligible. A torque approximately ten times the operating torque will result from a short-circuit load. This, short-circuit torque will be balanced within the equipment foundation and will not be transmitted to the foundation soil.

Our analyses indicate that the deflection resulting from the unbalanced moment will be on the order of 0.004 inches at the edge of the unit. We believe that there will be no influence from any small unbalance in the equipment since the operating frequency is well above the resonant frequency of the soil-foundation system.

The following Plates and Appendix are attached and complete this report:

Plate 1 -	Plot Plan
Plate 2 -	Foundation Design Data
Appendix -	Field Explorations and Laboratory Tests

Respectfully submitted,

DAMES & MOORE

Robert M. Perry

State of New York

P.E. Registration No. 35284

Arthur Rothman

RMP-AR: ts

2C.3 REPORT APPENDIX - FIELD EXPLORATIONS AND LABORATORY TESTS

2C.3.1 FIELD EXPLORATIONS

The subsurface conditions in the plant area were explored during this investigation by drilling 7 supplementary test borings to depths ranging from 35 feet to 90 feet below the ground surface. The locations of the borings are shown on the Plot Plan. The field exploration program was conducted under the technical direction of a Dames & Moore Engineering Geologist.
The borings were drilled approximately four inches in diameter utilizing truck-mounted rotary drilling equipment. Driller's mud was used where necessary to prevent the walls of the borings from caving.

Continuous observations of the materials encountered in the borings were recorded in the field during drilling operations. Undisturbed soil samples, suitable for laboratory testing, were extracted from the borings utilizing the Dames & Moore sampler illustrated in Figure 3 of this Appendix. The sampler is three and one-quarter inches in outside diameter and approximately two and one-half inches in inside diameter. Rock cores were obtained from the two test borings in the reactor area to a depth of 50 feet below the rock surface utilizing a Series NX core barrel. The cores recovered are two and one-eight inches in diameter. The soil samples and rock cores were shipped to our New York office and laboratory where they were further examined and subjected to appropriate laboratory tests.

Detailed descriptions of the soils and rock encountered in the borings are presented on Plates A-1A and A-lB, Log of Borings. The soils were classified in accordance with the Unified Soil Classification System described on Plate A-2.

The number of blows required to drive the sampler a distance of one foot into the soil utilizing a 500-pound drive weight, falling a distance of 18 inches is presented in the column at the left of the log of each boring. The percent of core recovery obtained during coring operations is also presented in this column.

The elevations which appear at the top of each boring log refer to United States Coast and Geodetic Survey Datum and were determined by representatives of Rochester Gas and Electric Company.

2C.3.2 LABORATORY TESTS

Soil: A number of undisturbed samples of the natural compact granular soils were tested to evaluate their strength characteristics. Triaxial compression tests were performed on the soil samples in the manner described in Figure 4. In addition to the tests on samples of the natural undisturbed soils, triaxial compression tests were performed on samples of remolded and recompacted granular material. These tests were used in our compacted fill studies to evaluate the variation in strength characteristics with changes in density.

A load-deflection curve was plotted for each strength test and the shearing strength of the soil was determined from this curve. Determinations of the moisture content and dry density of the soils were made in conjunction with each strength test.

The results of the strength tests and the corresponding moisture and density determinations are tabulated in Table 1. Summary of Soil Strength Test Data.

Rock: Unconfined compression, triaxial compression and tension tests were performed on selected rock cores extracted from the borings. These tests were performed by subjecting rock cores approximately two and one-eight inches in diameter and four to six inches in height to an axial strain and recording the resisting stress developed by the rock. A stress-strain curve was plotted for each of the compression tests and the shearing strength of the rock was determined from this curve. The results of the strength tests on the rock cores are presented below:

BORING	DEPTH (feet)	CELL PRESSURE (psi)	ONE-HALF DEVIATOR STRESS (psi)	TYPE OF TEST
201	42	-	50[a]	Tension
201	45	-	50[a]	Tension
201	49	1,000	4,700	Triaxial Compression
202	47	-	3,900	Unconfined Compression
202	$50^{1/2}$	1,500	4,400	Triaxial Compression

a. Indicates peak tensile stress normal to bedding planes.

The following Plates are attached and complete this Appendix:

Plate A-lA - Log of Borings (Borings 201 and 202)

Plate A-1B - Log of Borings (Borings 203 through 207)

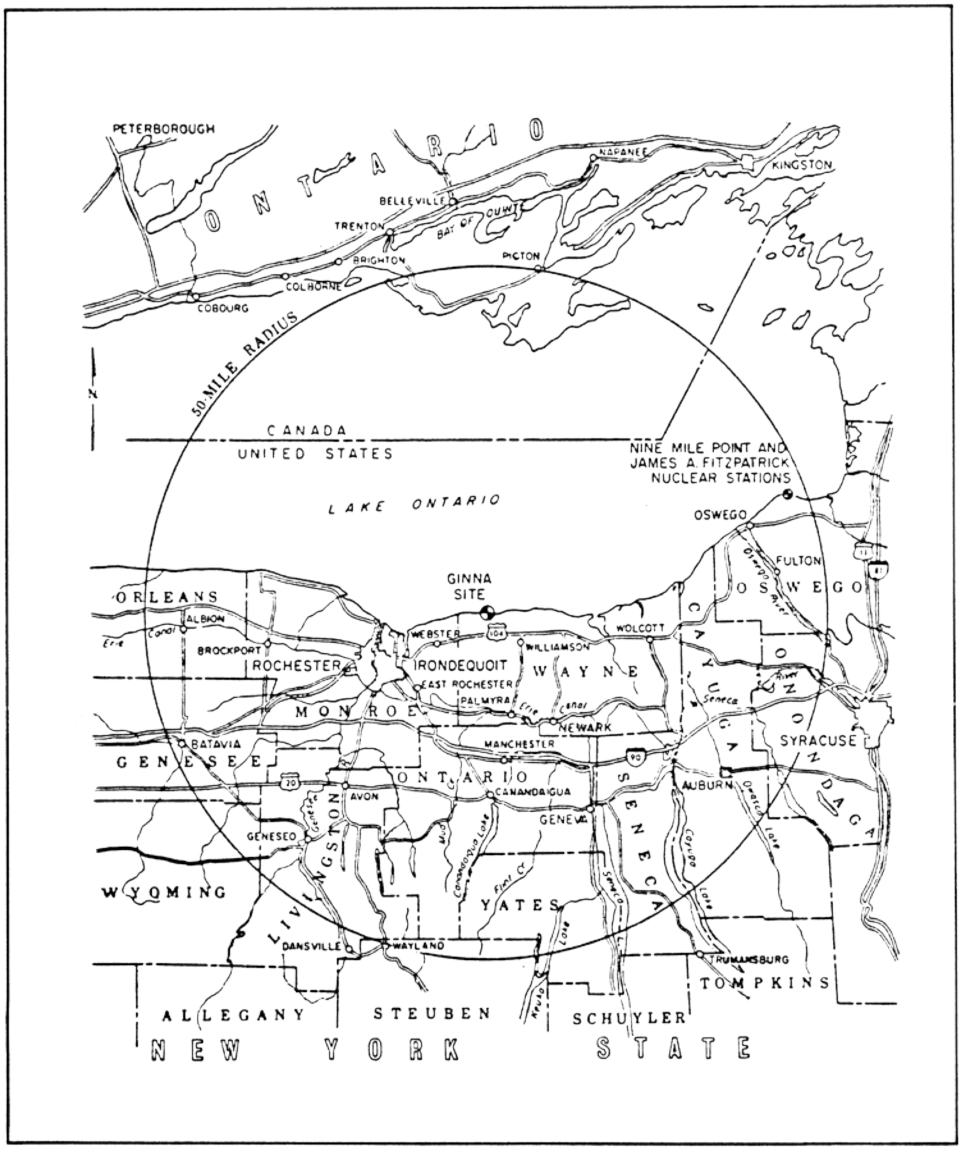
Plate A-2 - Unified Soil Classification System and Key to Test Data

Table 1
SUMMARY OF SOIL STRENGTH TEST DATA

BORING	DEPTH (feet)	DRY DENSITY (pcf)	MOISTURE CONTENT (percent)	CELL PRESSURE (psf)	ONE-HALF DEVIATOR STRESS (psf)	REMARKS
202	$30^{1/2}$	114	11.2	1,500	3,900	Natural
		110	11.0	1,500	2,100	Recompacted
				2,000	2,900	Recompacted
				3,000	4,400	Recompacted
		115	10.6	1,500	3,300	Recompacted
				2,000	3,750	Recompacted
		127	10.5	1,500	4,150	Recompacted
203	$10^{1/2}$	117	10.8	500	1,400	Natural
				1,500	2,700	Natural
		124	11.2	500	2,700	Recompacted
				1,500	4,300	Recompacted
203	$15^{1/2}$	120	12.1	1,500	1,600	Natural
				3,000	3,800	Natural
				6,000	8,300	Natural
		111	11.5	500	800	Recompacted
				1,000	1,800	Recompacted
				3,000	4,000	Recompacted
204	$20^{1/2}$	112	7.3	2,000	4,000	Natural
		111	7.6	2,000	3,500	Recompacted
205	$15^{1/2}$	125	11.6	1,000	3,000	Natural
		122	11.8	1,000	1,800	Recompacted

BORING	DEPTH (feet)	DRY DENSITY (pcf)	MOISTURE CONTENT (percent)	CELL PRESSURE (psf)	ONE-HALF DEVIATOR STRESS (psf)	REMARKS
207	16 1/2	144	6.5	2,000	5,200	Natural

Figure 2.1-1 Location of the R. E. Ginna Nuclear Power Plant



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.1-1 Location of the R. E. Ginna Nuclear Power Plant

Figure 2.1-2 R. E. Ginna Site

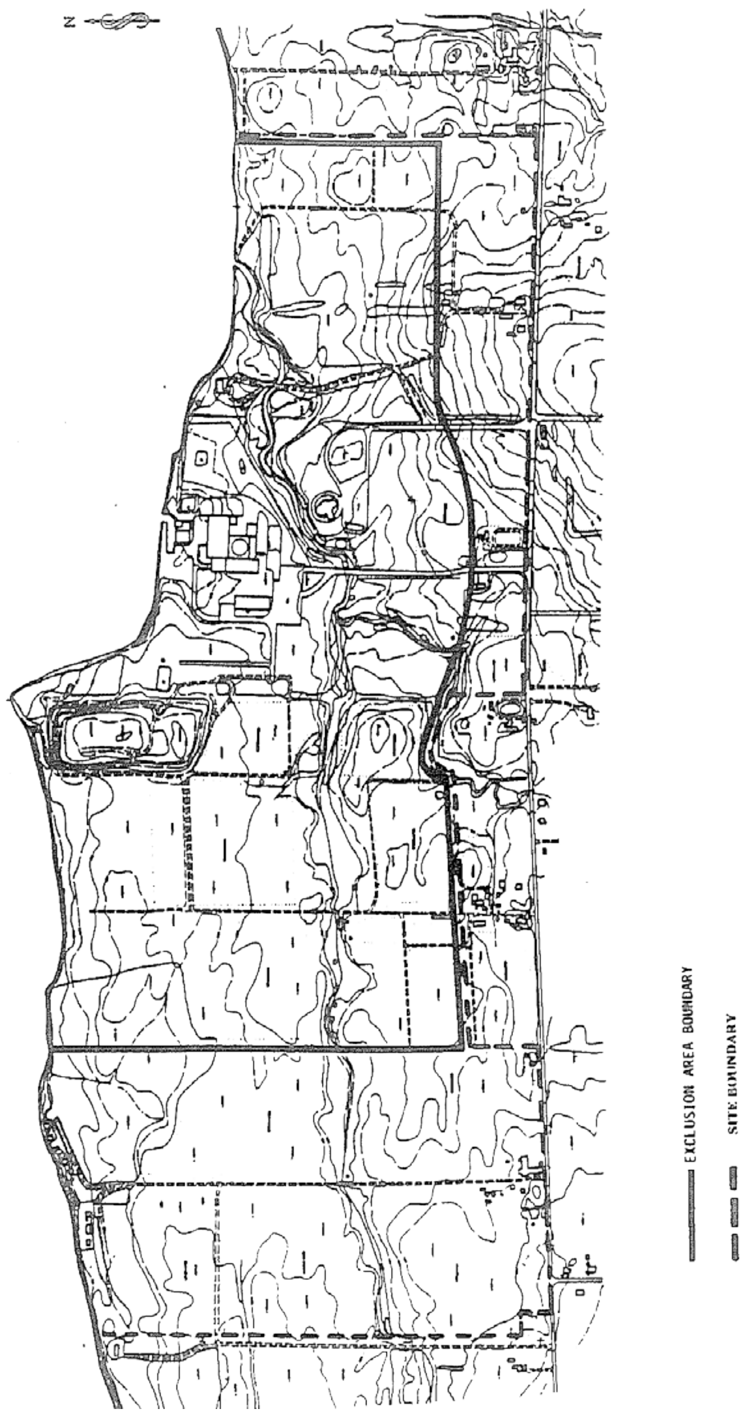


Figure 2.1-3 Figure Deleted

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Figure 2.1-4 Projection of Population Distribution 0-5 Miles

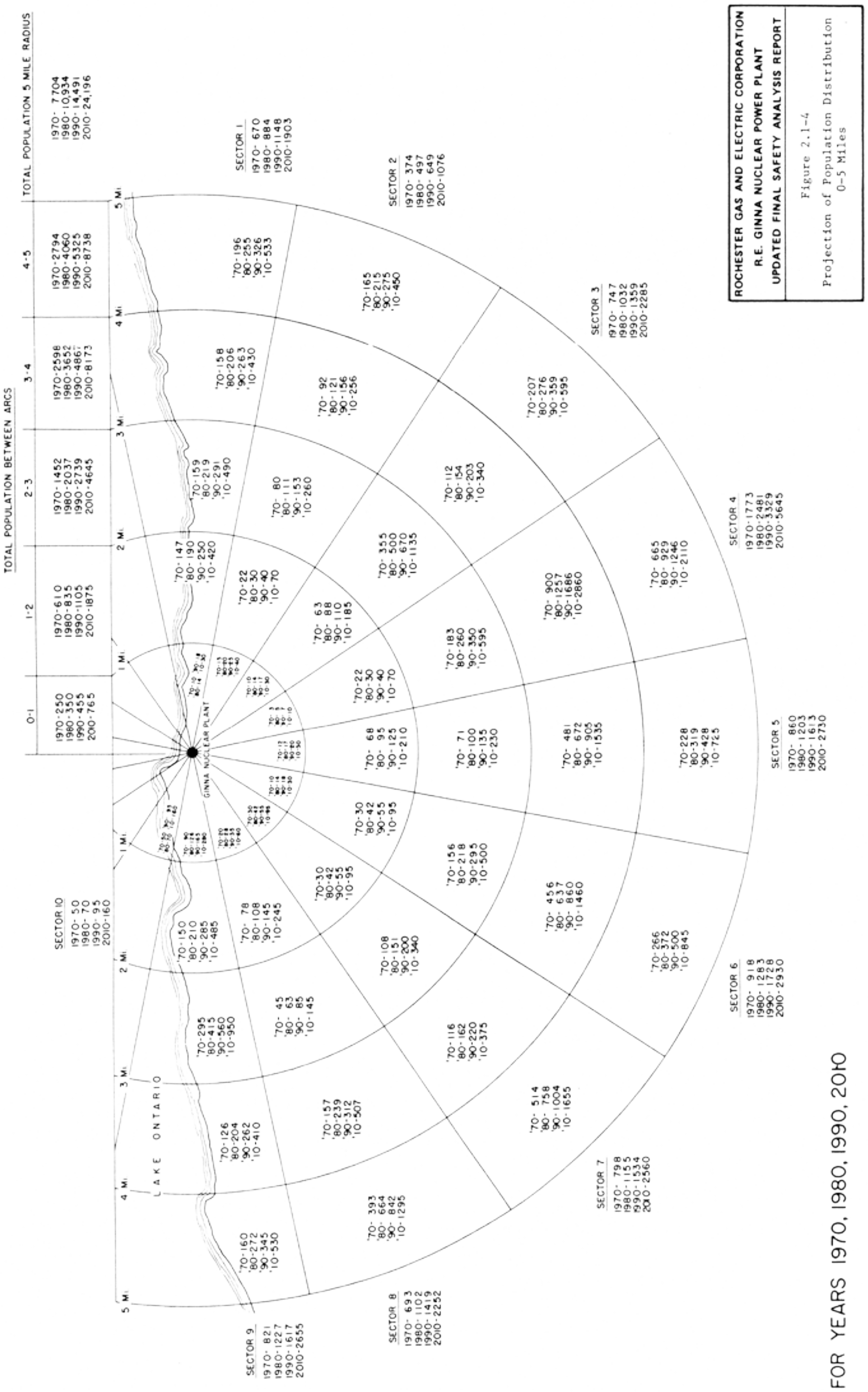
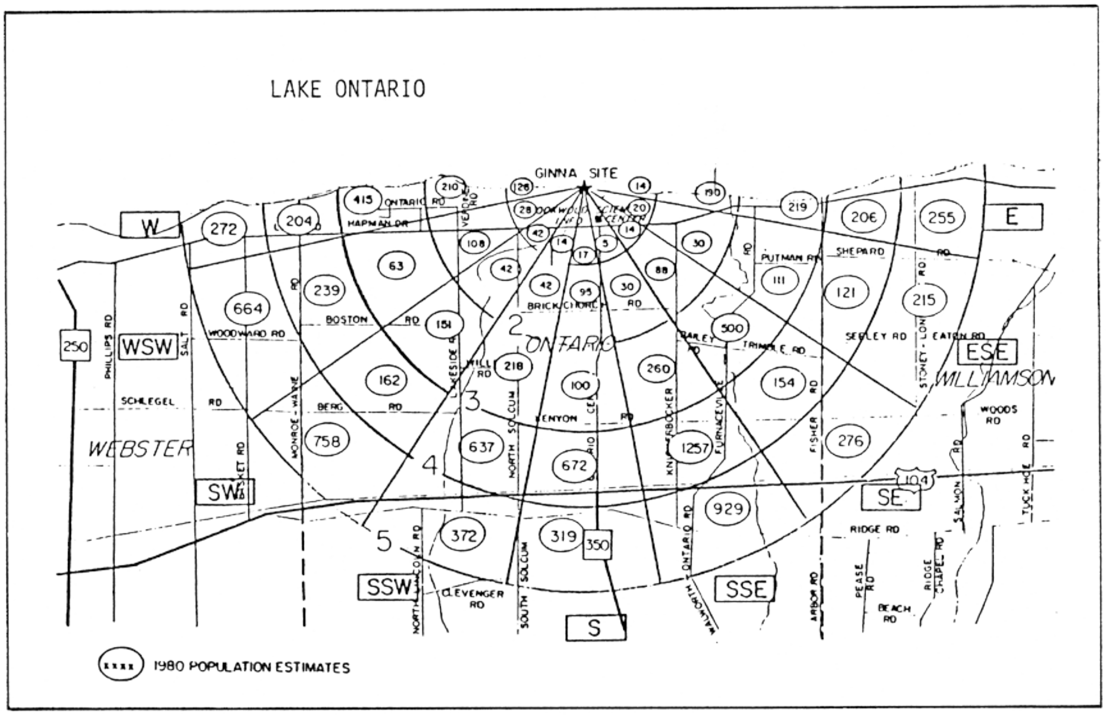


Figure 2.1-5 1980 Population Estimates 0-5 Miles



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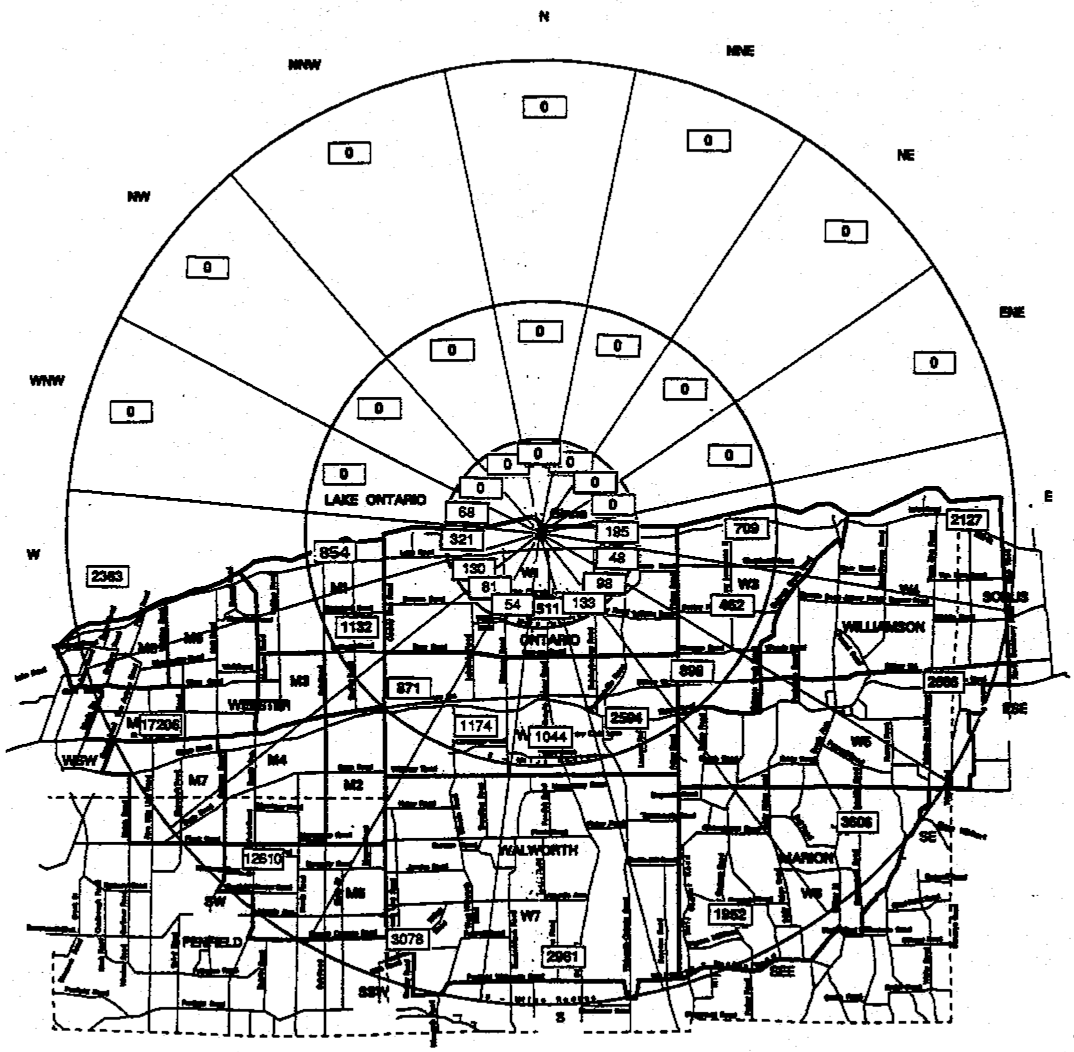
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 2.1-5

1980 Population Estimates
0-5 Miles

Figure 2.1-5a 1992 Population Estimates 0-10 Miles



Miles	Ring Population	Cumulative Population
0-2	1539	1539
2-5	9738	11277
5-10	48569	59846

BEST IMAGE AVAILABLE

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Figure 2.1-5a

1992 Population Estimates
0-10 Miles

REV 10 12/93

Figure 2.1-6 Projection of Population Distribution 0-40 Miles

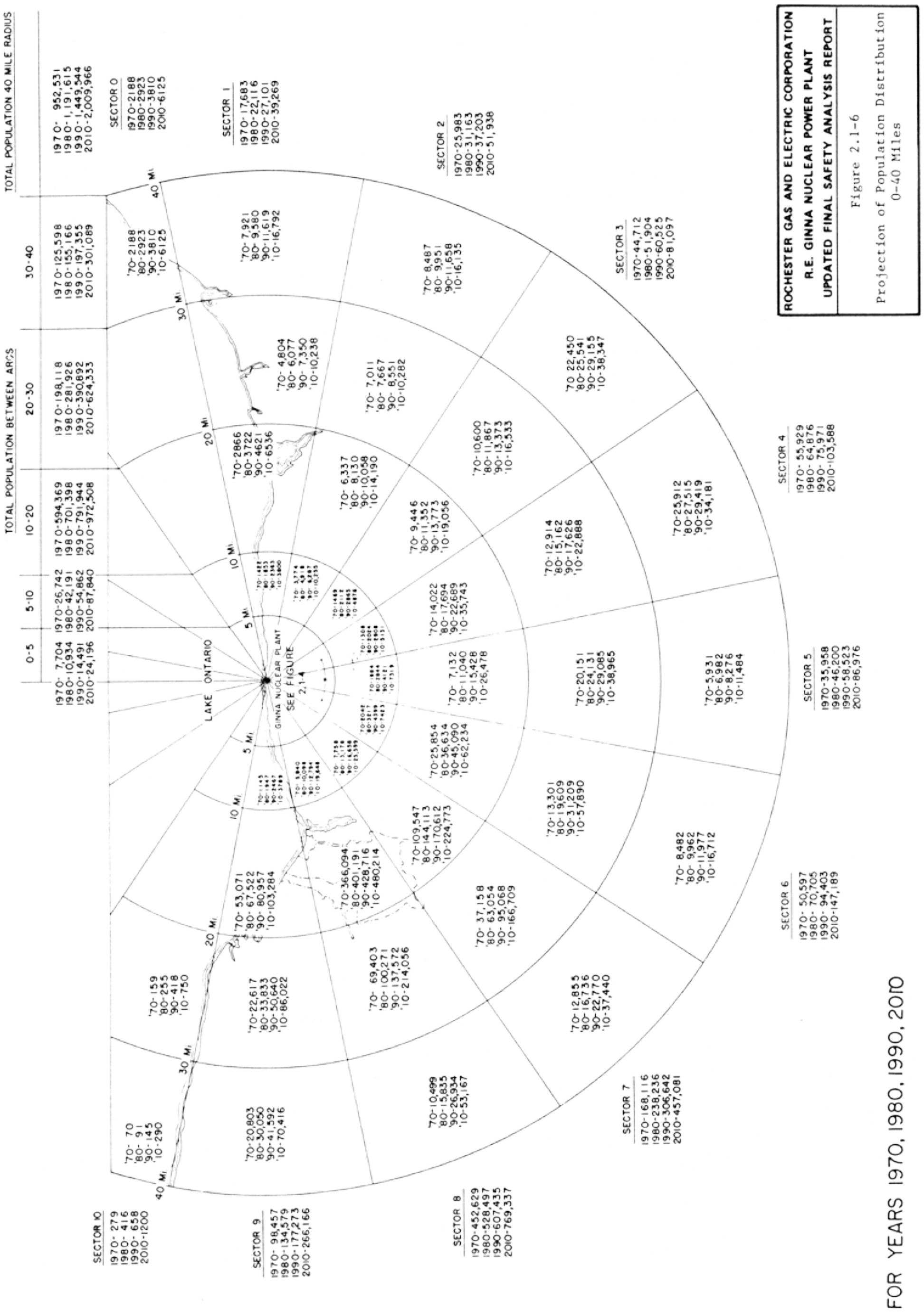
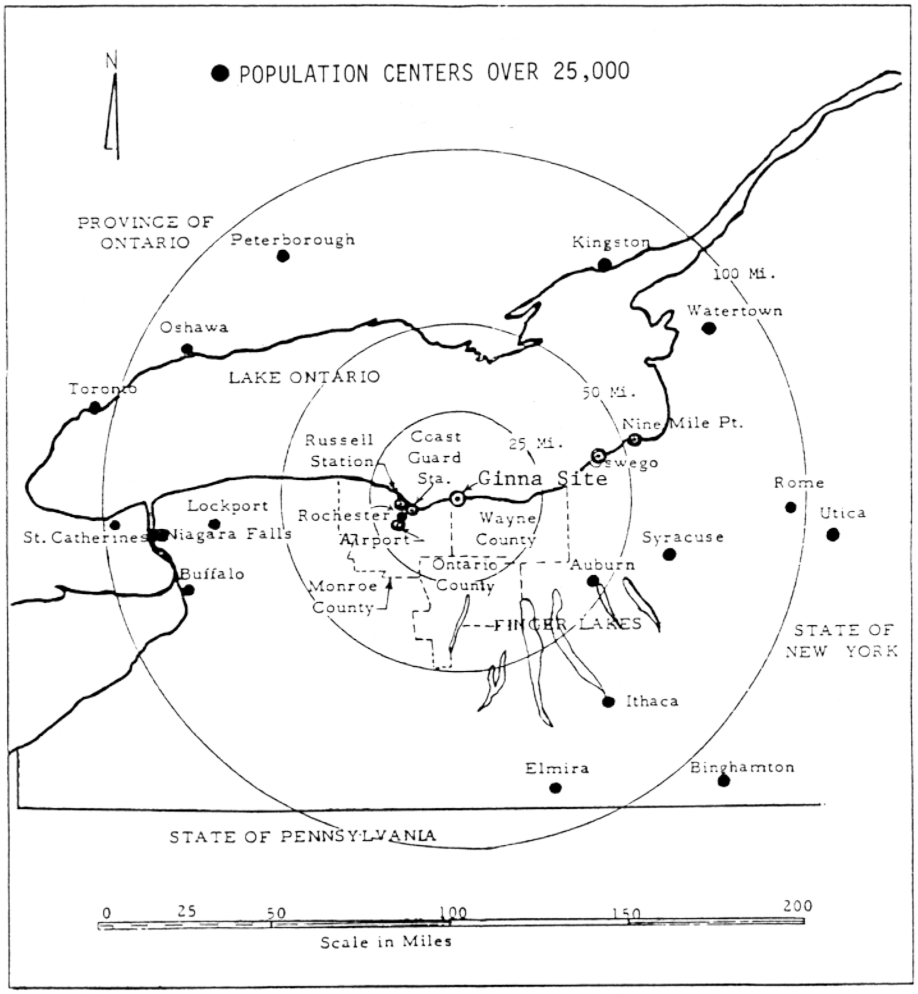


Figure 2.1-7 Location of Ginna Site



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.1-7 Location of Ginna Site

Figure 2.1-8 Population Centers Over 2000

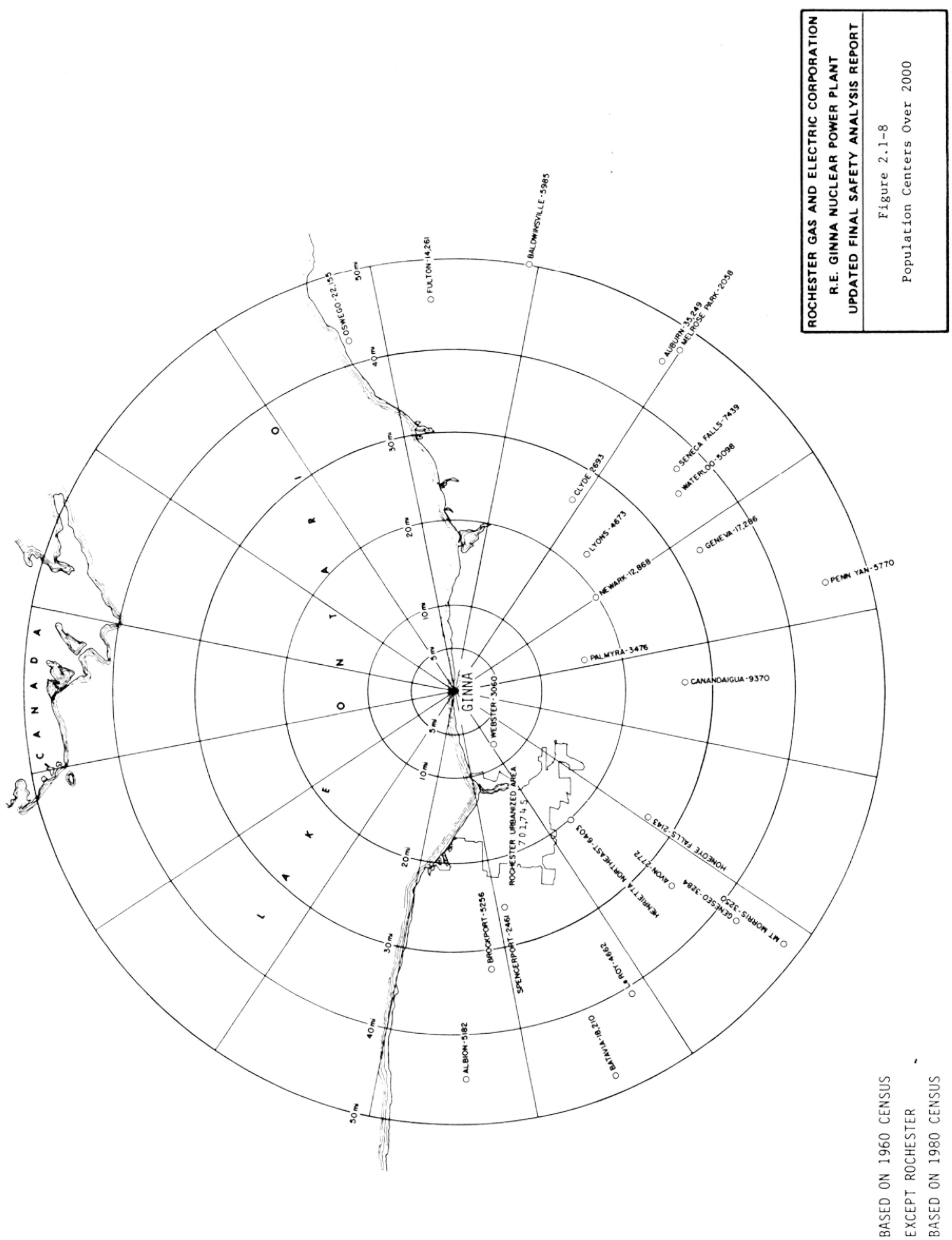
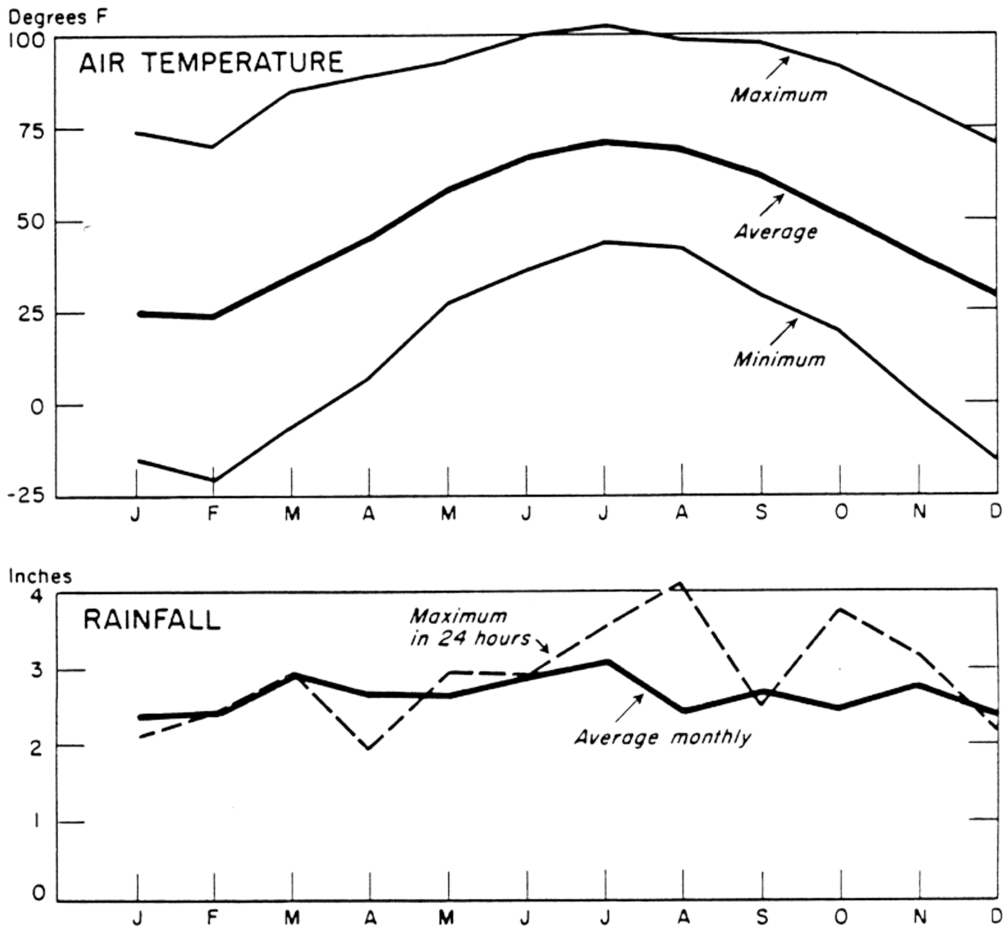


Figure 2.3-1 Climate of the Ginna Site Region



SKY COVER % (DAYTIME)

81	78	72	67	67	59	55	56	56	61	80	81

FASTEST WIND - Direction and mph

W 73	W 66	W 65	W 59	SW 63	SW 61	W 56	NE 59	SW 59	SW 55	E 59	W 51

From more than 30 years of record at Rochester Airport.

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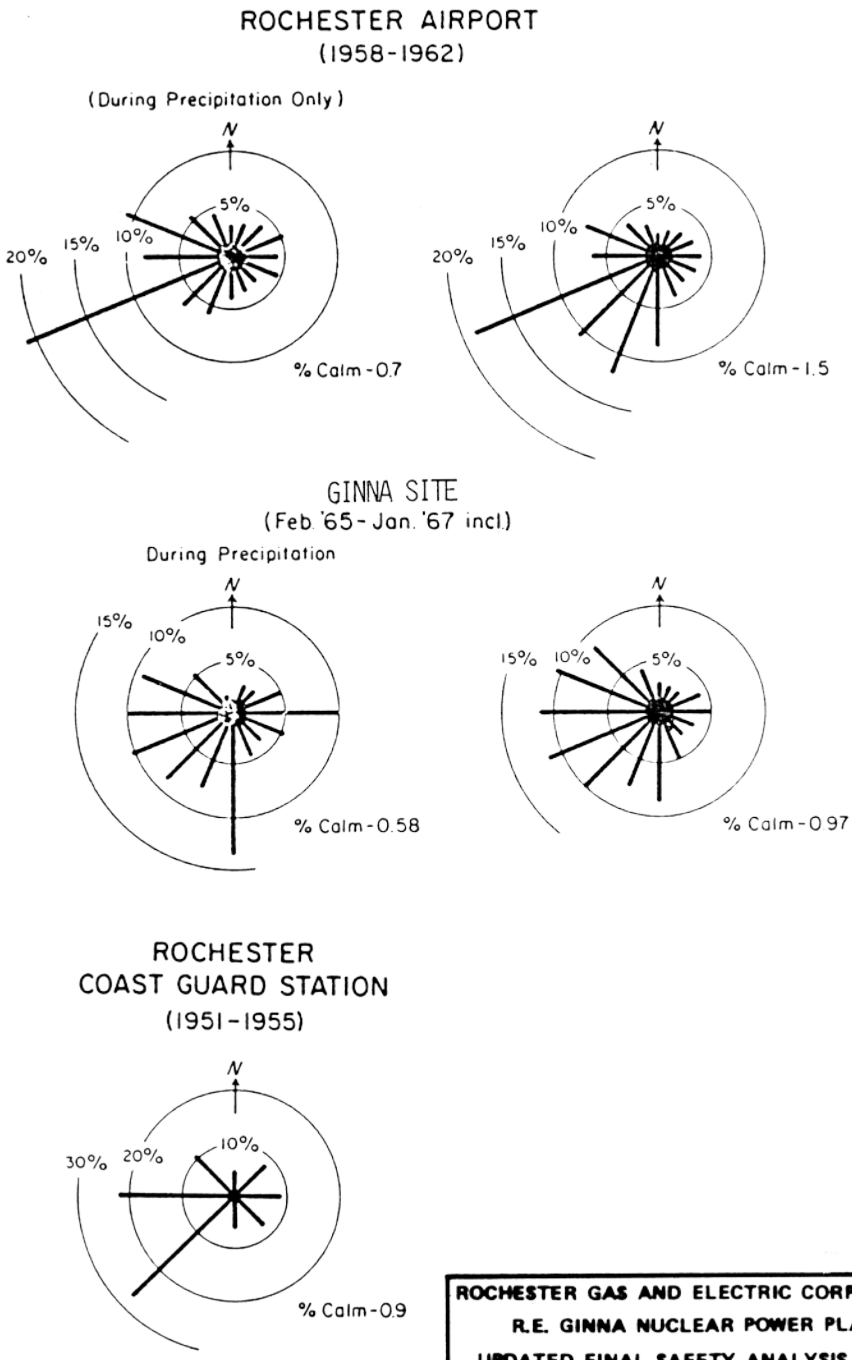
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 2.3-1
Climate of the Ginna Site Region

Figure 2.3-2 Wind Direction Patterns Long Period Averages



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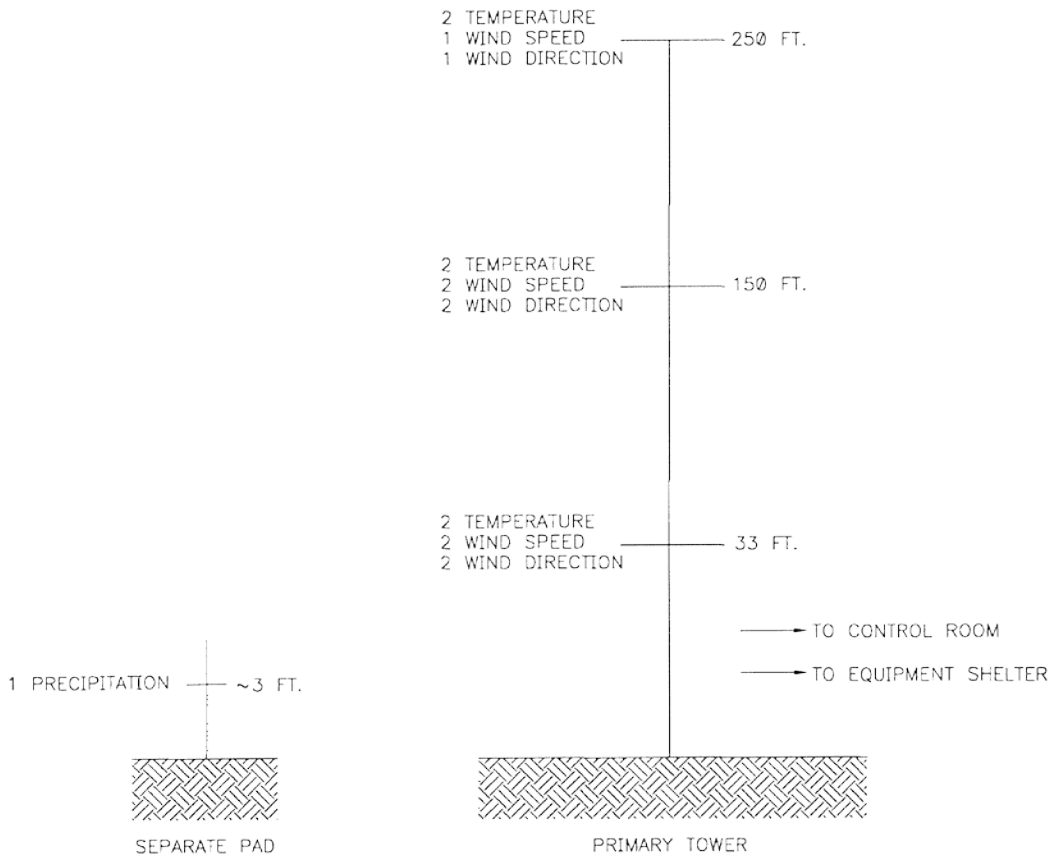
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 2.3-2
Wind Direction Patterns
Long Period Averages

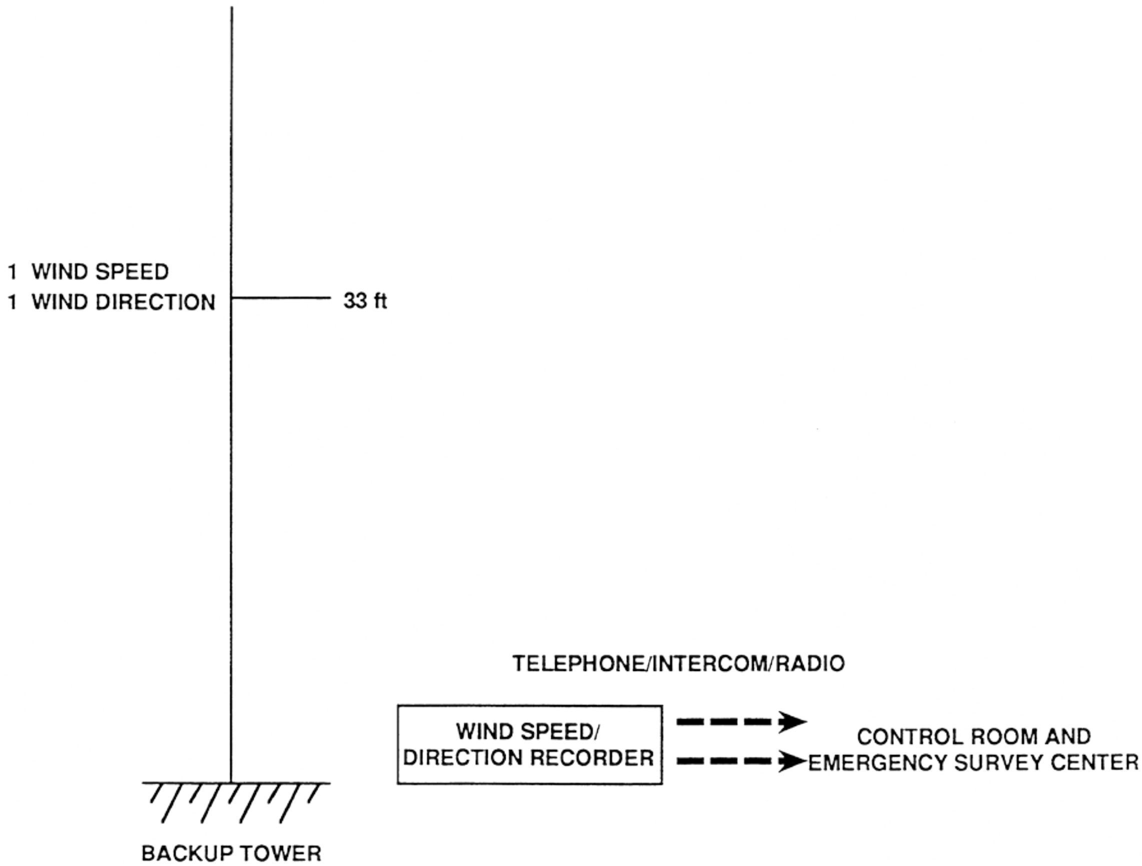
Figure 2.3-3 Sensor Placements, Primary Meteorological Tower



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-3 Sensor Placements, Primary Meteorological Tower

REV. 15 10/99

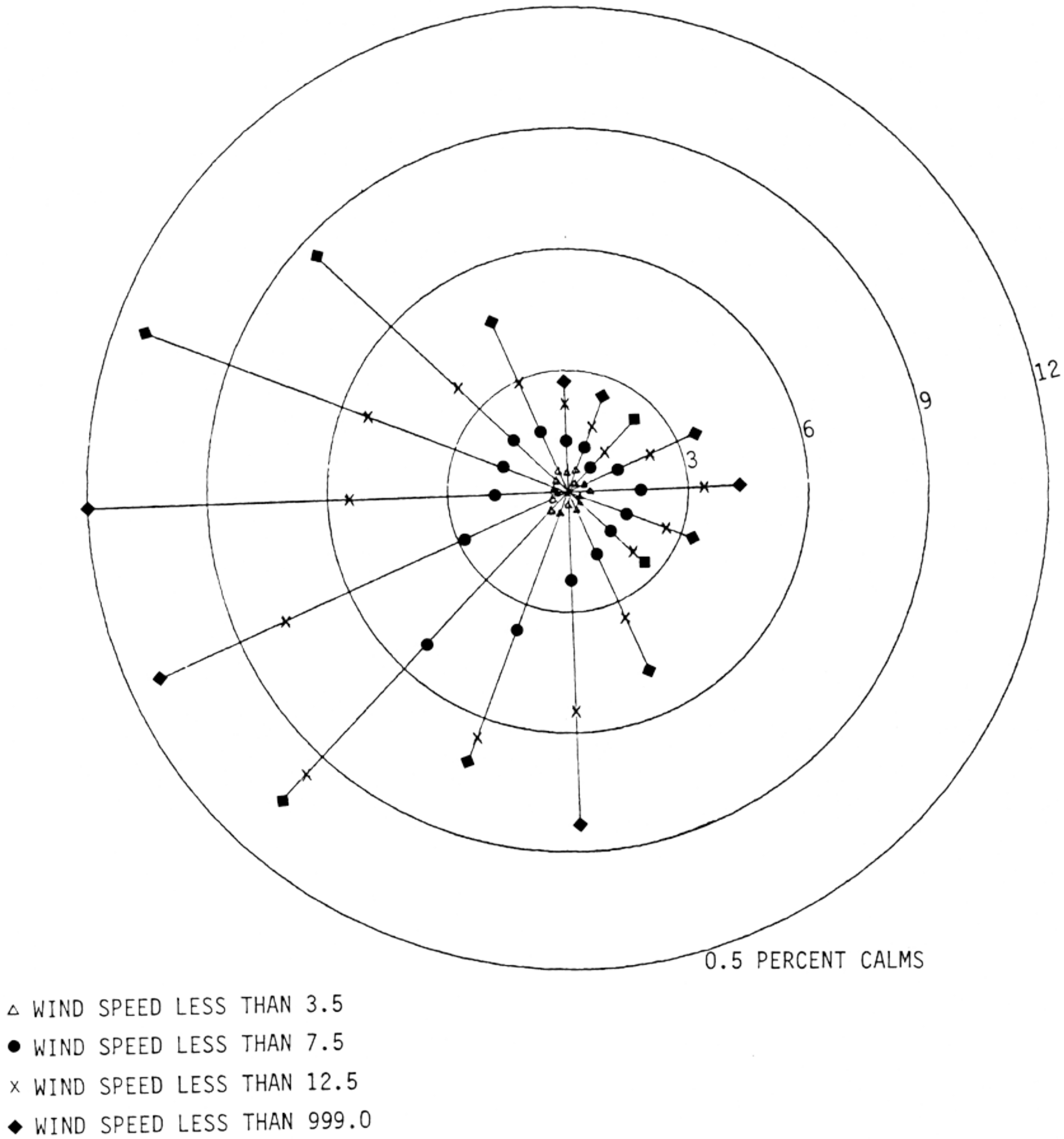
Figure 2.3-4 Sensor Placements, Backup (Substation 13A) Meteorological Tower



ROCHESTER GAS AND ELECTRIC CORPORATION R. E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-4 Sensor Placements, Backup (Substation 13A) Meteorological Tower

REV 10 12/93

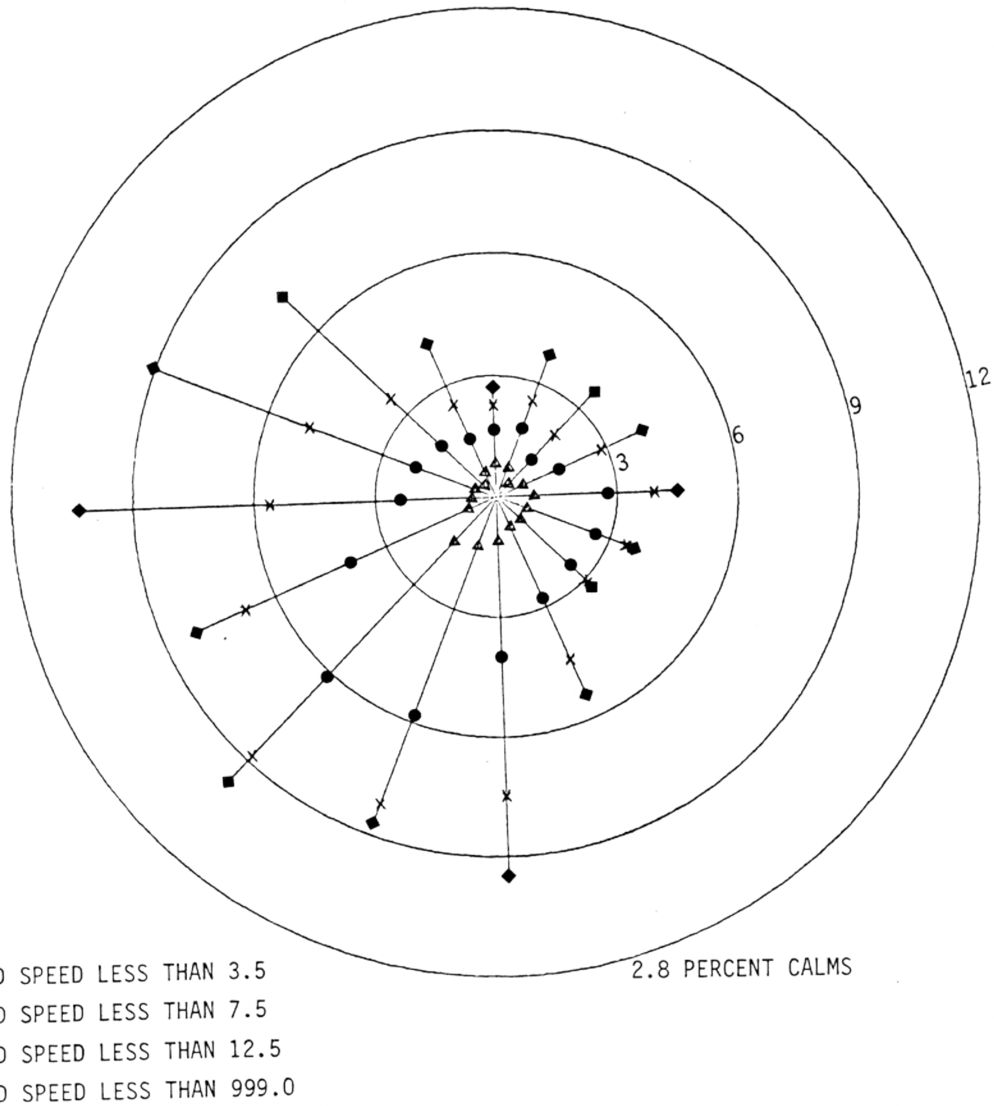
Figure 2.3-5 Ginna 1966, 50-Ft Wind Rose



ROCHESTER GAS AND ELECTRIC CORPORATION
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

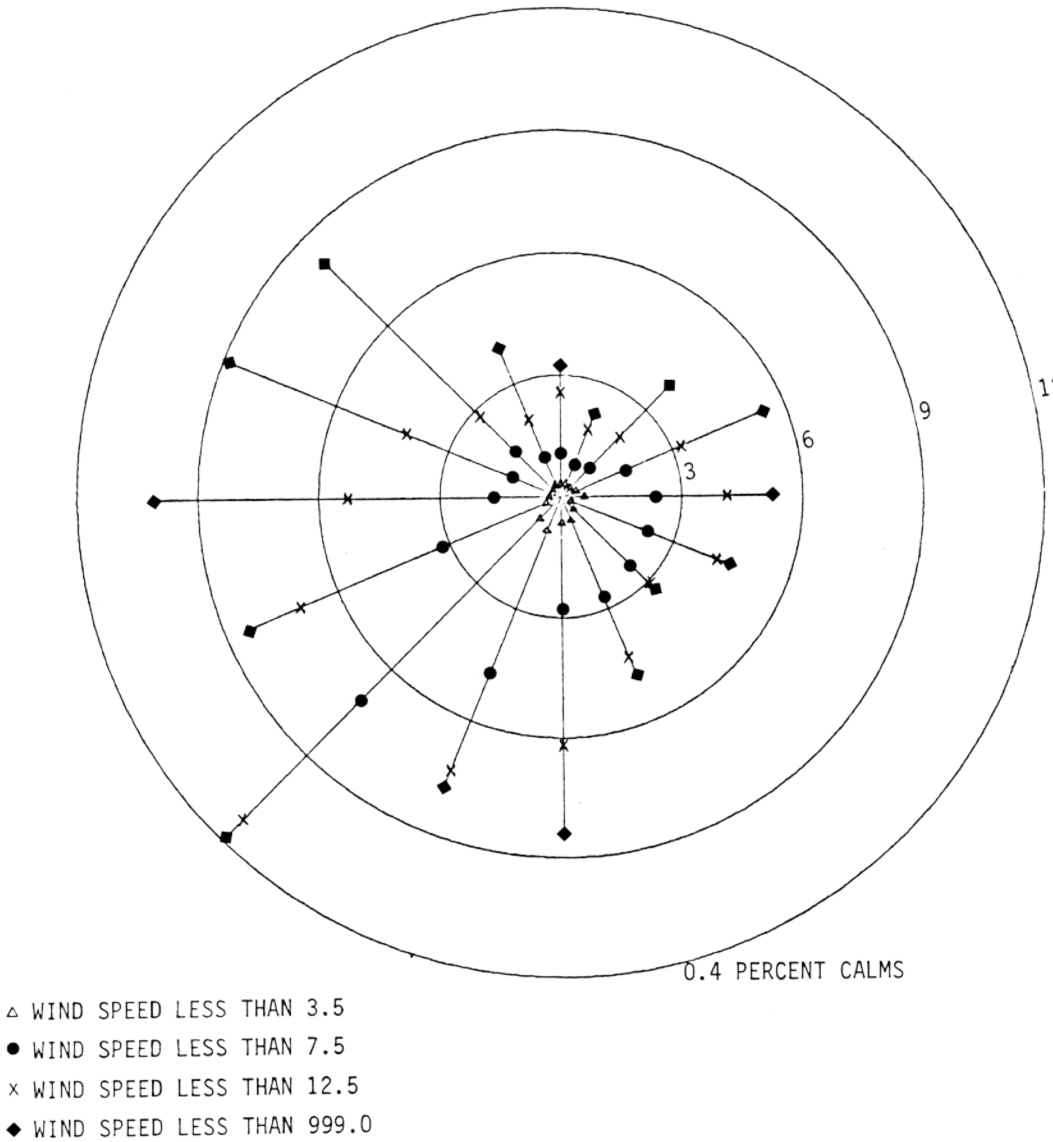
Figure 2.3-5
Ginna 1966, 50-Ft Wind Rose

Figure 2.3-6 Ginna 1967, 50-Ft Wind Rose



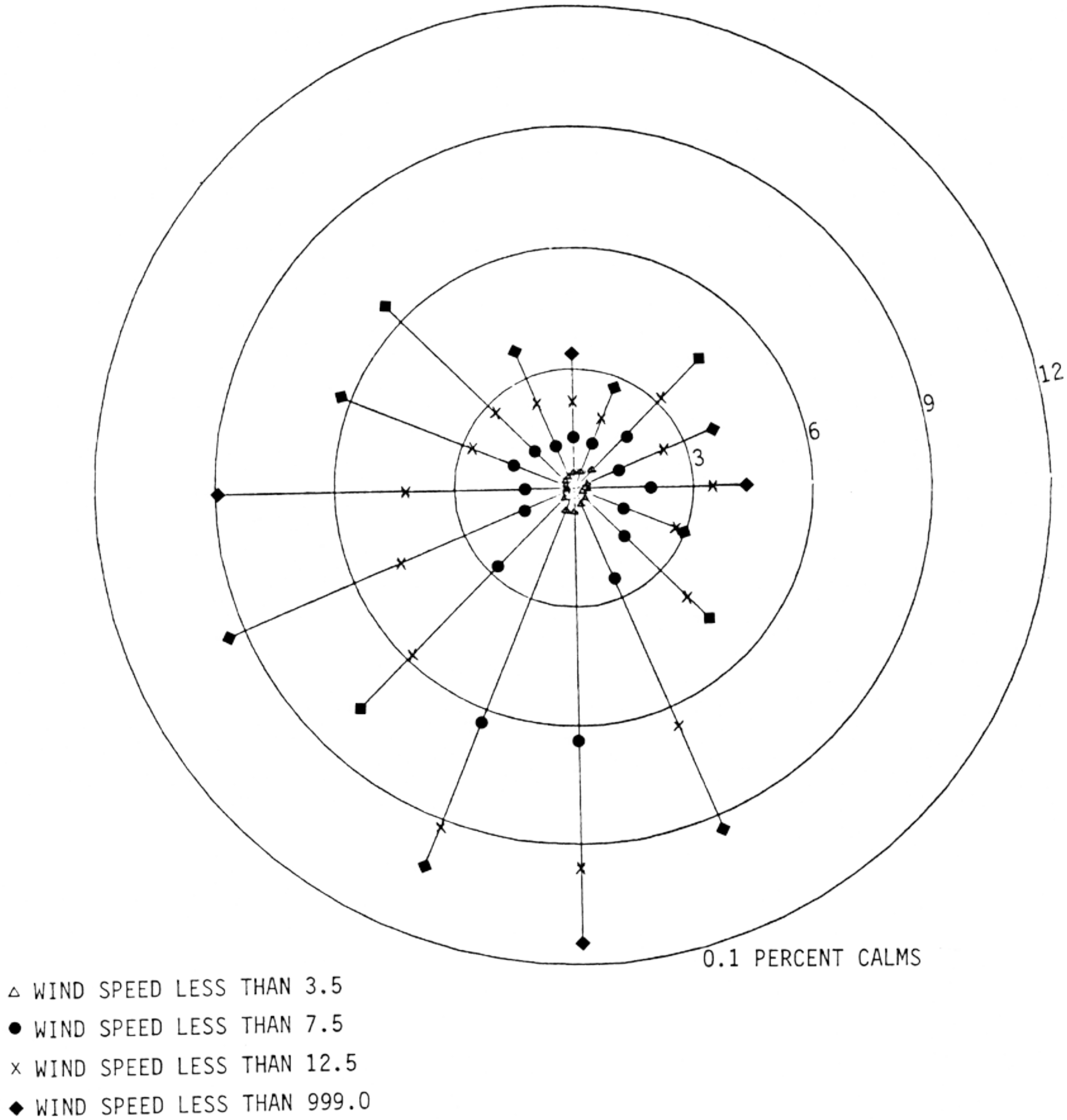
ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-6 Ginna 1967, 50-Ft Wind Rose

Figure 2.3-7 Ginna 1973-1974, 50-Ft Wind Rose



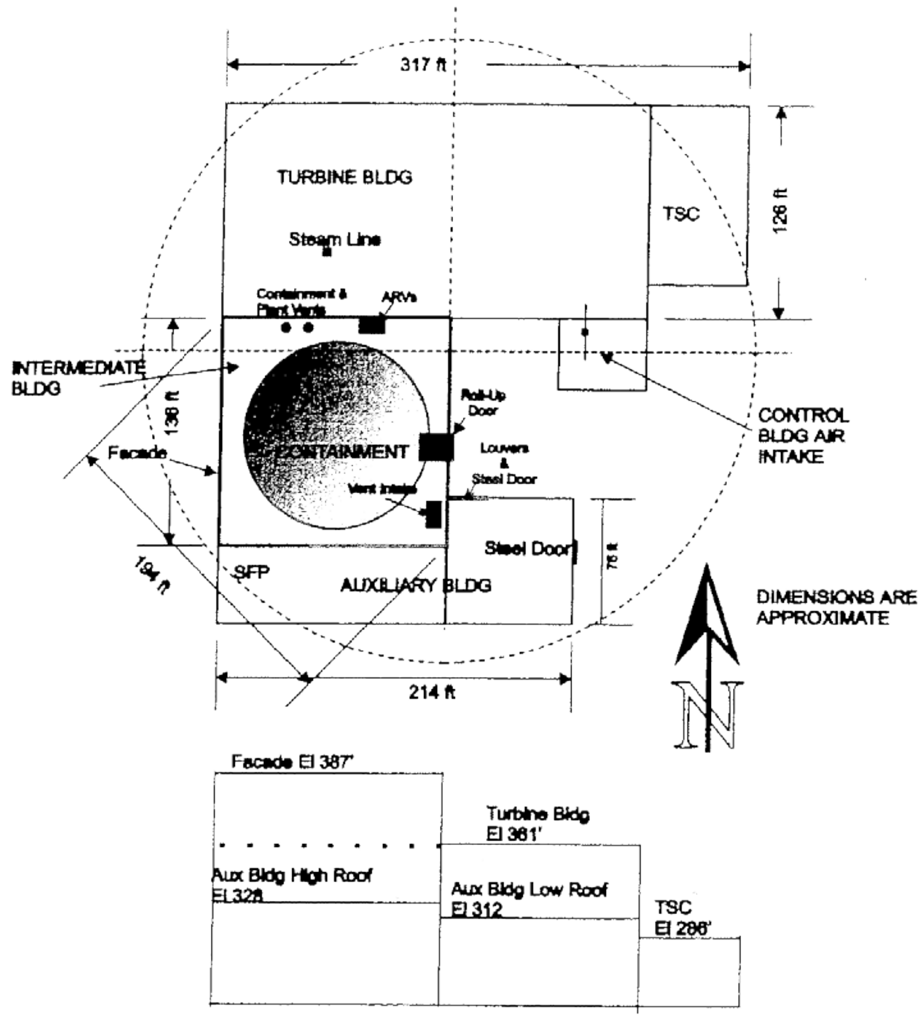
ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-7 Ginna 1973 - 1974, 50-Ft Wind Rose

Figure 2.3-8 Ginna 1975, 33-Ft Wind Rose



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-8 Ginna 1975, 33-Ft Wind Rose

Figure 2.3-9 Site Plan - Activity Release Points and Elevation



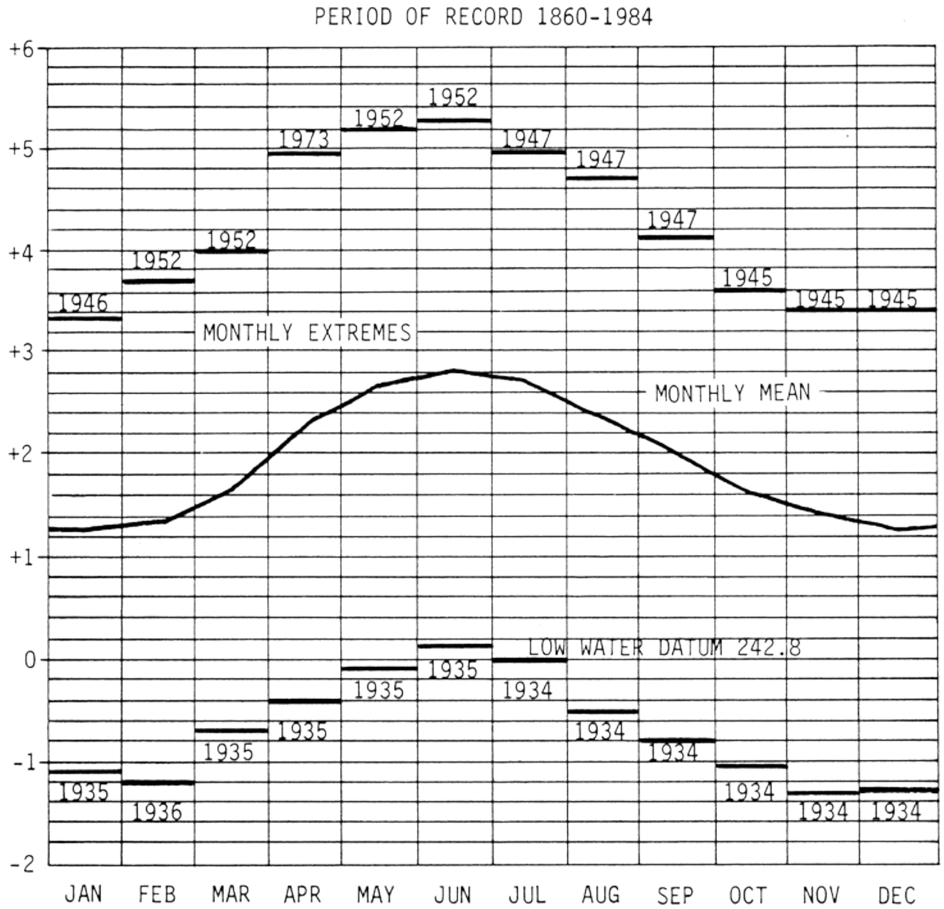
NOTES:

1) DIMENSIONS ARE APPROXIMATE.

2) CENTER OF SITE IS APPROXIMATE.

R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 2.3-9
Site Plan - Activity Release Points and Elevations

Figure 2.4-1 Lake Ontario Levels



NOTE: MONTHLY BULLETIN OF LAKE LEVELS, DISTRICT CORPS OF ENGINEERS, DETROIT, MICHIGAN

INTERNATIONAL GREAT LAKES DATUM (1955) +1.23 ft = U.S. COAST & GEODETIC SURVEY DATUM (1935)

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Figure 2.4-1 Lake Ontario Levels

Figure 2.4-2 General North-South Cross Section Ginna Site

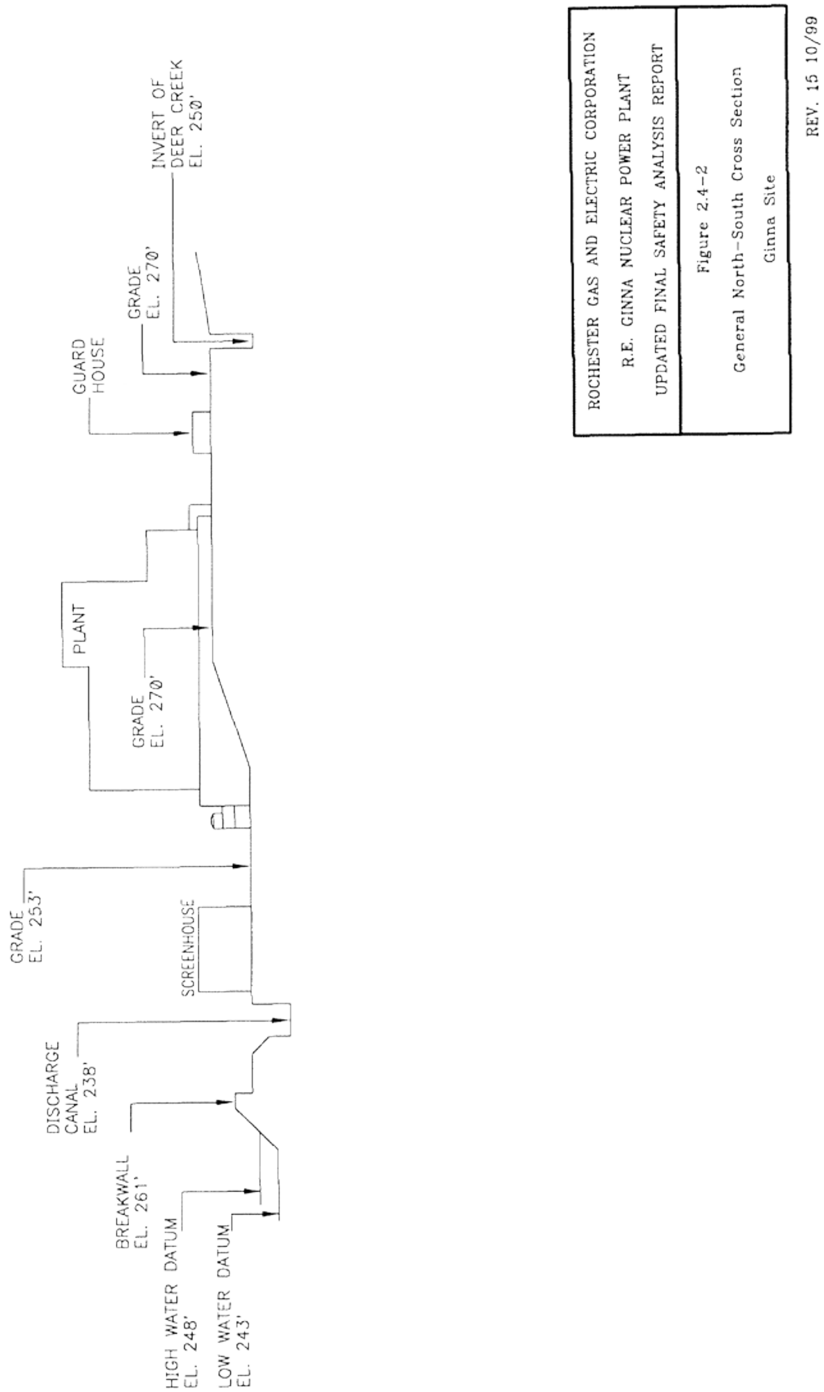
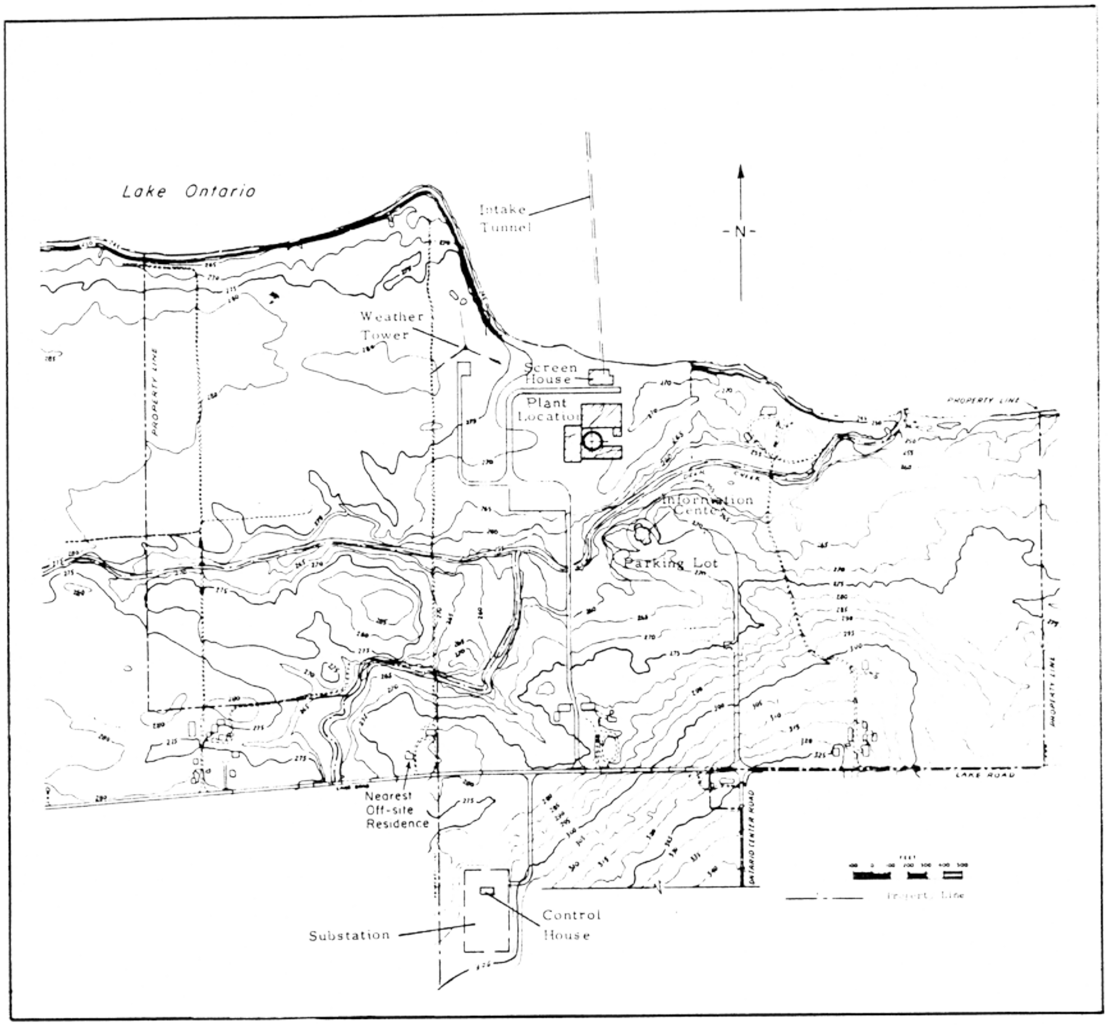


Figure 2.4-3 ***FIGURE DELETED***

FIGURE DELETED

Figure 2.4-4 Ginna Site Layout and Topography



NOTE: PROPERTY LINE IS THAT EXISTING AT TIME OF ORIGINAL FSAR SUBMITTAL. RG&E HAS SINCE ACQUIRED ADDITIONAL PROPERTY.

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Figure 2.4-4

Ginna Site
Layout and Topography

Figure 2.5-1 Plot Plan Showing Boring Locations

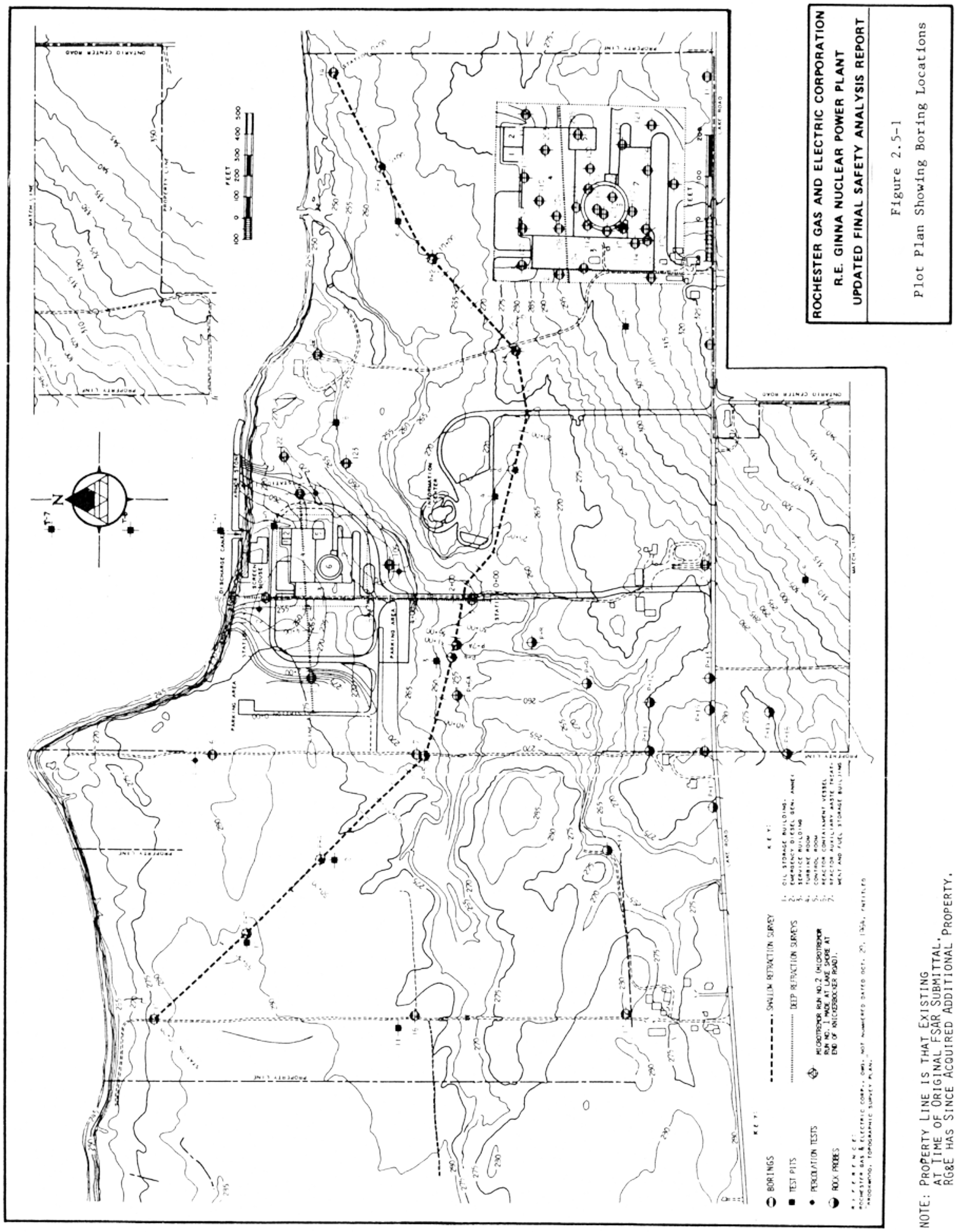


Figure 2.5-2 Epicentral Location Map

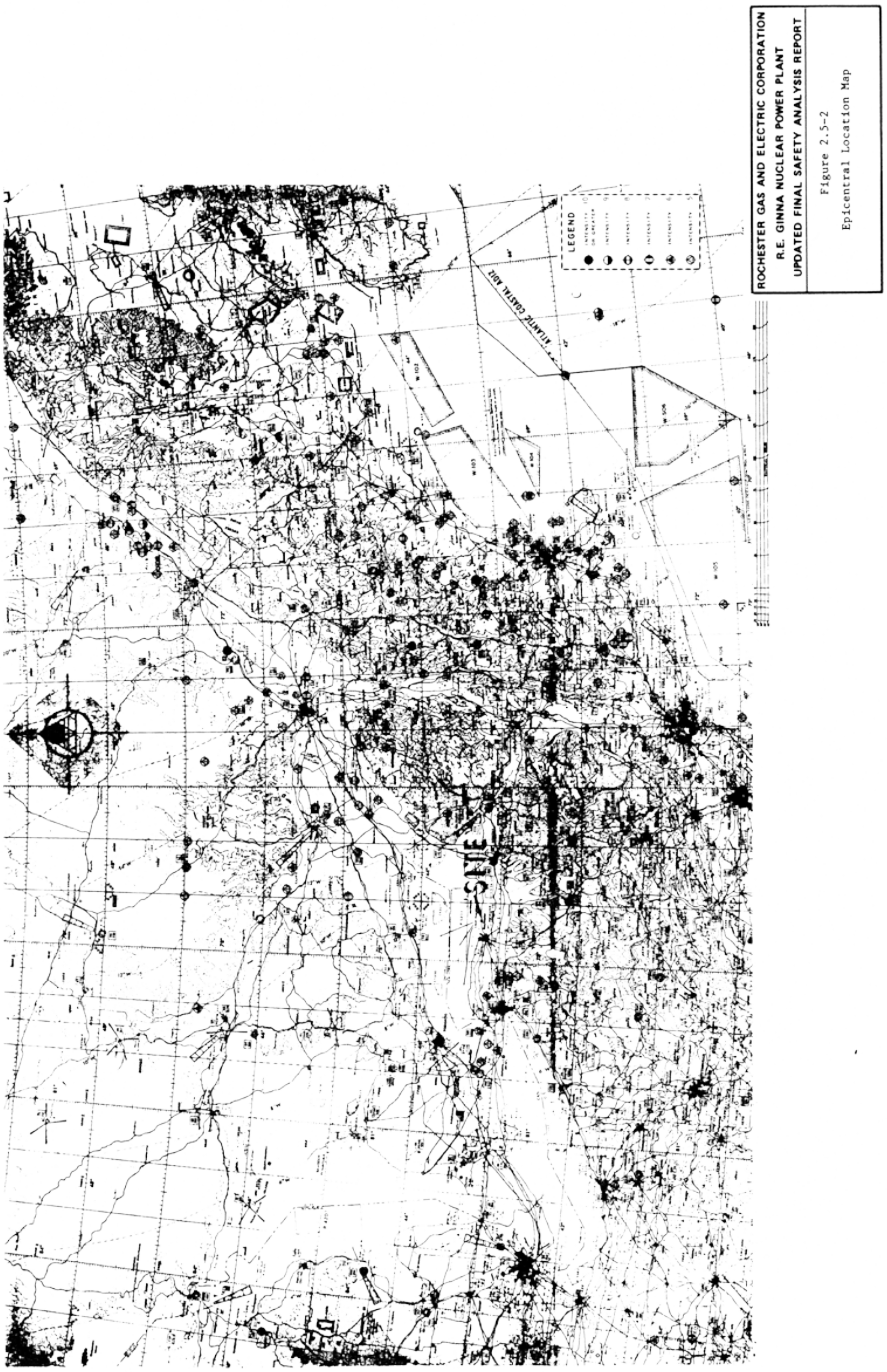
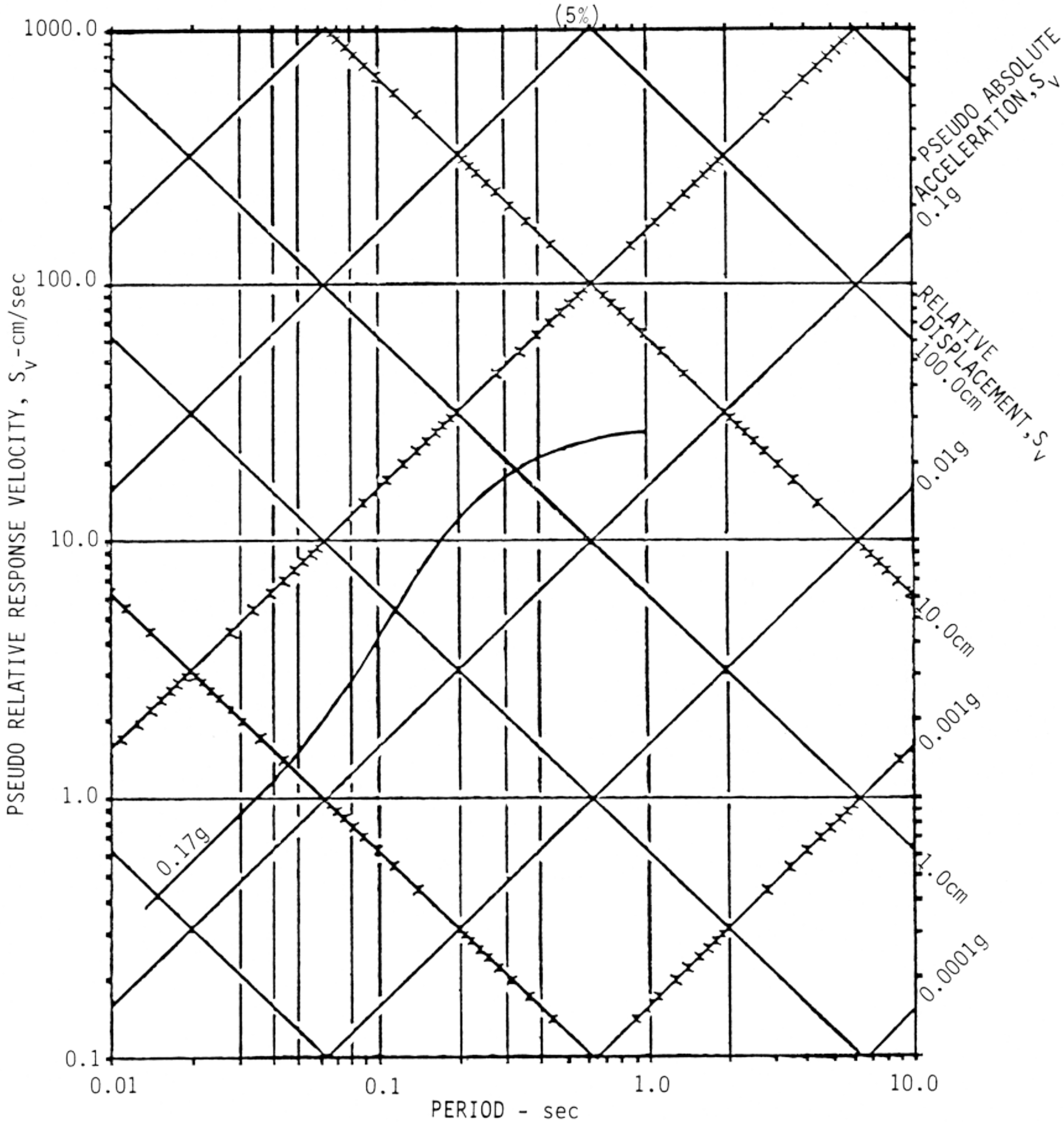


Figure 2.5-3 NRC Systematic Evaluation Program, Site Specific Spectrum, Ginna Site (5% Damping)



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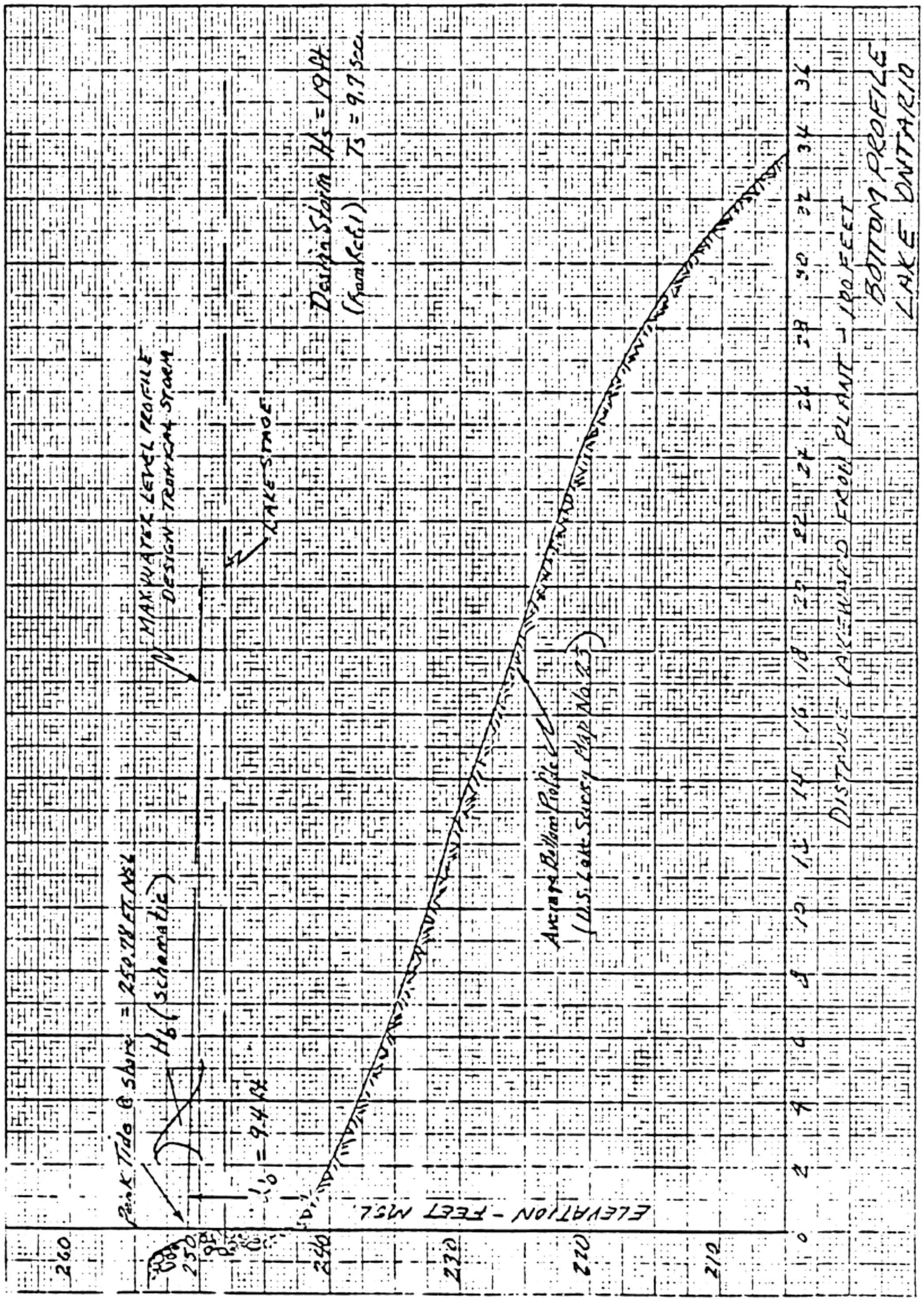
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 2.5-3

NRC Systematic Evaluation Program
Site Specific Spectrum, Ginna Site
(5% Damping)

Figure 1 Lake Ontario Bottom Profile



2A-7

Figure Exhibit 1 A typical northeaster with a single center and a single frontal system, occurring 0100 EST, April 2, 1958, four hours before a peak surge of 3.5 ft. at Boston, Mass.

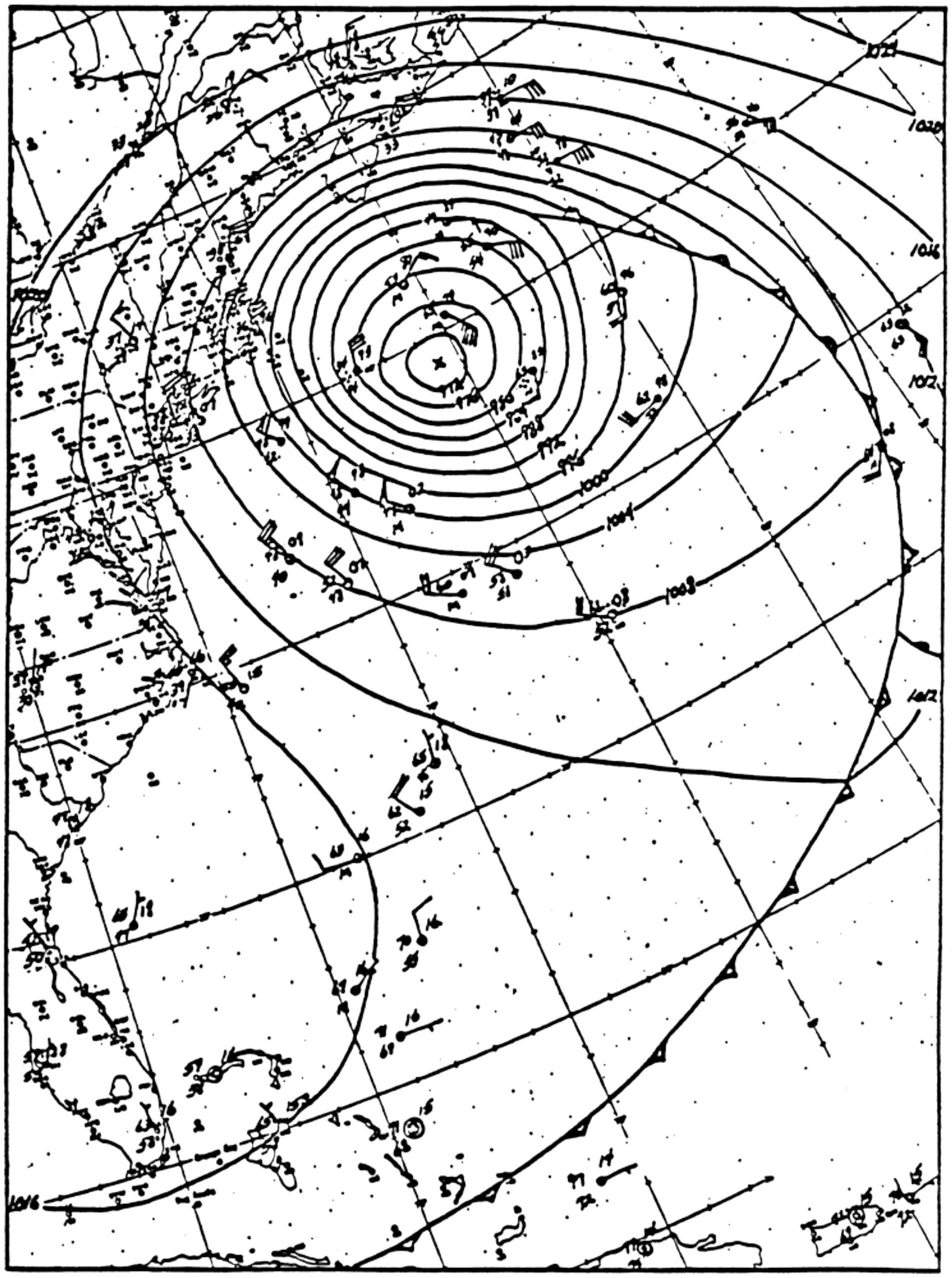


Figure Exhibit 2 A complex northeaster with two low centers and a double frontal system, occurring 1900 EST, March 1, 1914, three hours before a peak surge of 4.1 ft. at Portland, Maine.

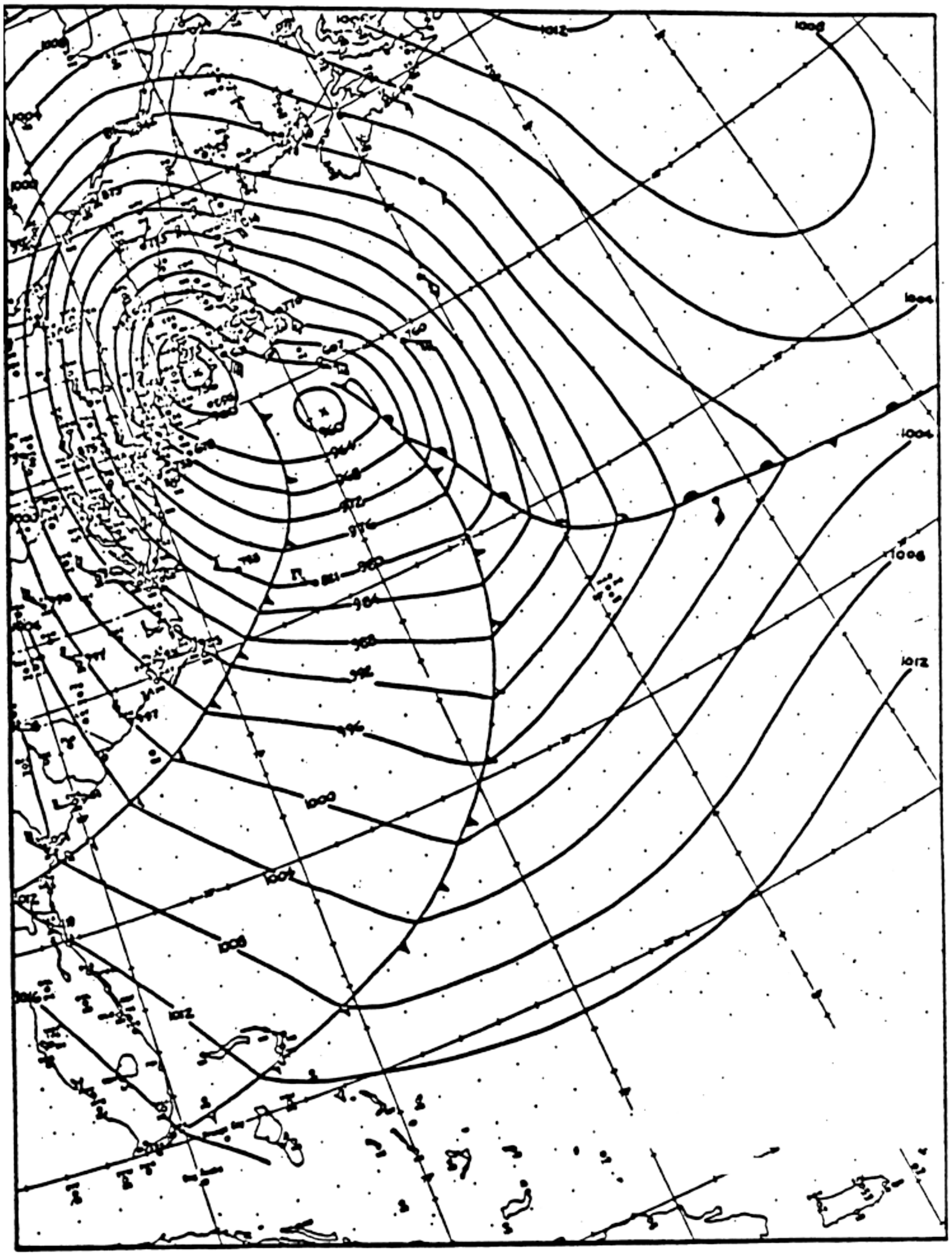


Figure Exhibit 3 Lake Ontario Stage Data

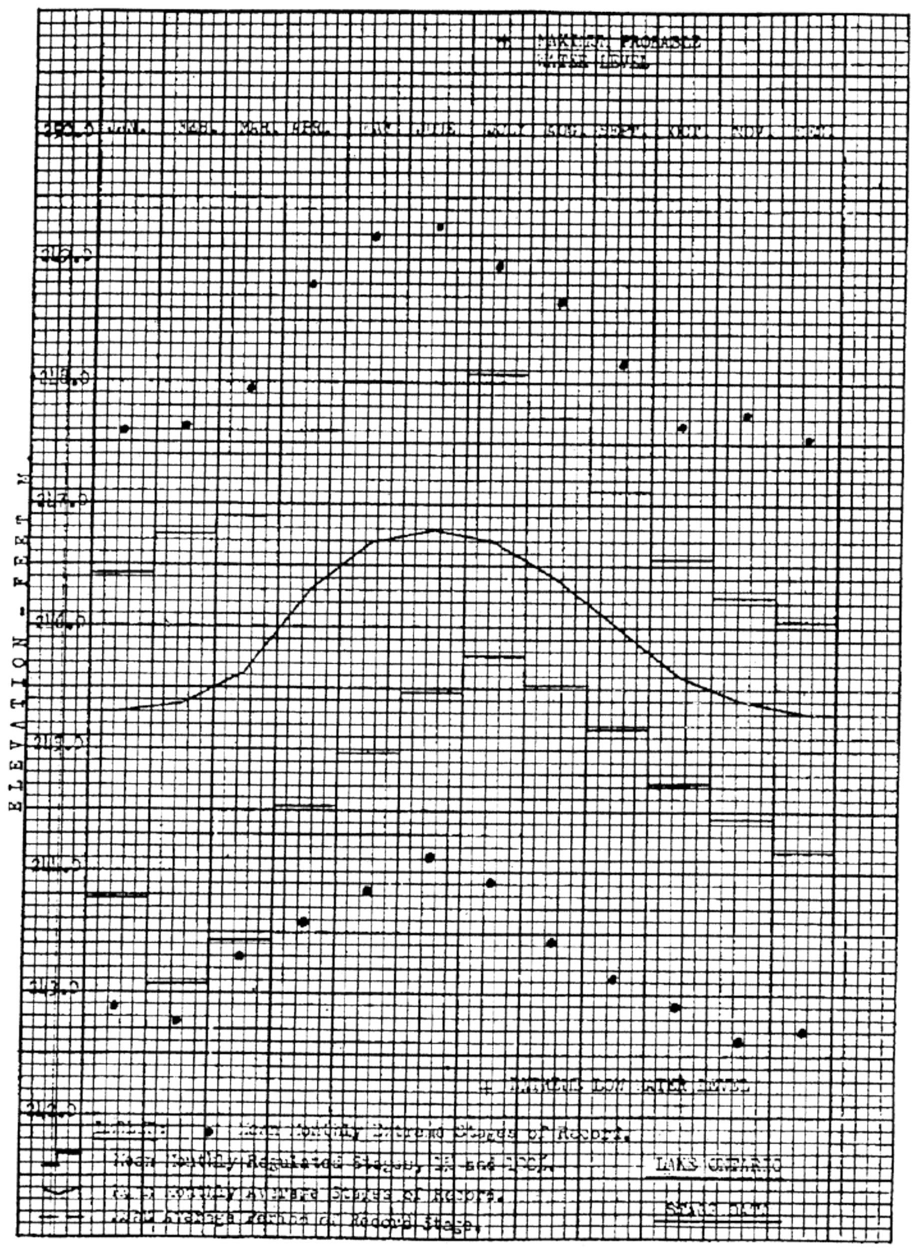
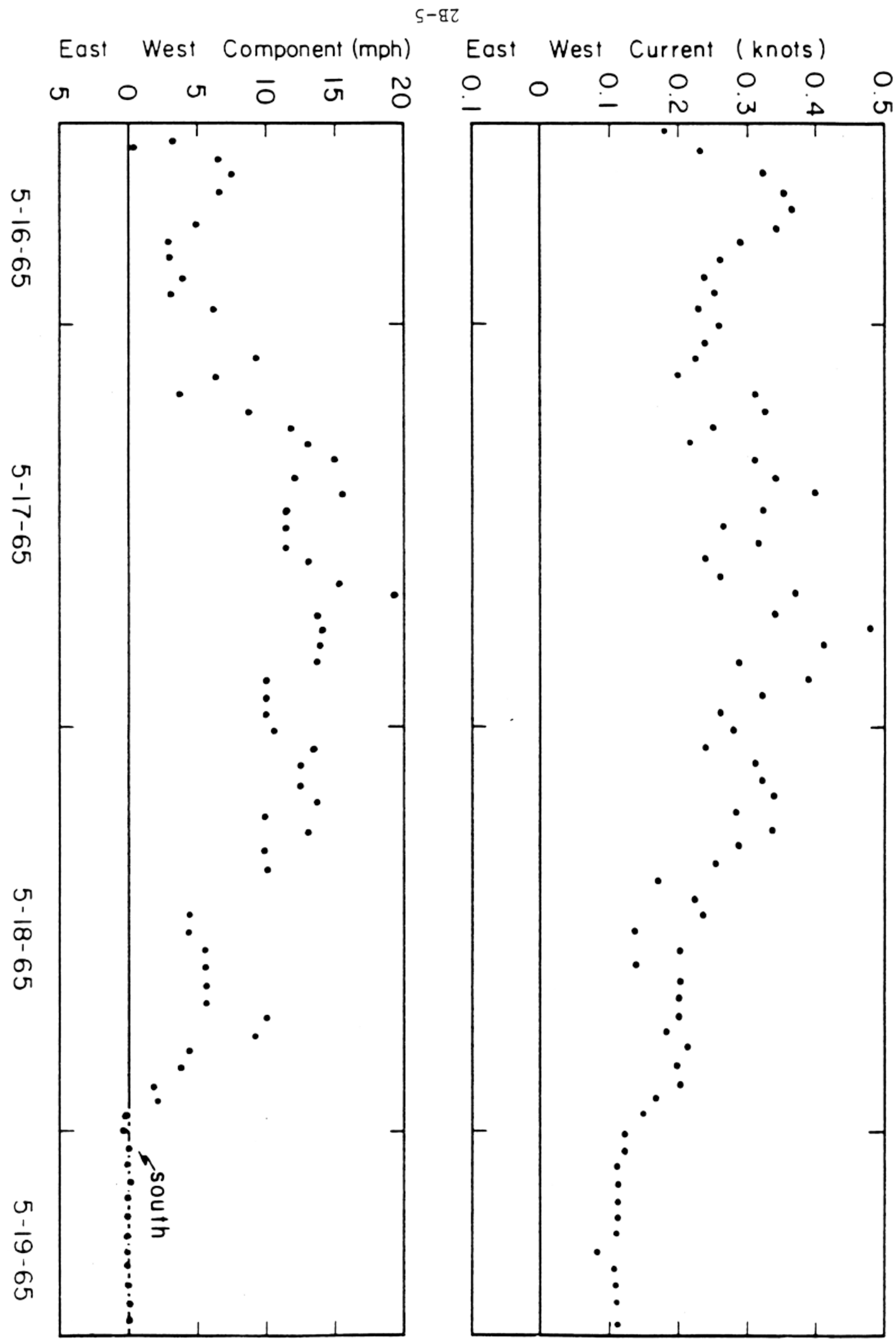
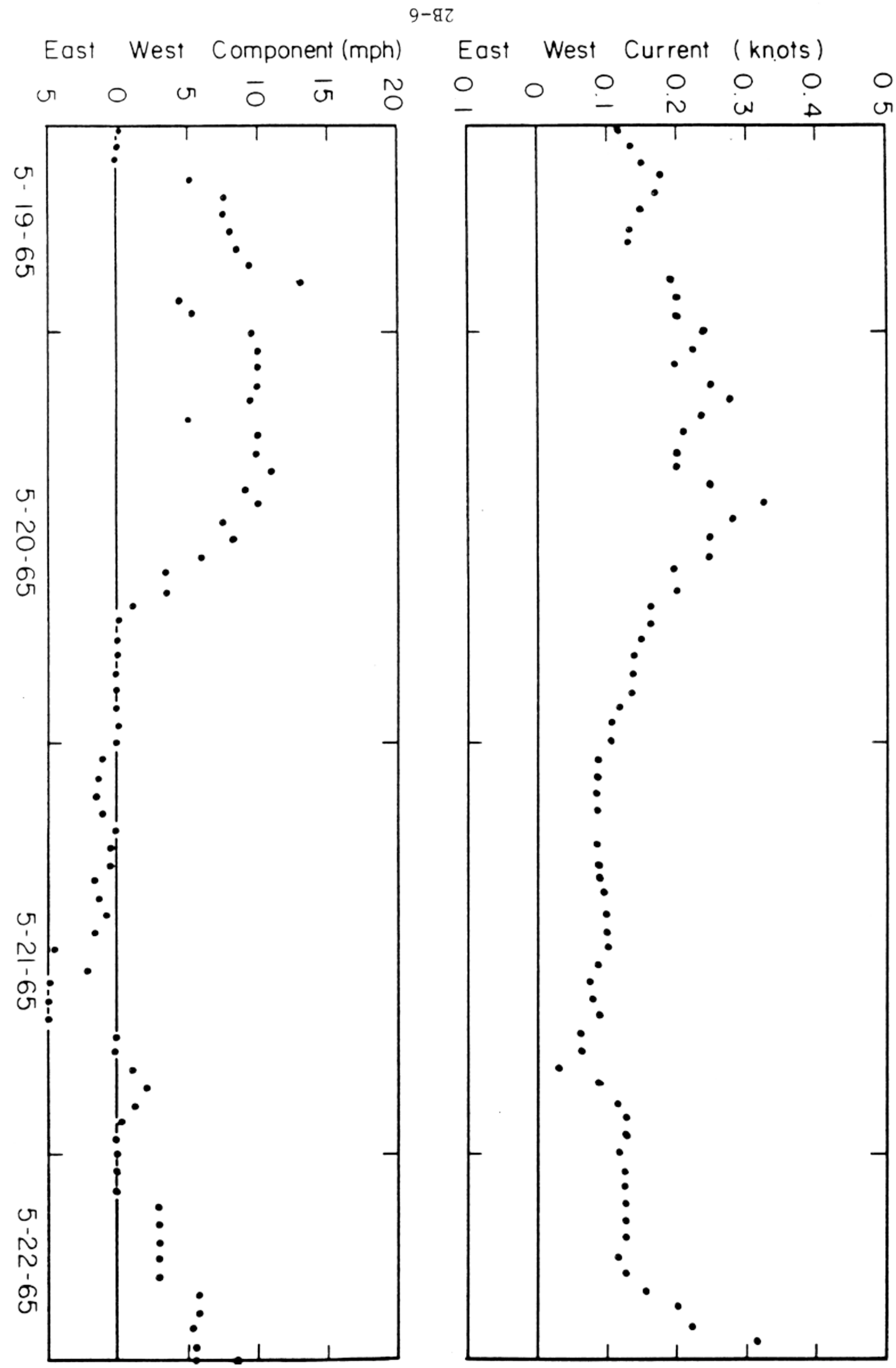


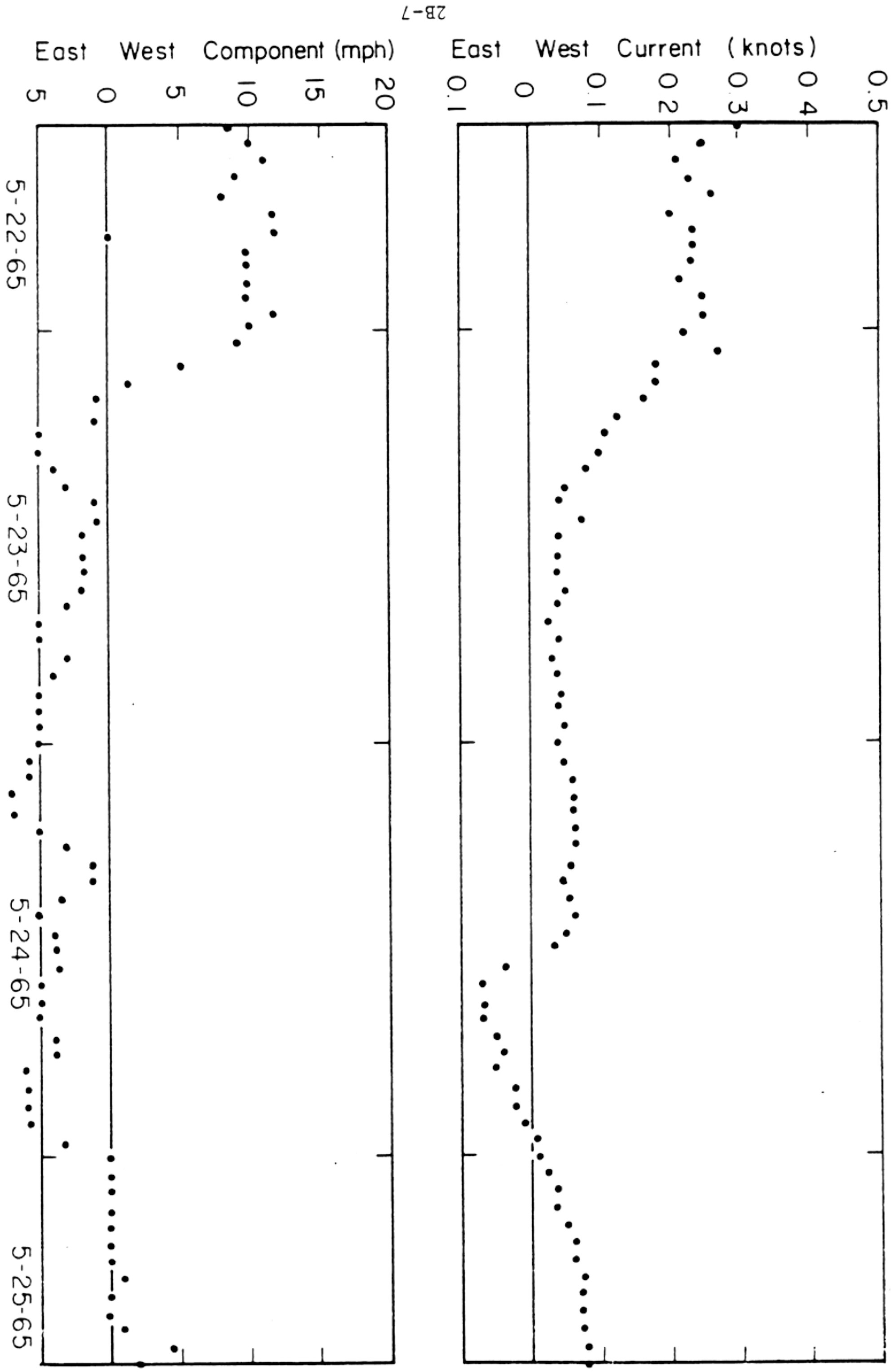
Figure 1 Upper - Lake Current, Lower - Wind Speed



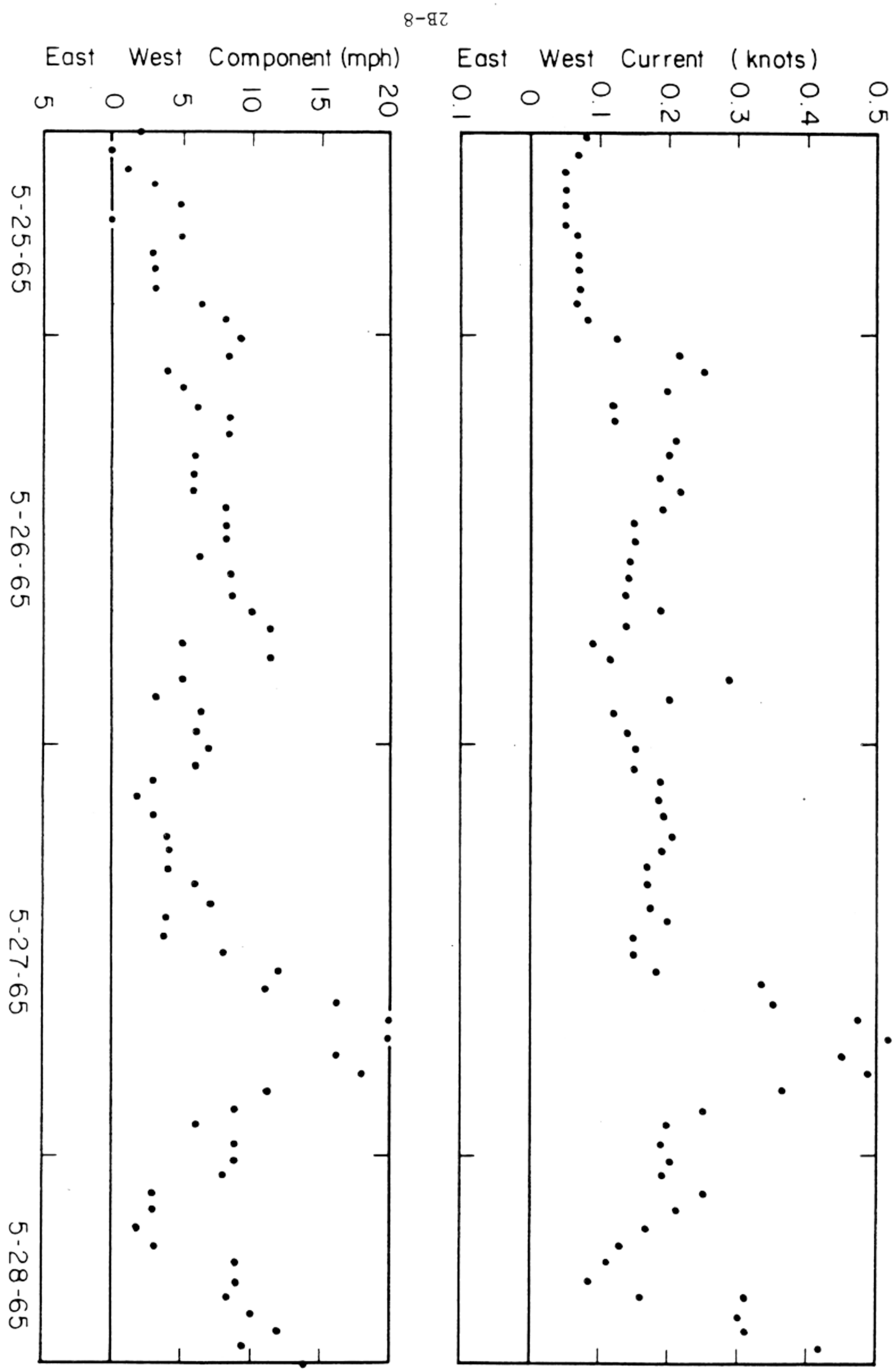
Sheet 2 of Figure 1



Sheet 3 of Figure 1



Sheet 4 of Figure 1



Sheet 5 of Figure 1

2B-9

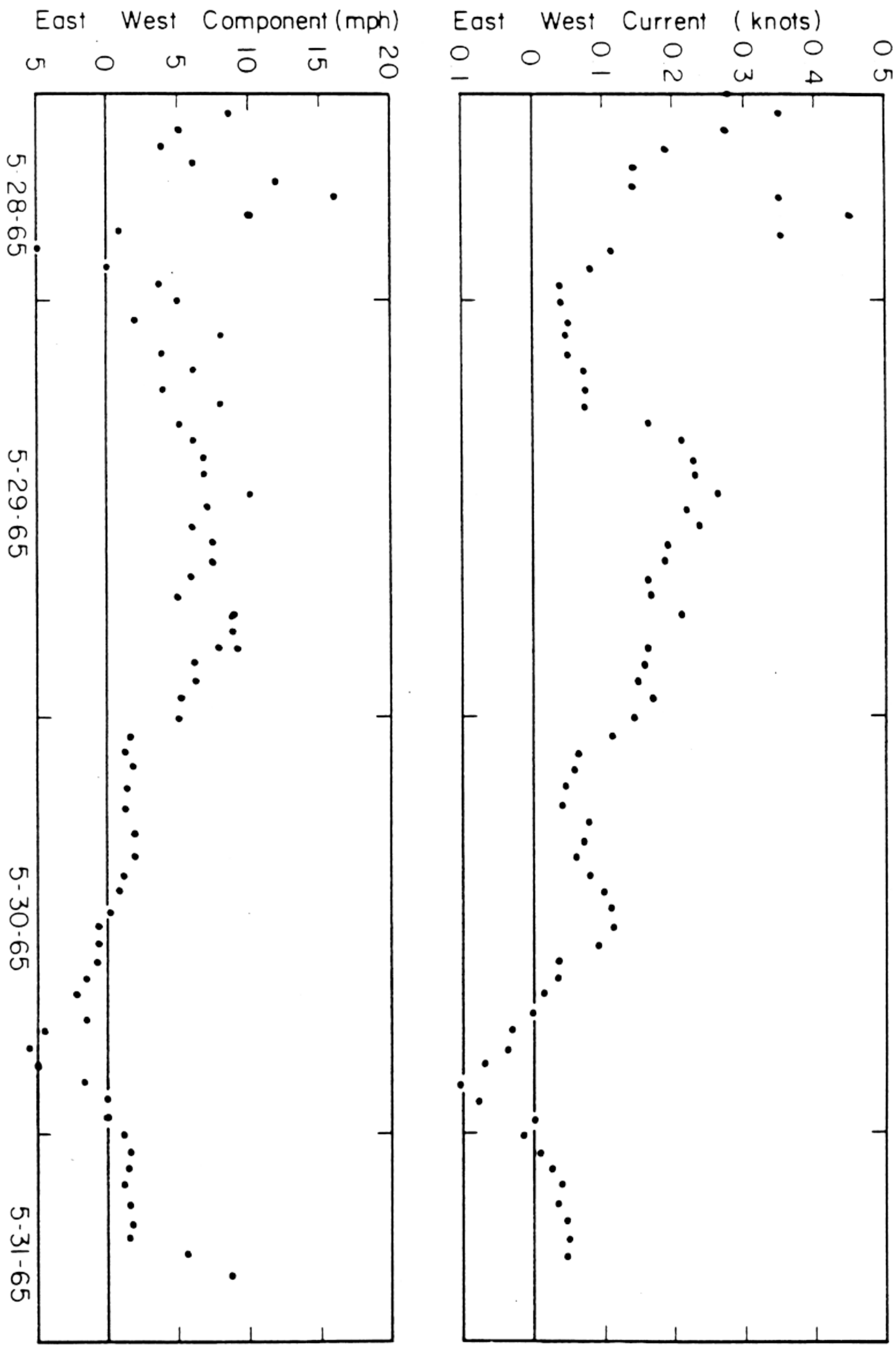
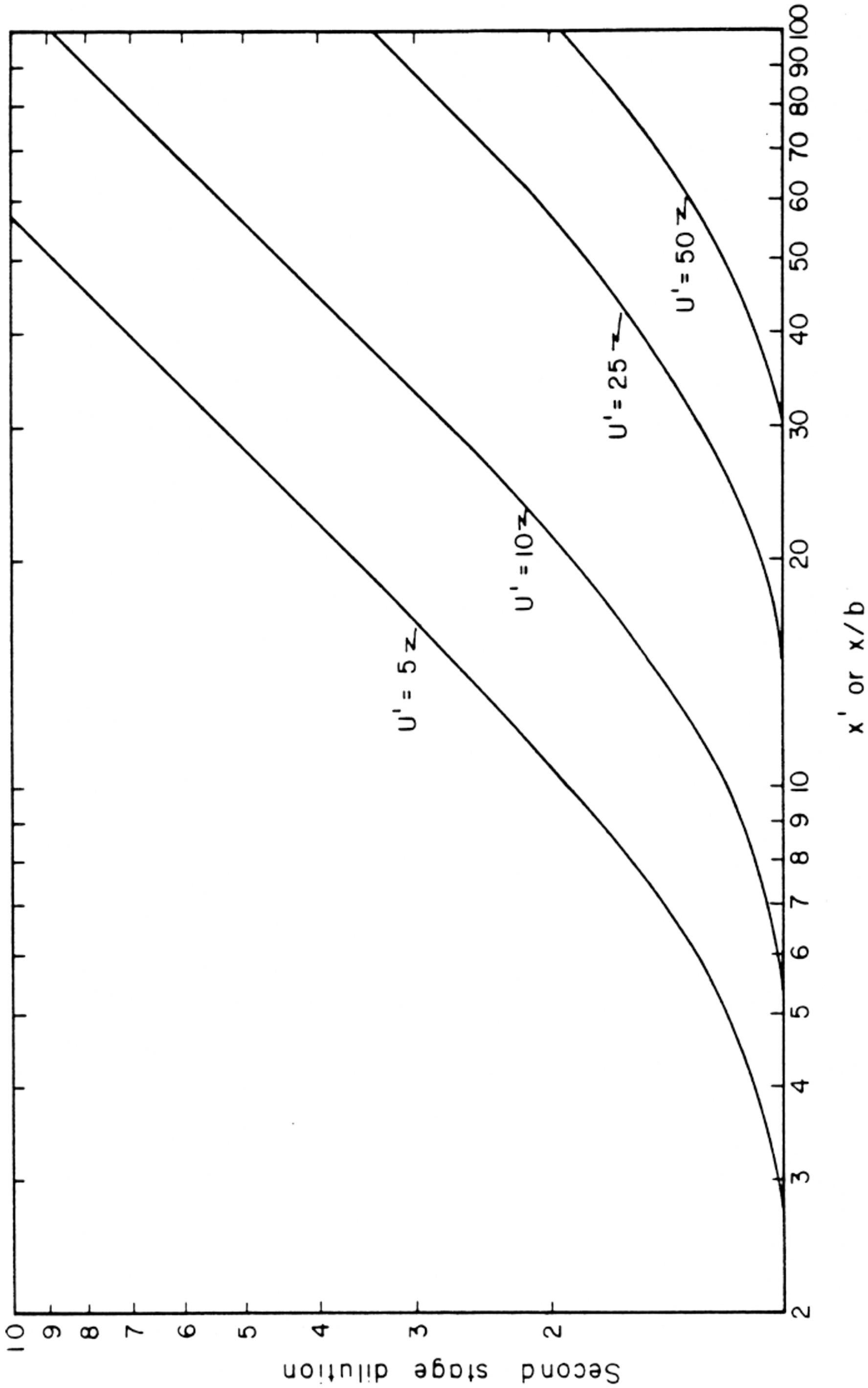


Figure 2



2B-23

Figure 3

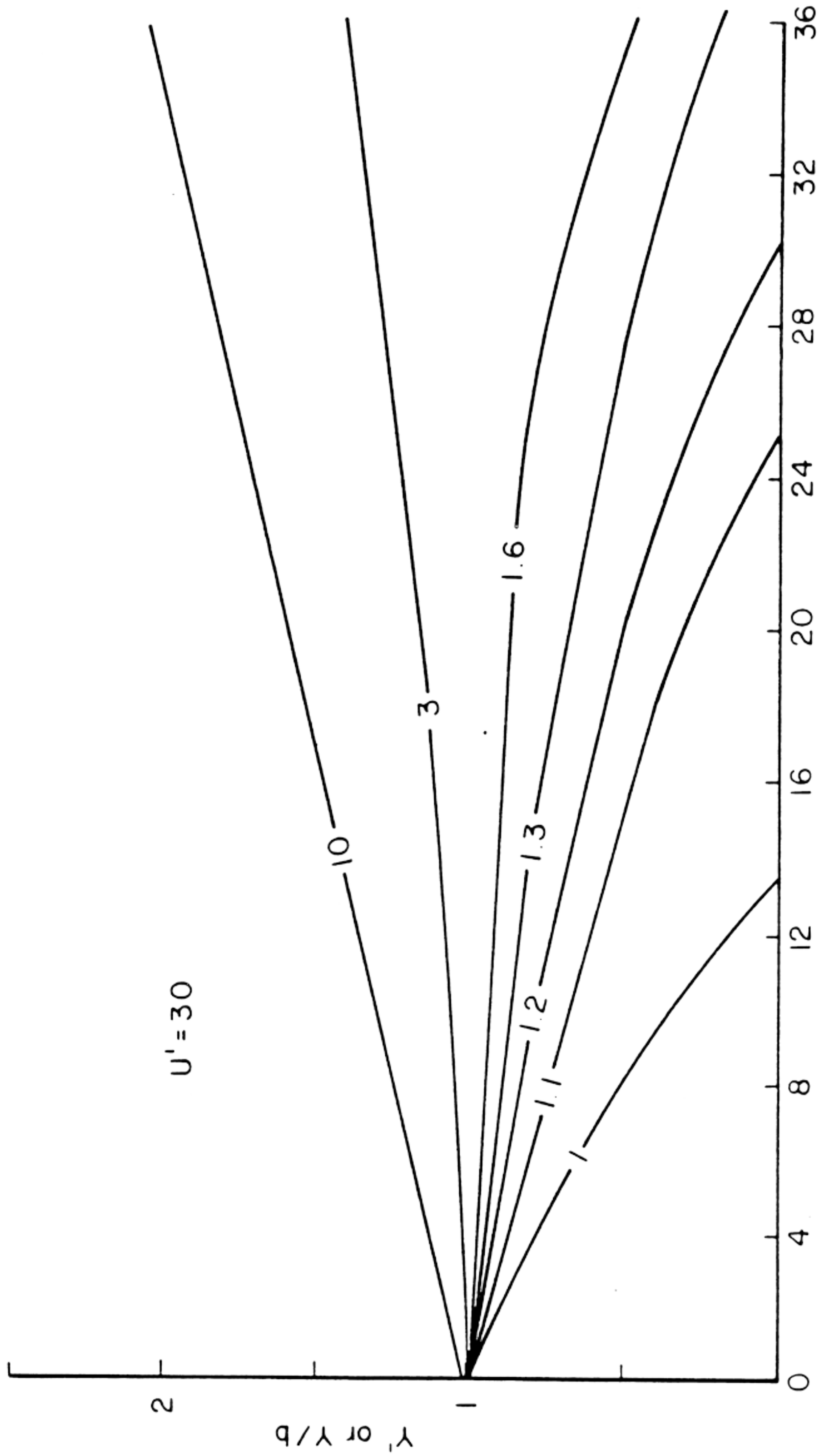


Figure 4 Maximum concentration versus distance, May 1965 East Distribution

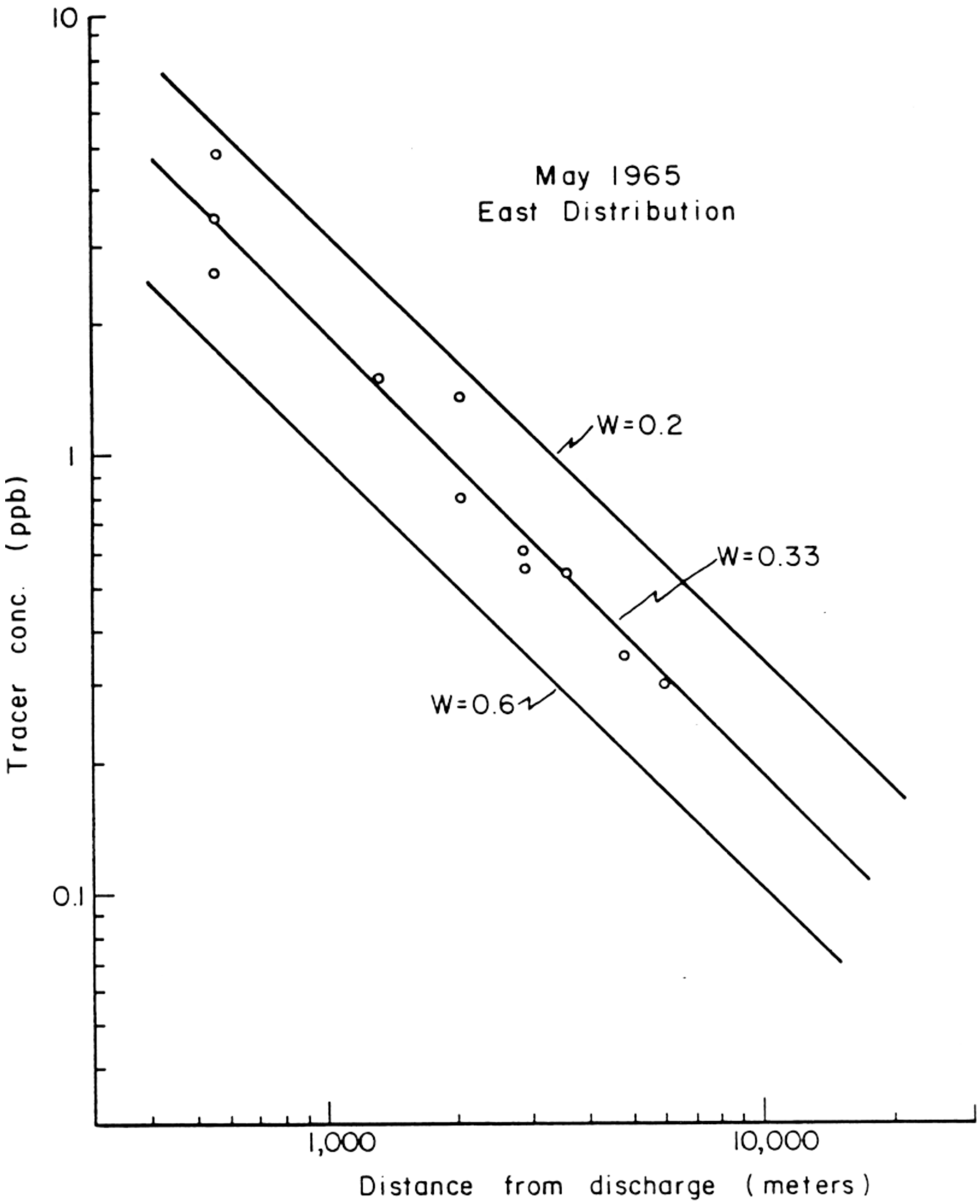


Figure 4. Maximum concentration versus distance.

Figure 5 Maximum concentration versus distance, July 1965 East Distribution

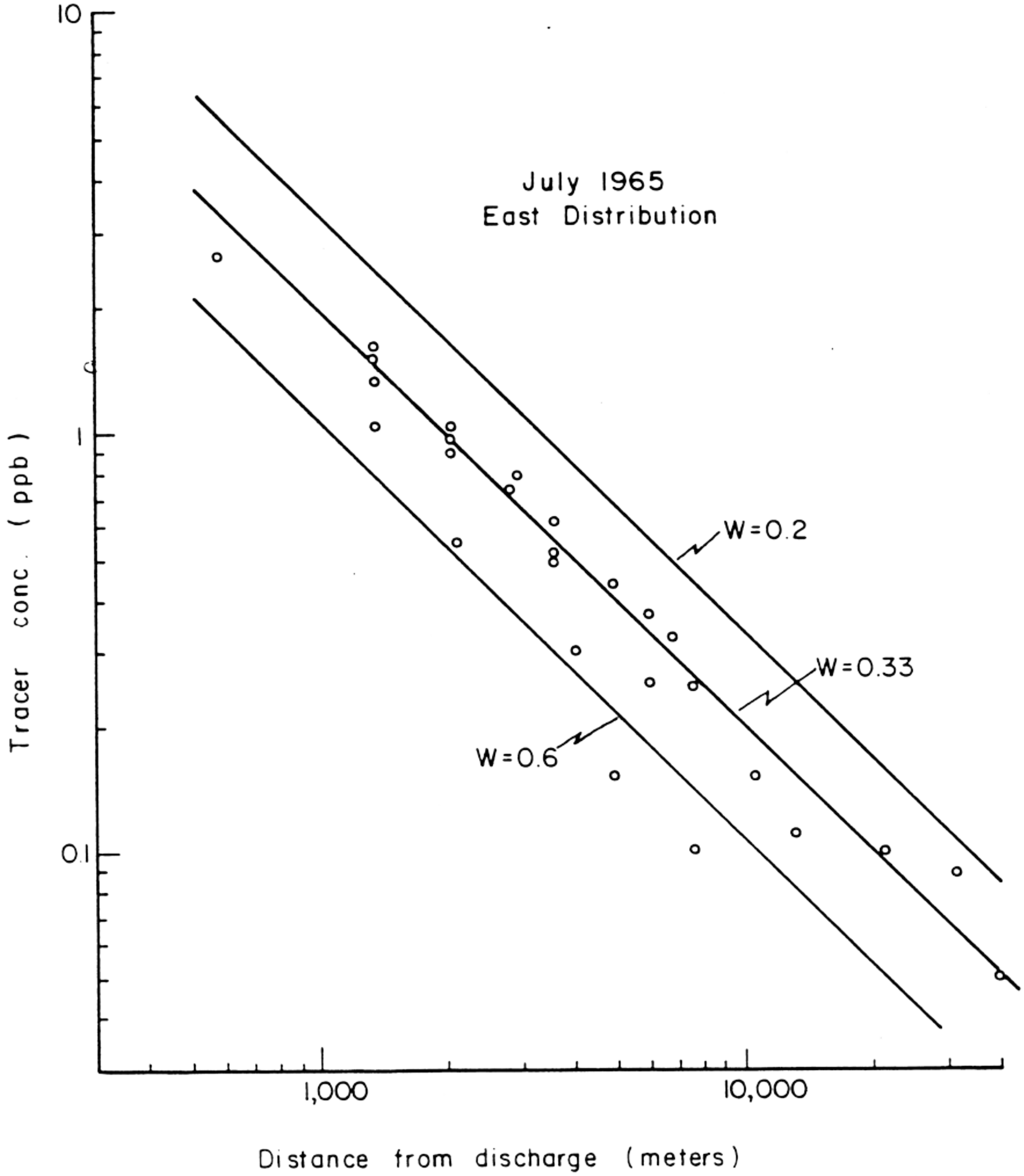


Figure 5. Maximum concentration versus distance.

Figure 6 Maximum concentration versus distance, October 1965 East Distribution

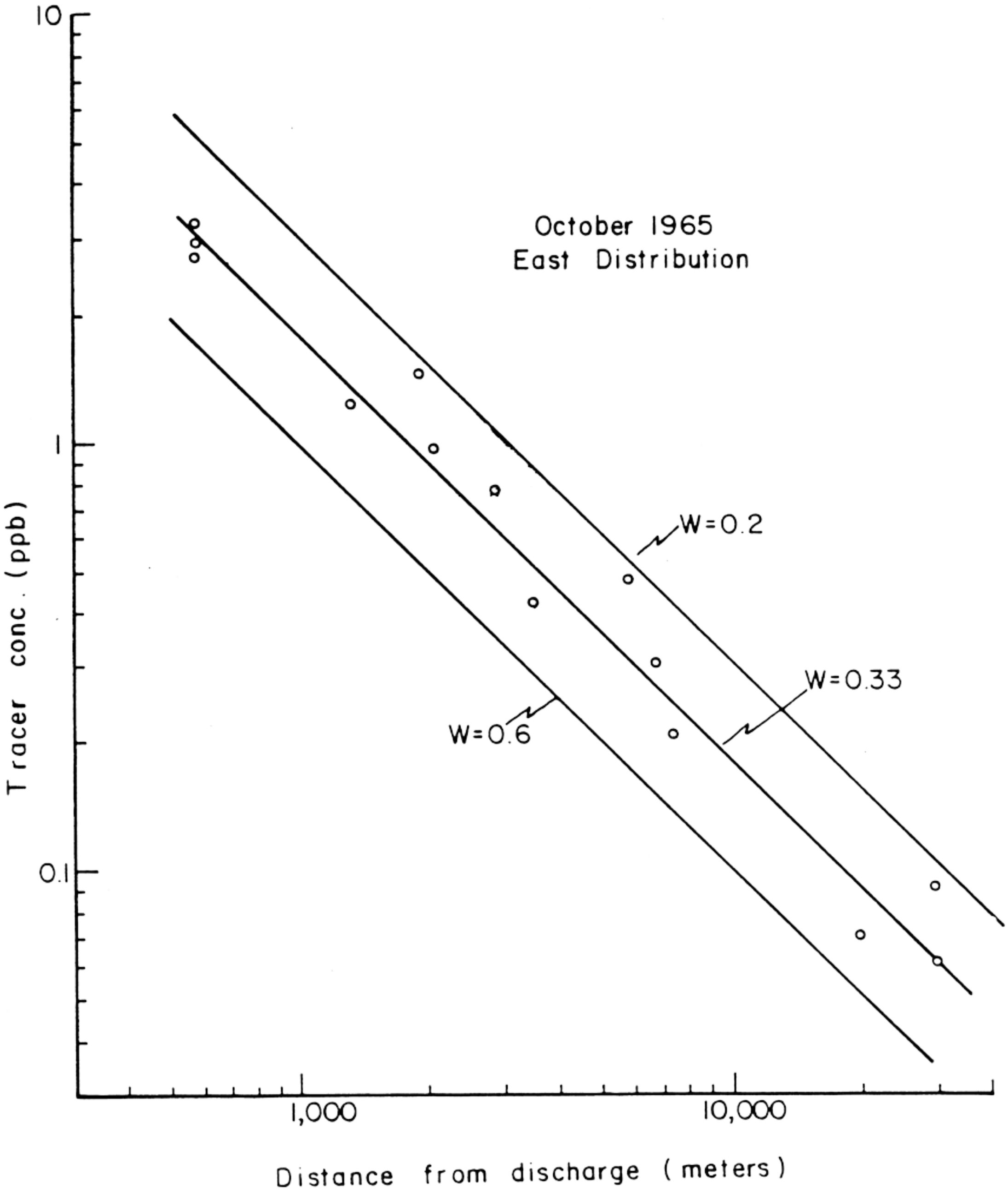


Figure 6. Maximum concentration versus distance.

Figure 7 Maximum concentration versus distance, May 1965 West Distribution

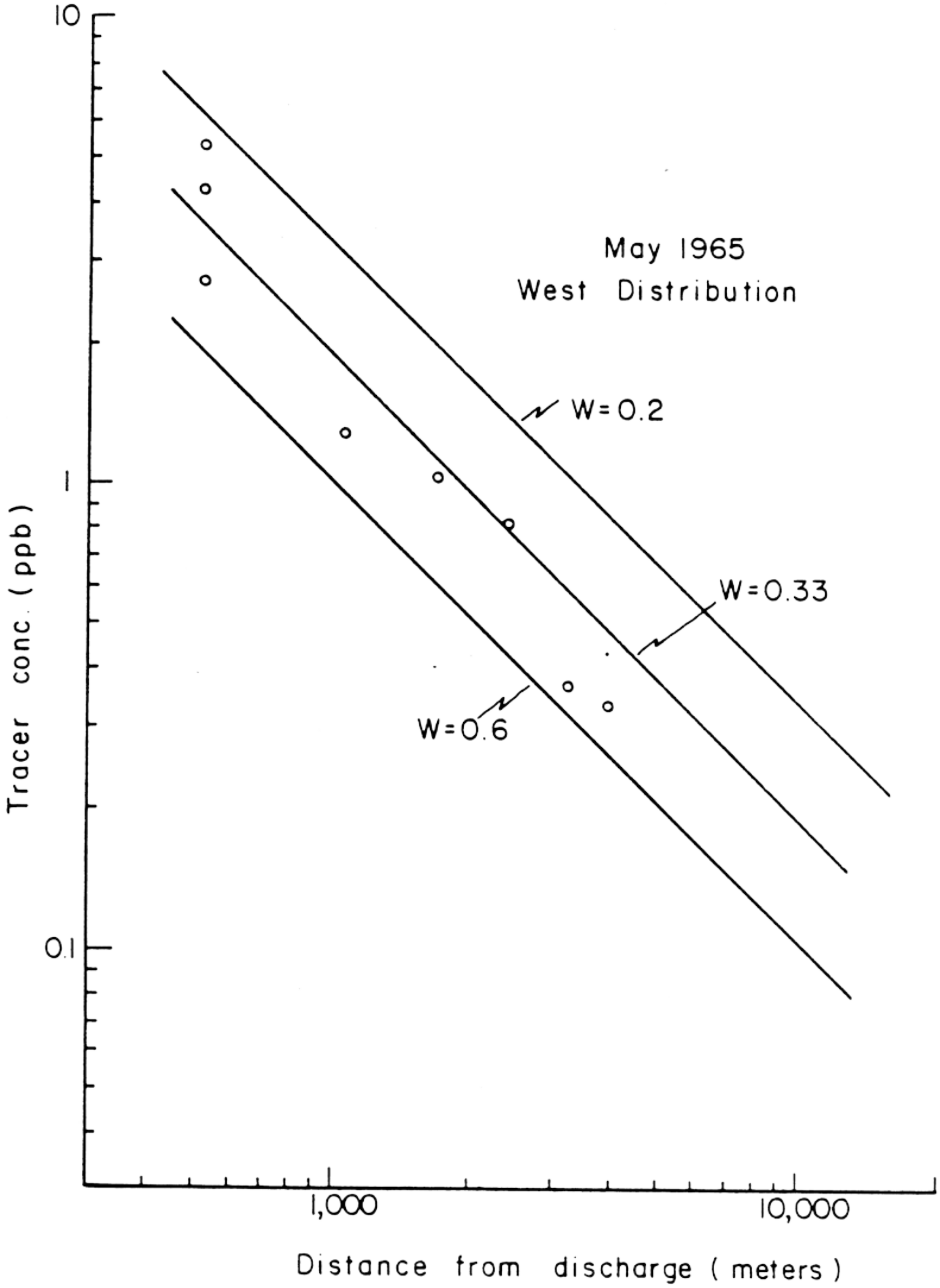


Figure 7. Maximum concentration versus distance.

Figure 8 Maximum concentration versus distance, October 1965 West Distribution

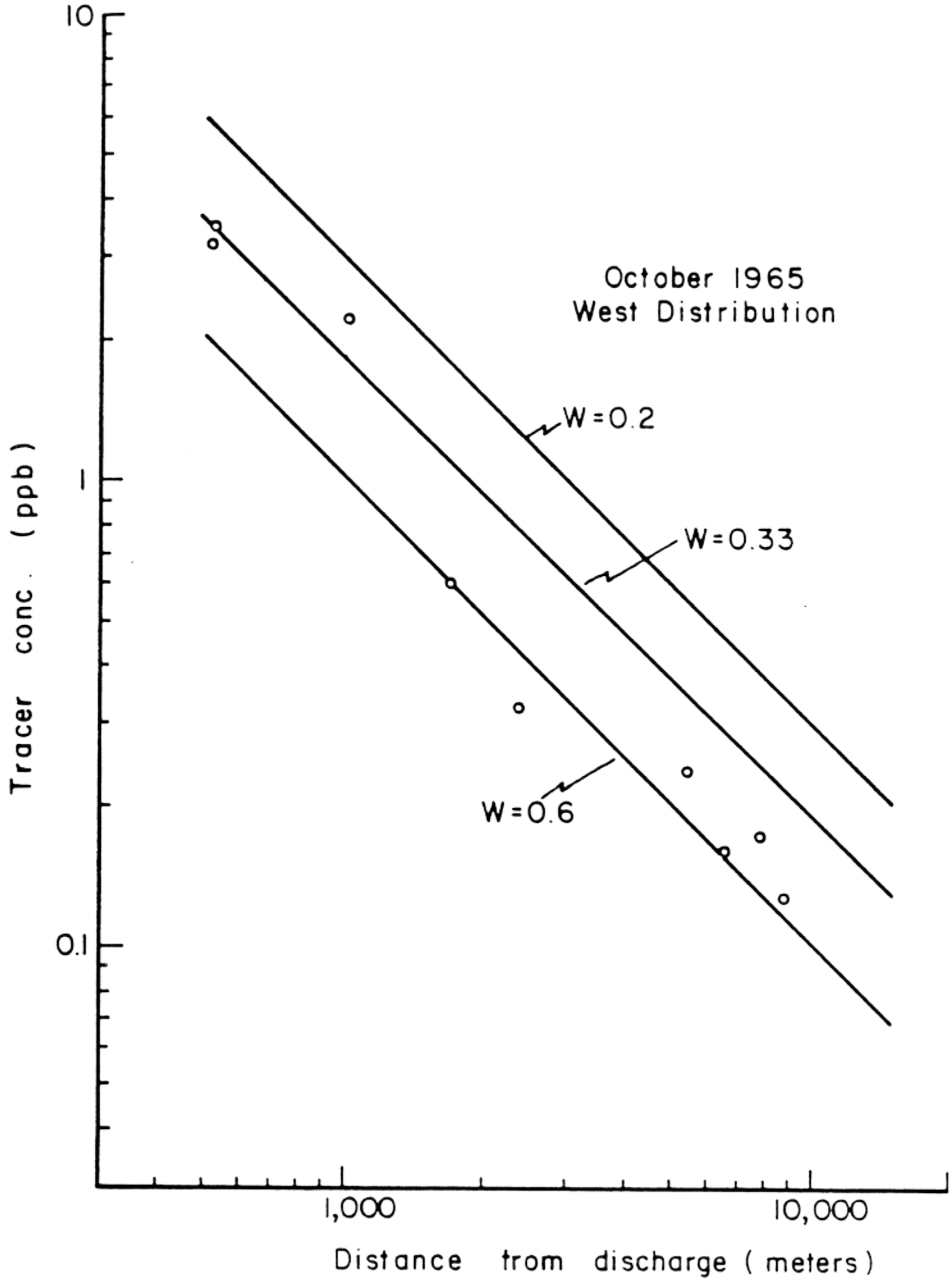


Figure 8. Maximum concentration versus distance.

Figure 9

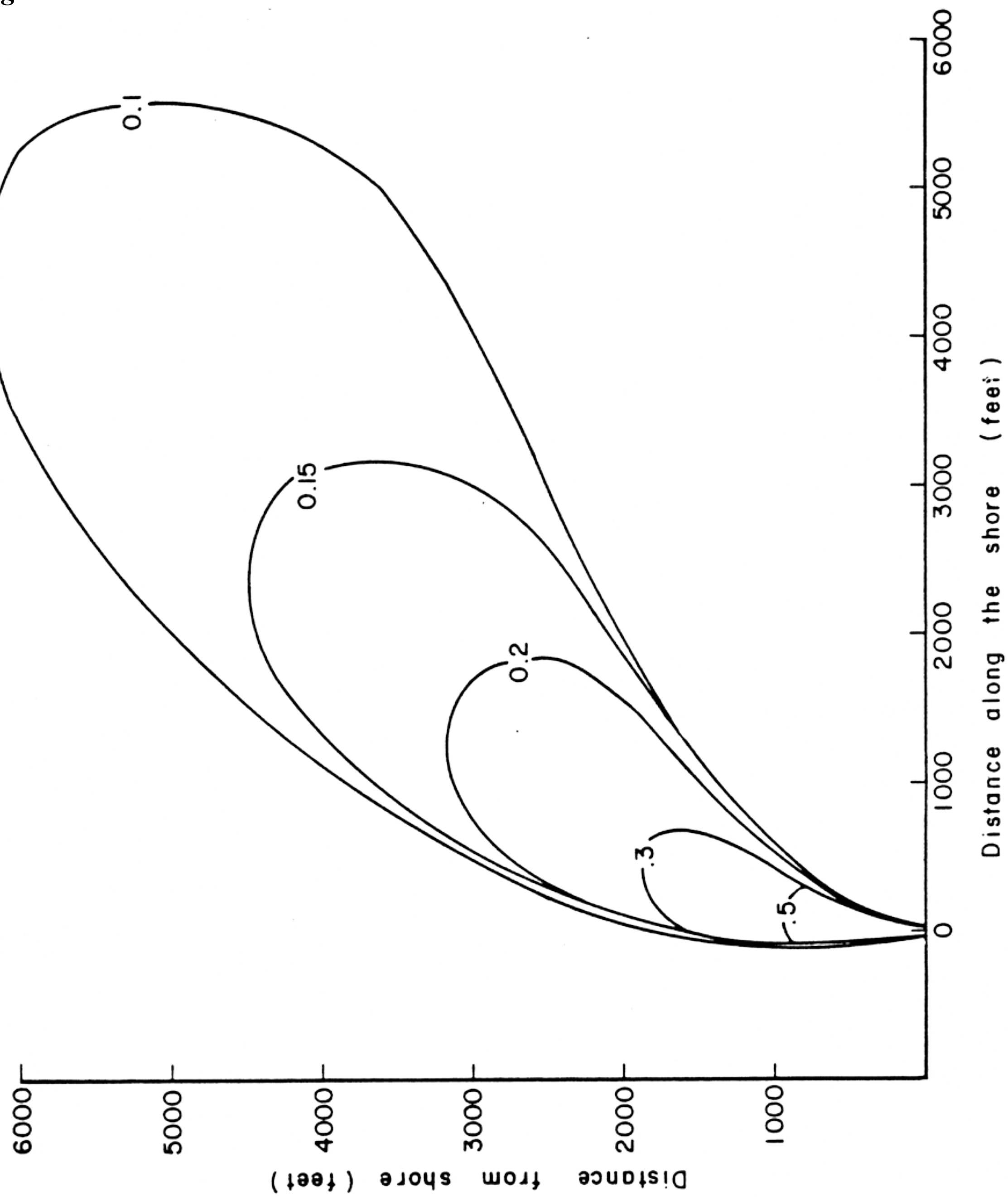

0.1
0.15
0.2
.3
.5
0
1000
2000
3000
4000
5000
6000
Distance along the shore (feet)
Distance from shore (feet)

Figure 10 Probable temperature elevation (degrees F) in upper six feet of Lake Ontario for horizontal canal discharge at the Brookwood site.

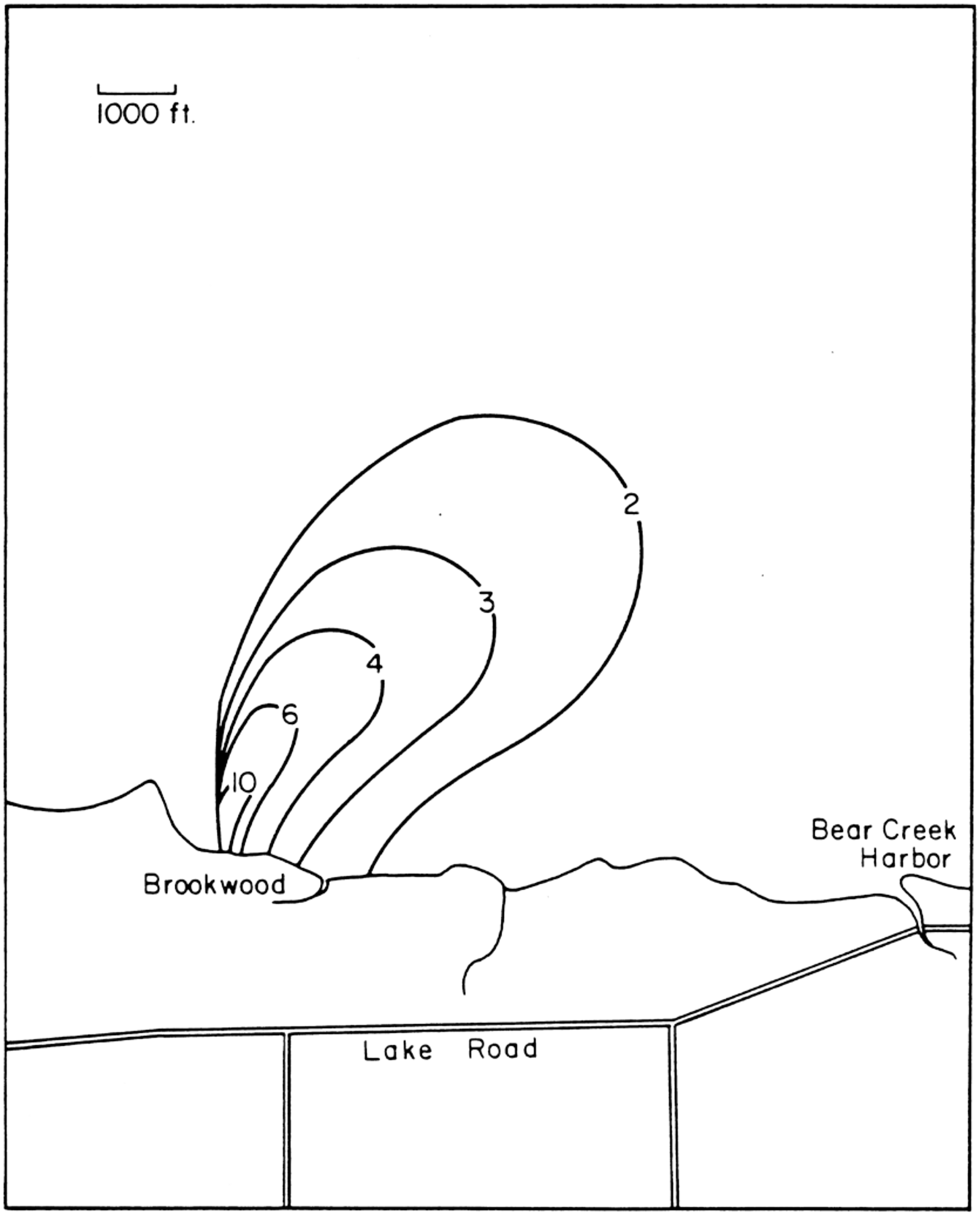
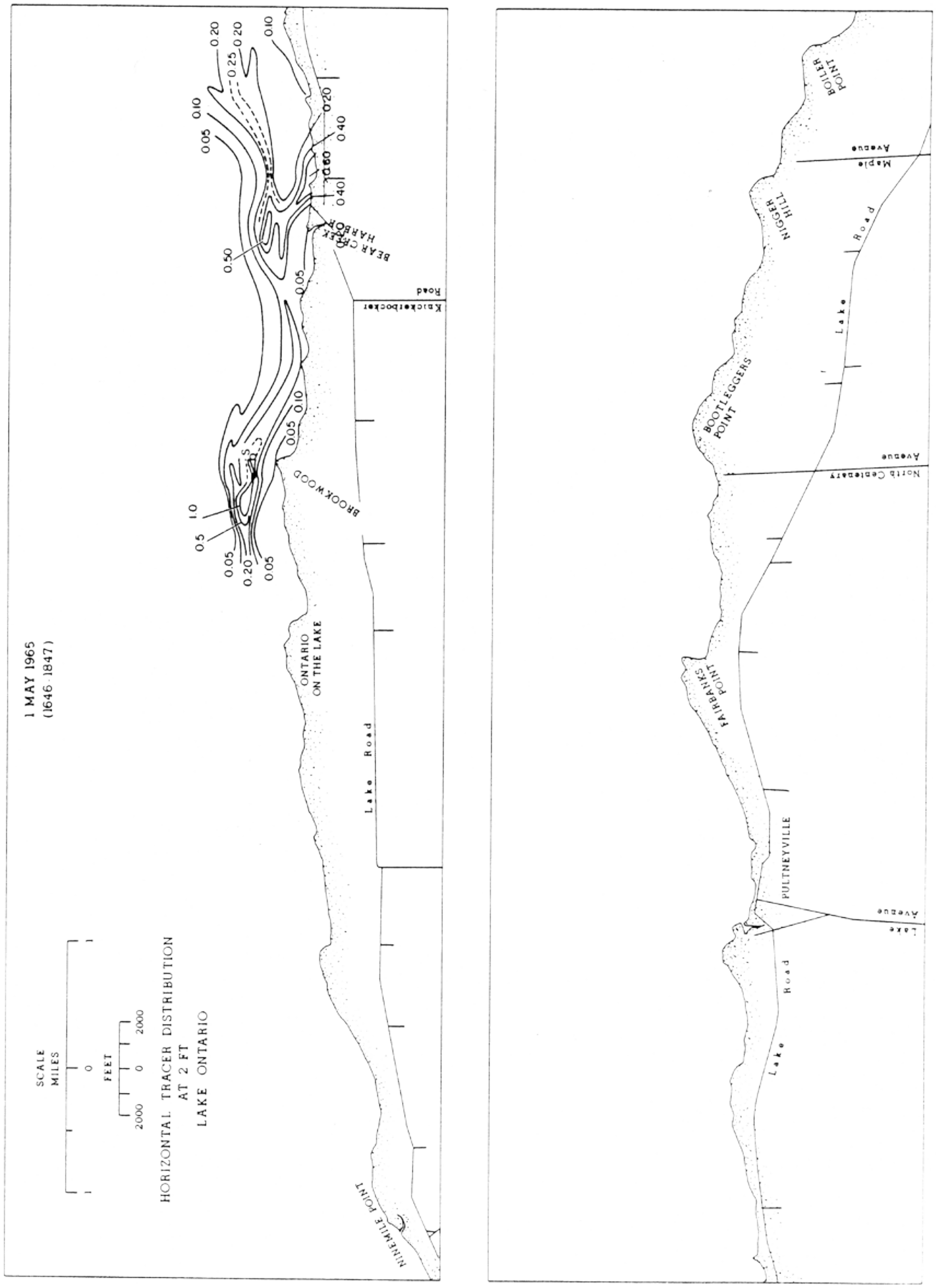
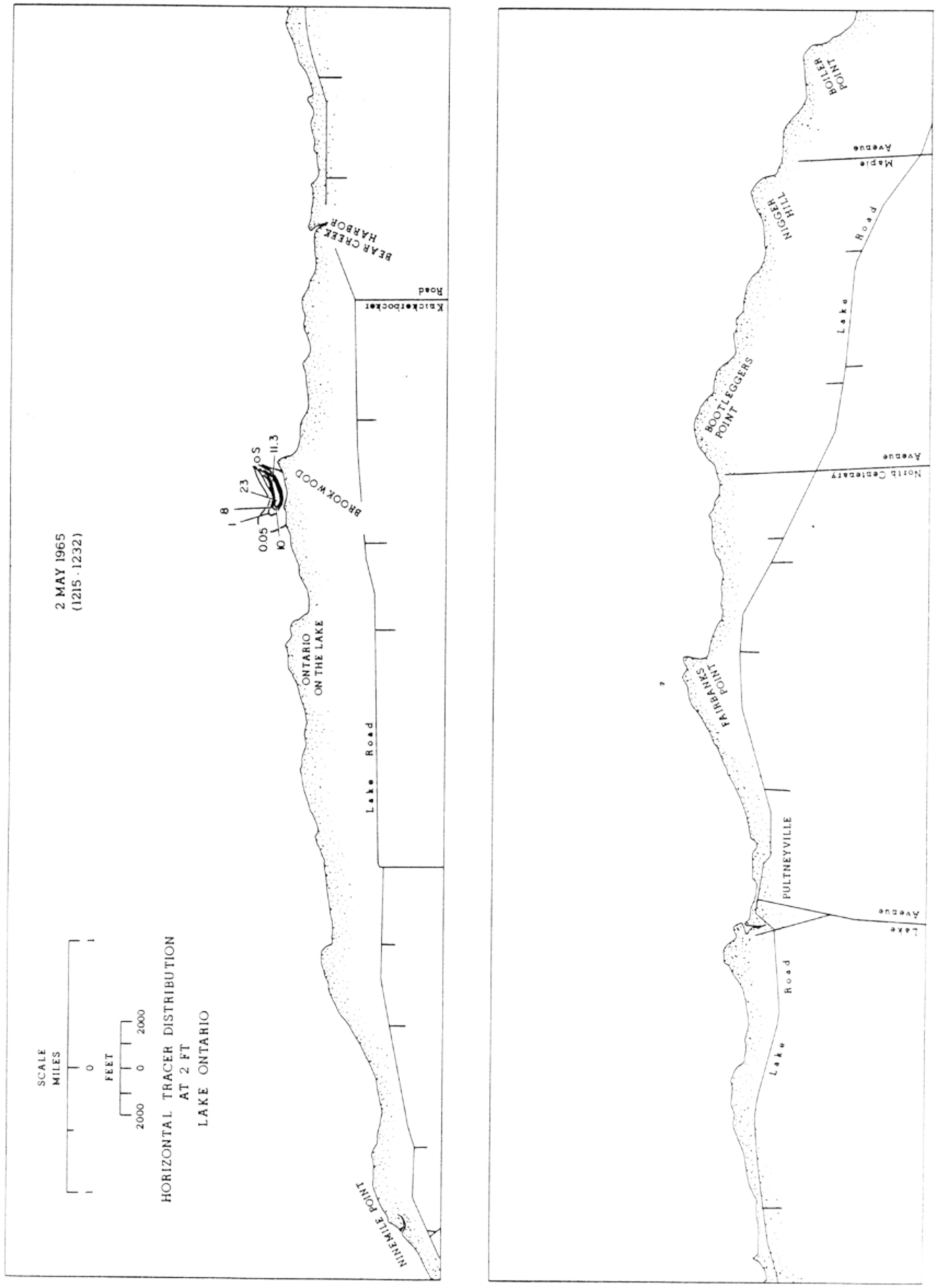


Figure 10. Probable temperature elevation (degrees F) in upper six feet of Lake Ontario for horizontal canal discharge at the Brookwood site.

Figure 1

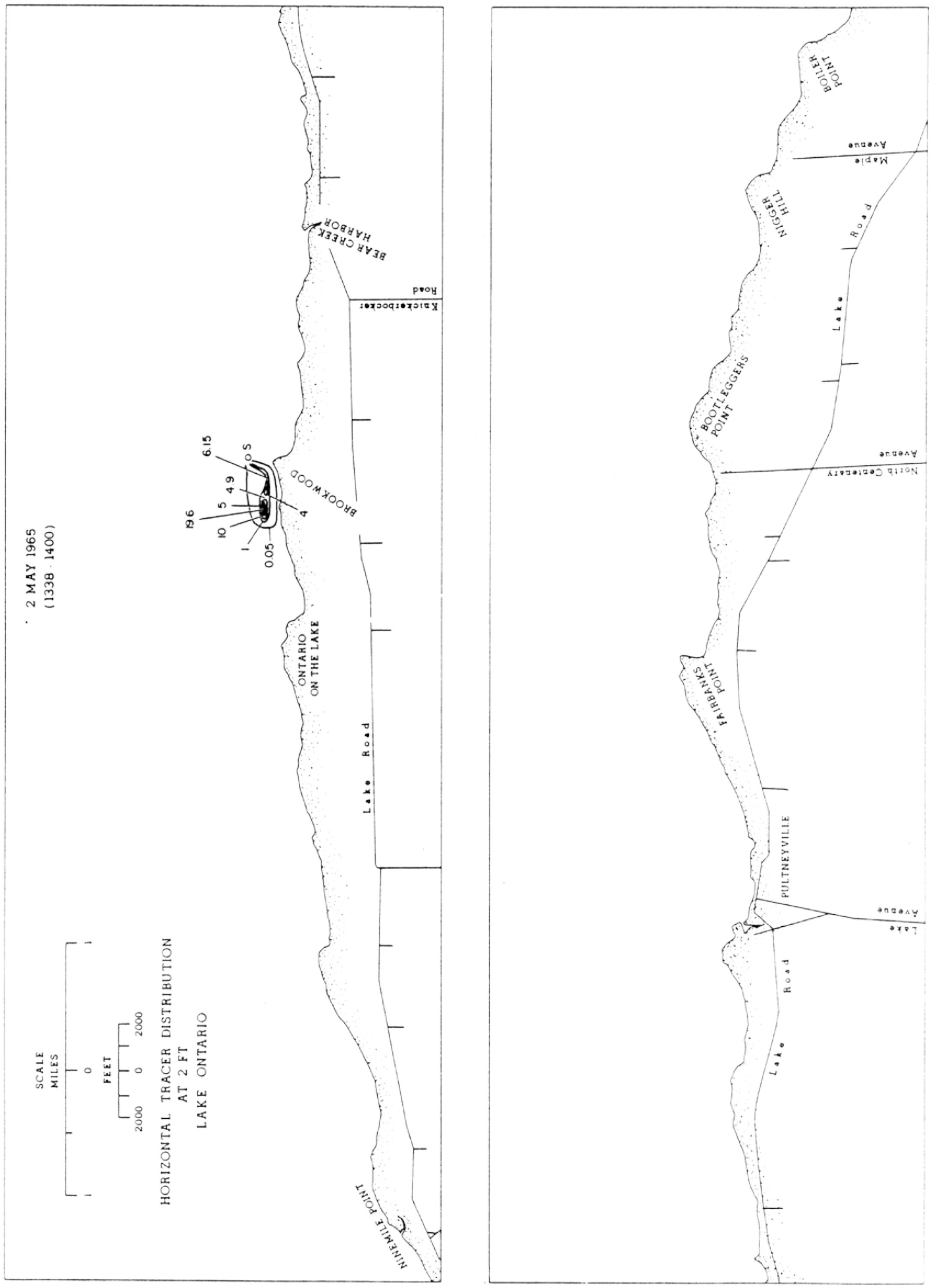

1 MAY 1965
(1646 1847)
SCALE
MILES
0
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
NINEMILE POINT
Lake Road
ONTARIO
ON THE LAKE
BROOKWOOD
BEAR CREEK
HARBOR
Knickerbocker Road
PULTNEYVILLE
FAIRBANKS
POINT
BOOTLEGGERS
POINT
North Centenary Avenue
NIGGER
HILL
Maple Avenue
BOILER
POINT
Lake Avenue

none


2 MAY 1965
(1215-1232)
SCALE
MILES
0
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
11.3
0.5
23
8
1
0.05
10
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

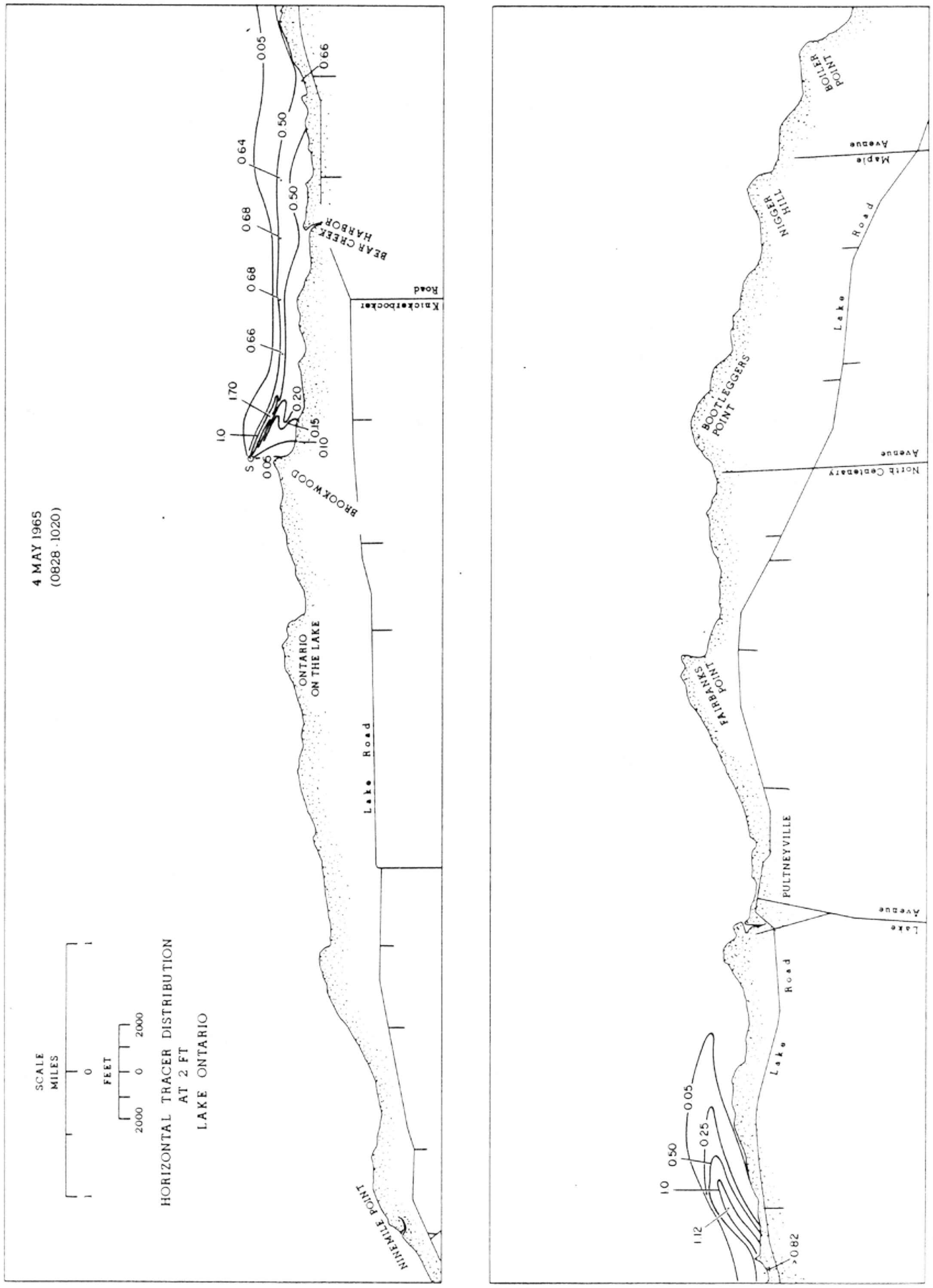
2B-44

none

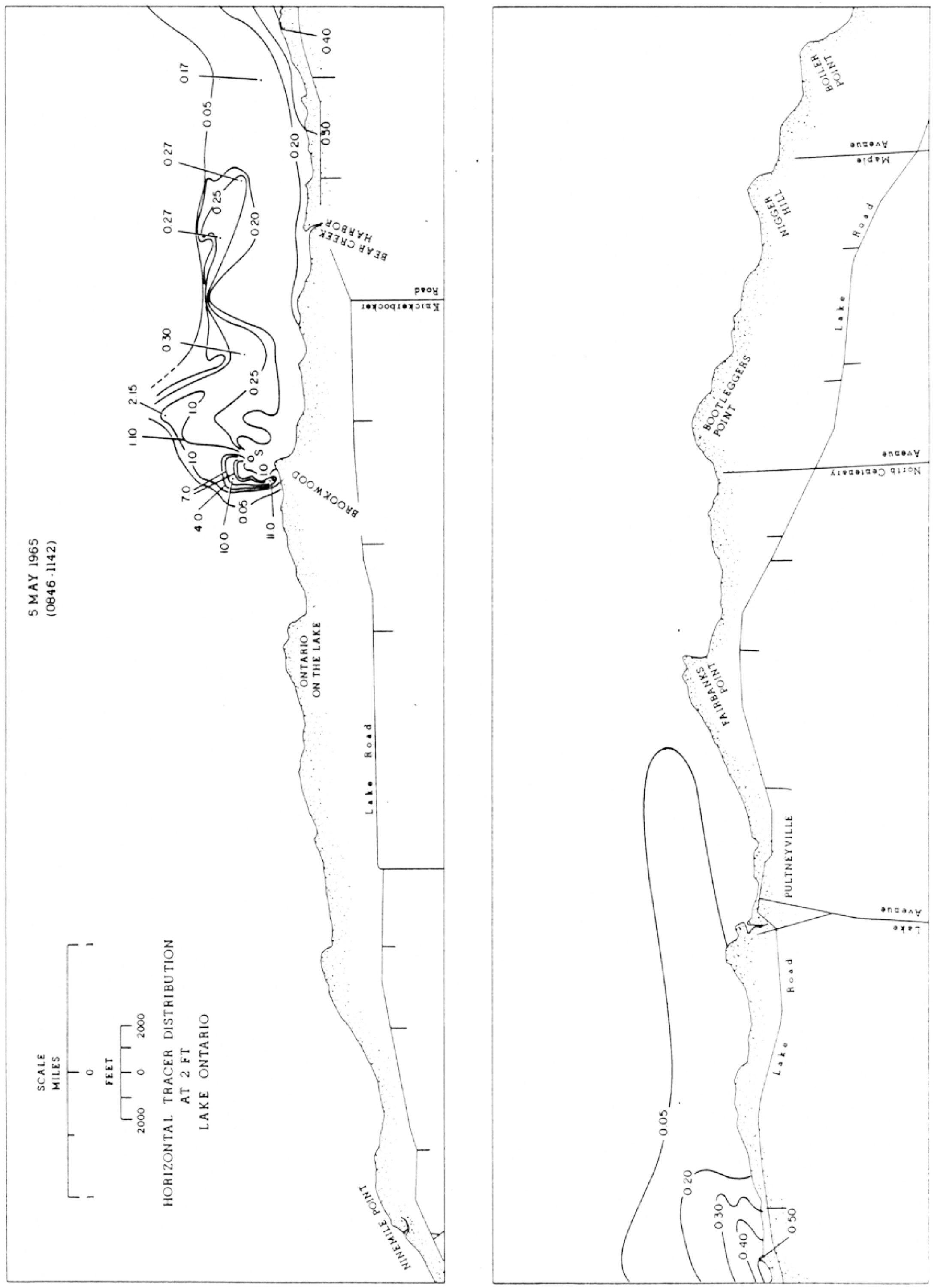

2 MAY 1965
(1338-1400)
SCALE
MILES
0
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
6.15
0.5
4.9
5
19.6
10
1
0.05
4
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

2B-45

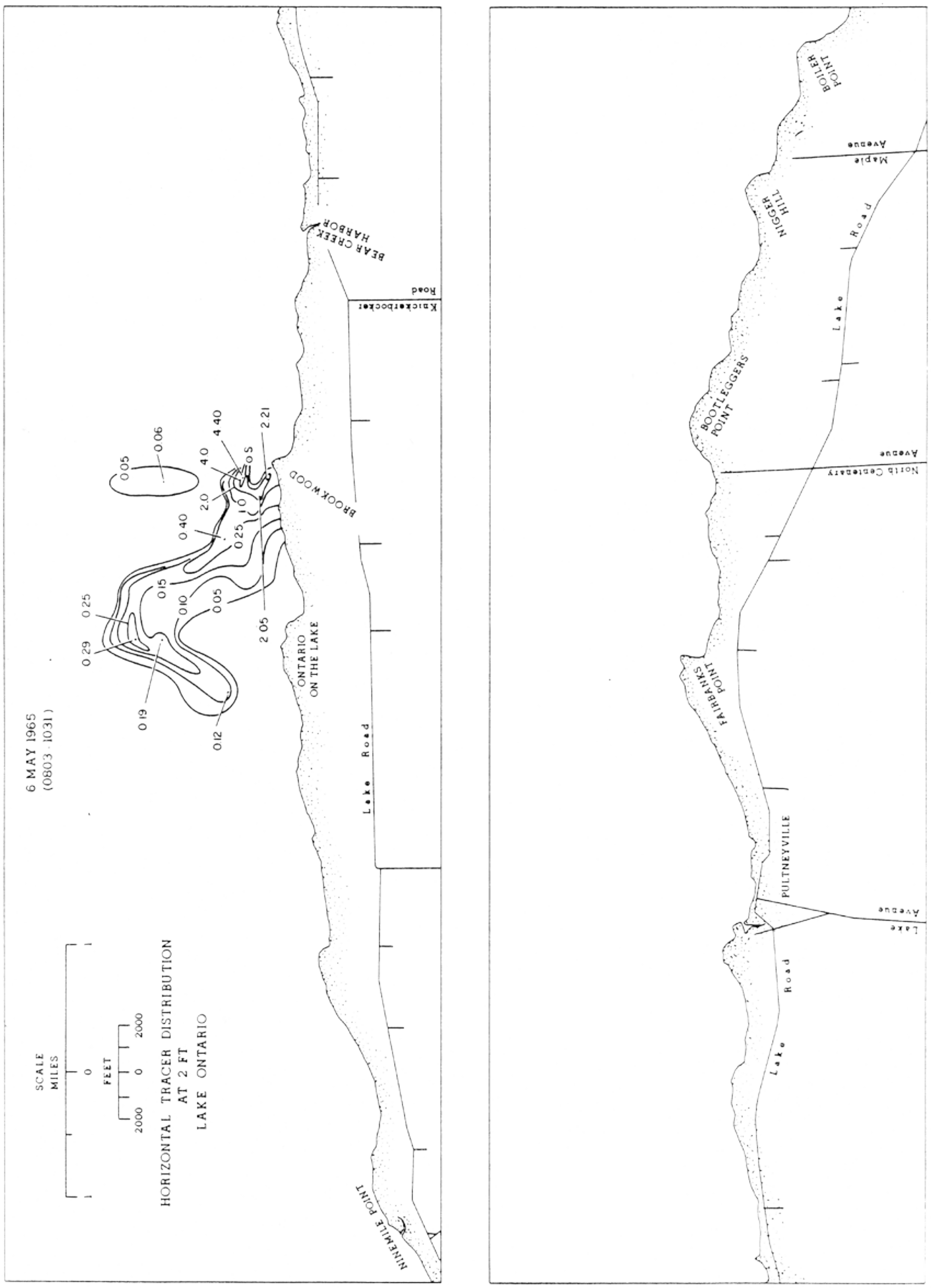
none


4 MAY 1965
(0828-1020)
SCALE
MILES
FEET
2000
0
2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

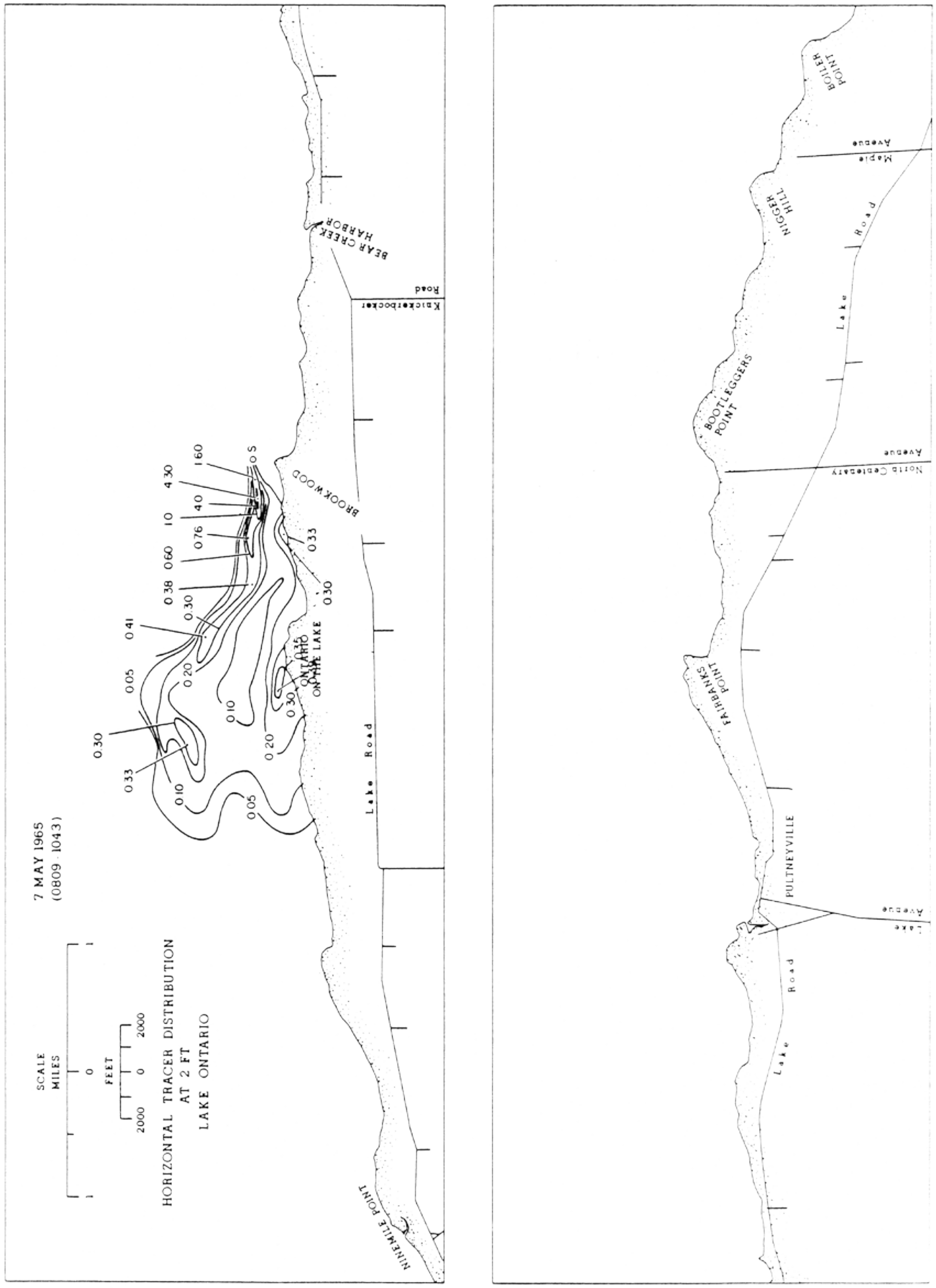
none


5 MAY 1965
(0846-1142)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
NINEMILE POINT
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Lake Road
Knickerbocker Road
FAIRBANKS POINT
BOOTLEGGERS POINT
NIGGER HILL
BOILER POINT
PULTNEYVILLE
North Centenary Avenue
Maple Avenue
Lake Avenue

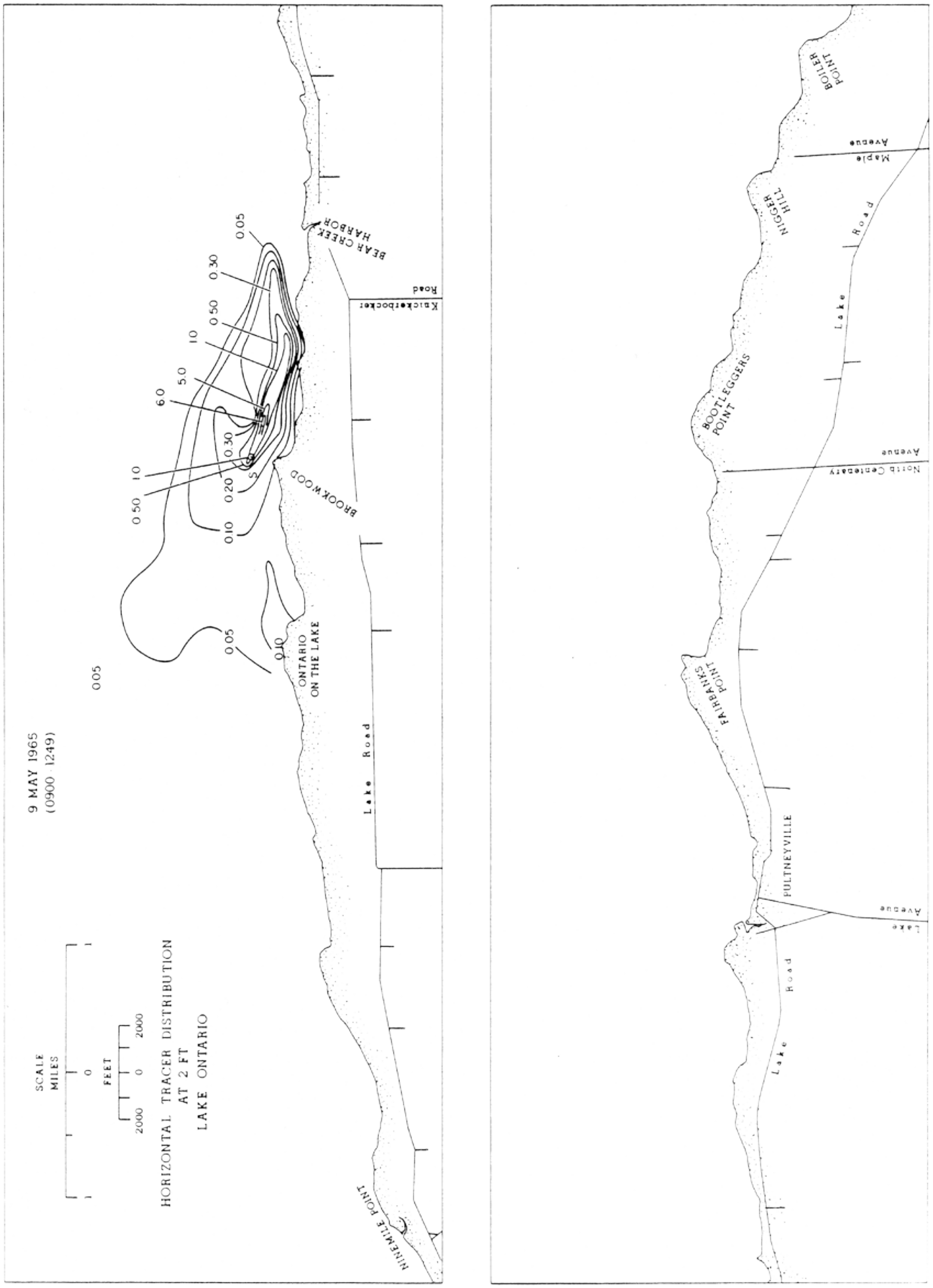
none


HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
6 MAY 1965
(0803-1031)
SCALE
MILES
0
FEET
2000 0 2000
NINEMILE POINT
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Knickerbocker Road
Lake Road
PULTNEYVILLE
FAIRBANKS POINT
BOOTLEGGERS POINT
NIGGER HILL
BOILER POINT
Maple Avenue
North Centenary Avenue
Lake Avenue
0.05
0.06
4.40
4.0
2.21
0.5
2.0
1.0
0.40
0.25
0.15
0.10
0.05
2.05
0.25
0.29
0.19
0.12

none

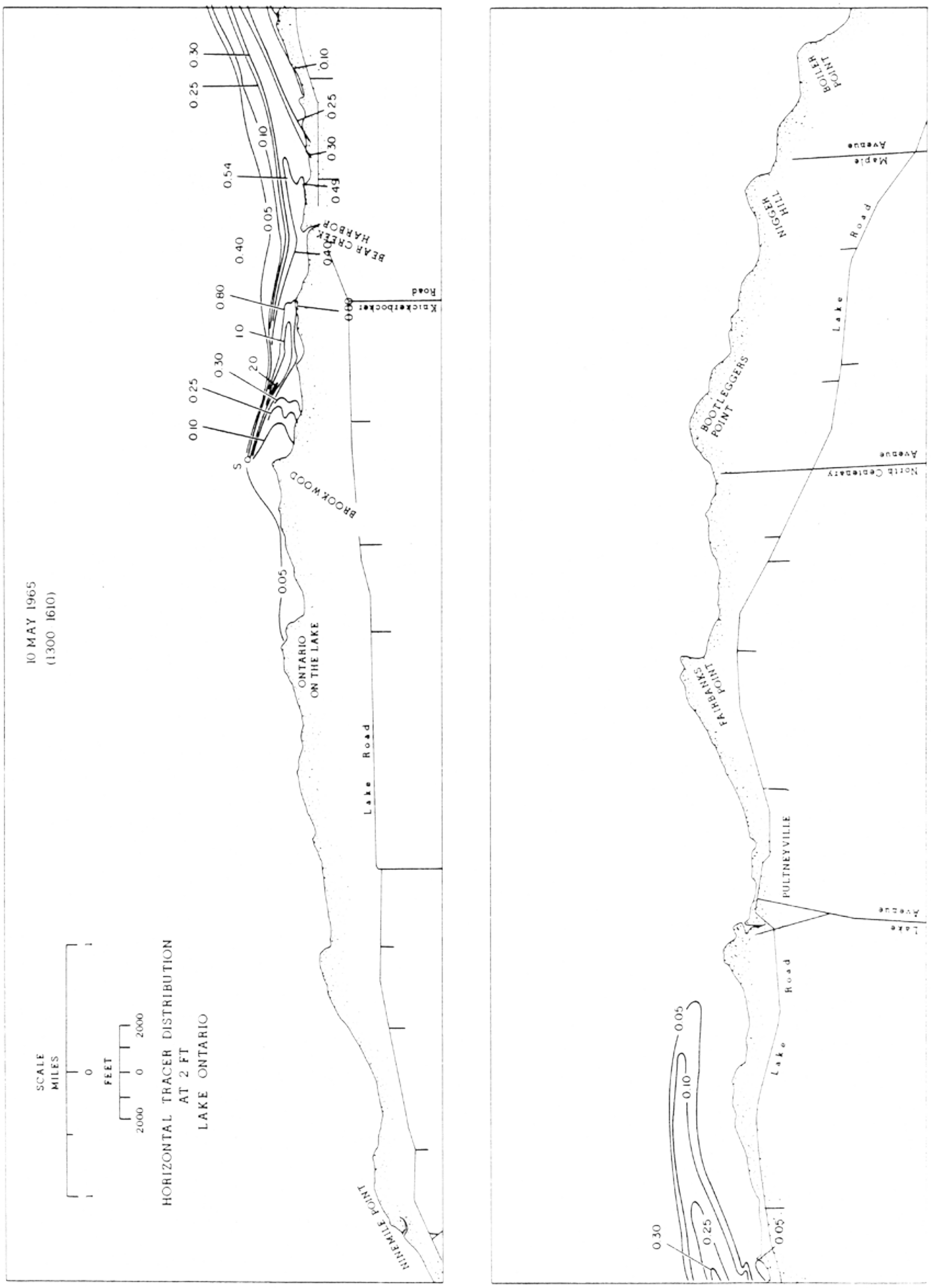

HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
7 MAY 1965
(0809 1043)
SCALE
MILES
FEET
2000 0 2000
NINEMILE POINT
Lake Road
ONTARIO ON THE LAKE
BROOKWOOD
Knickerbocker Road
BEAR CREEK HARBOR
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none


HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
9 MAY 1965
(0900 1249)
SCALE
MILES
FEET
2000 0 2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

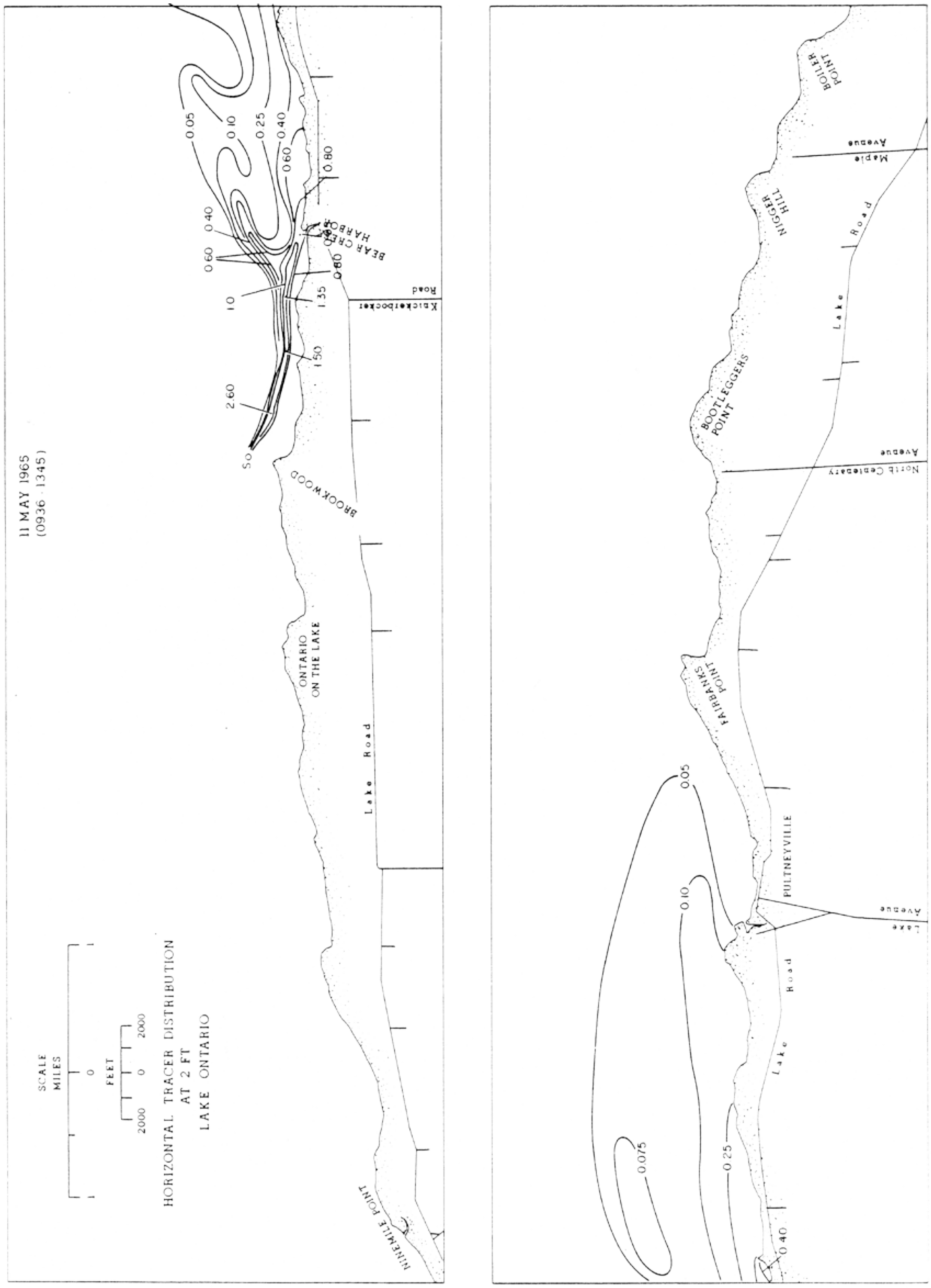
2B-50

none

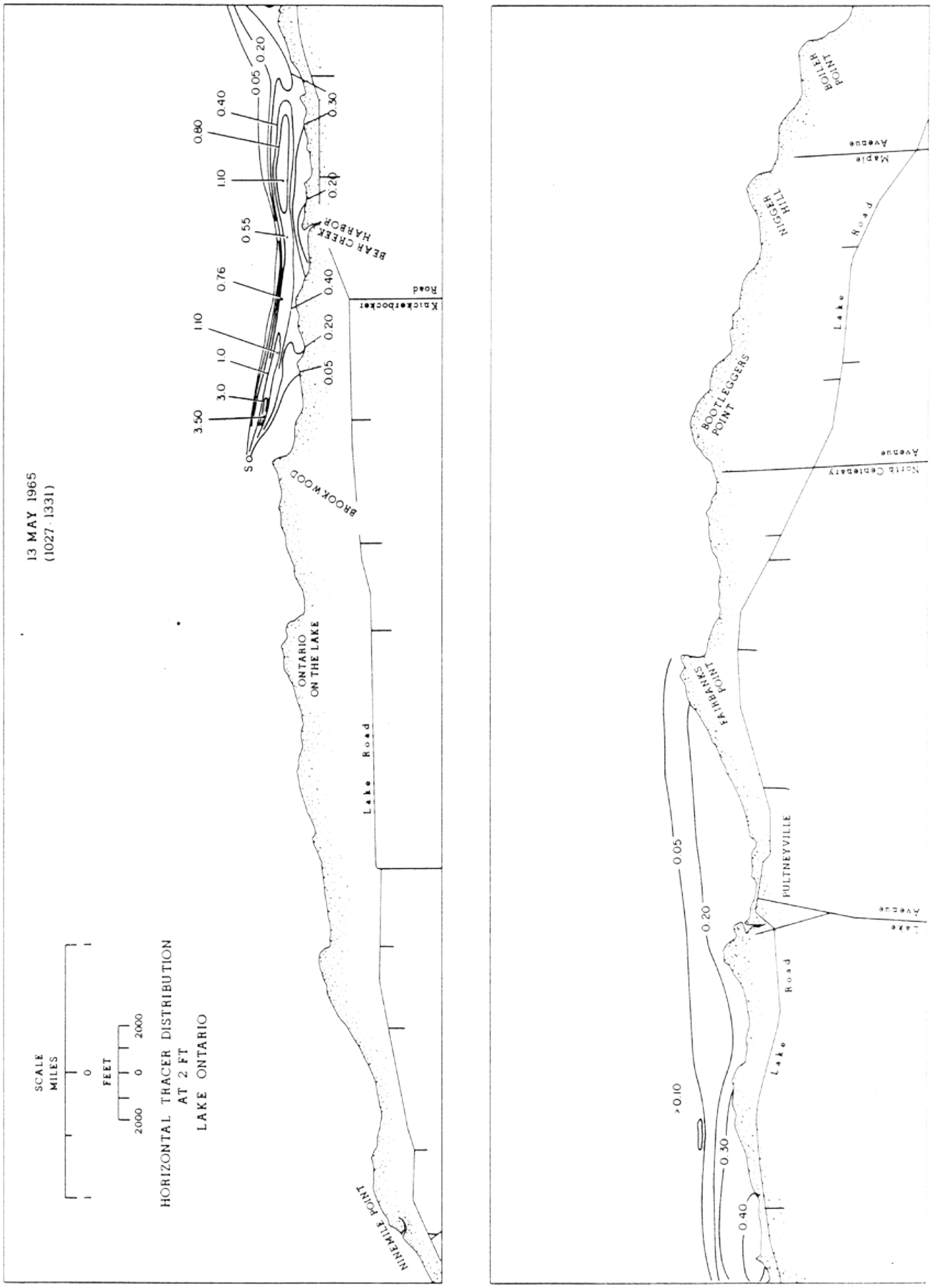

10 MAY 1965
(1300 1610)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

2B-51

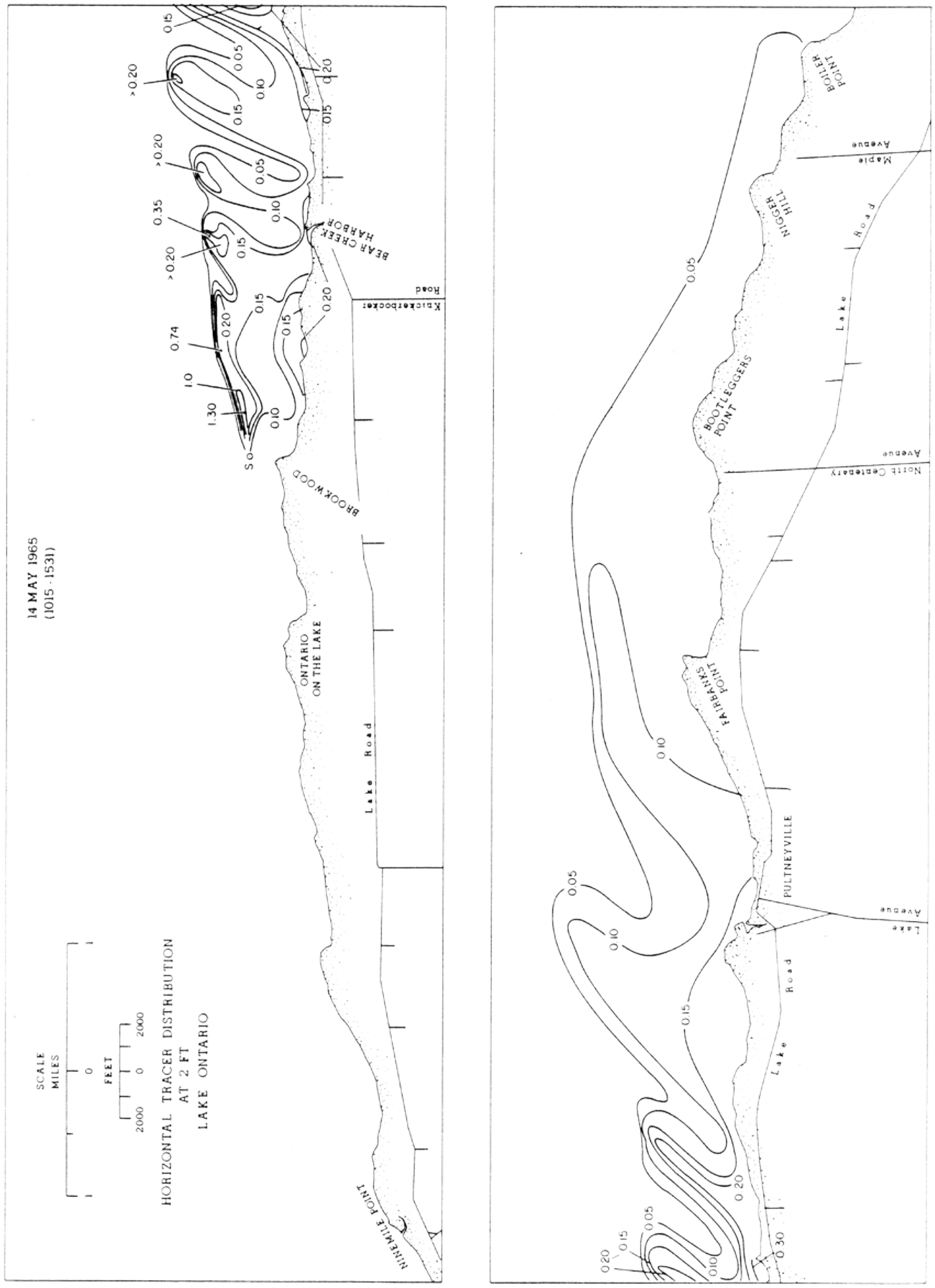
none


11 MAY 1965
(0936 - 1345)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000 0 2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
0.05
0.10
0.25
0.40
0.60
0.80
1.0
1.35
1.50
2.60
0.075

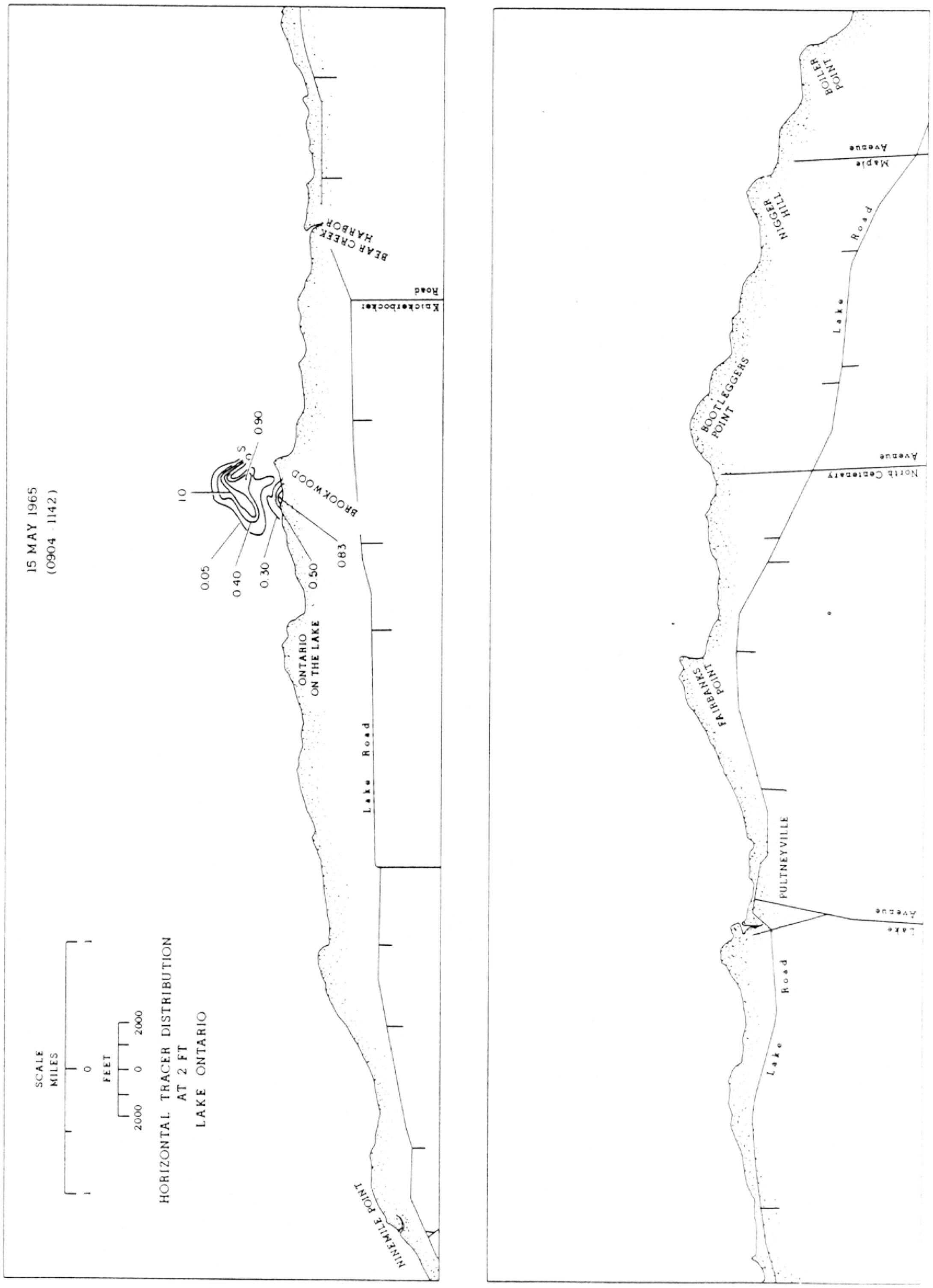
none


HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
13 MAY 1965
(1027-1331)
SCALE
MILES
FEET
2000 0 2000
NINEMILE POINT
Lake Road
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Knickerbocker Road
S
3.50
3.0
1.0
1.10
0.76
0.55
1.10
0.80
0.40
0.05
0.20
0.30
0.40
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road
>0.10

none

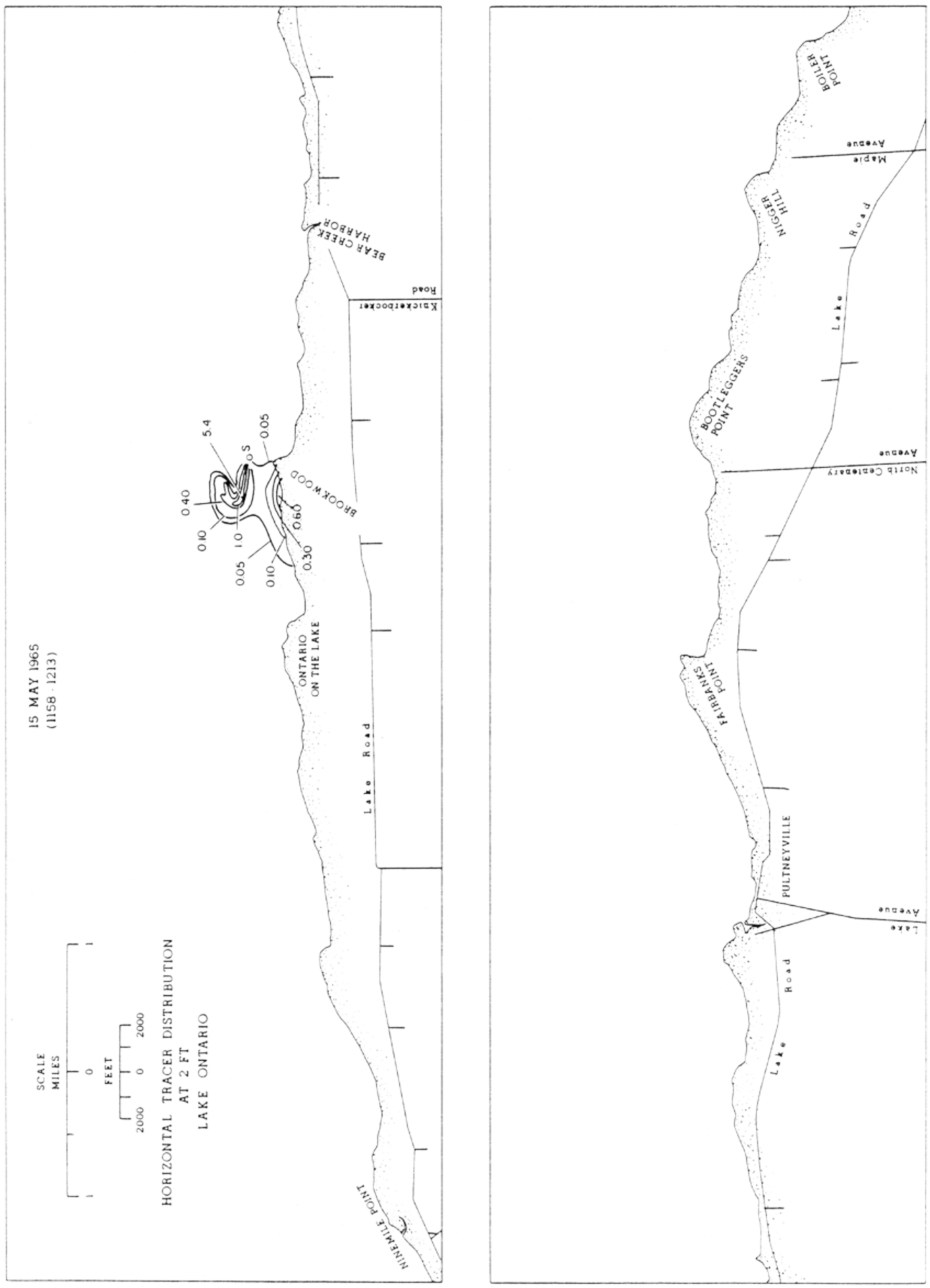

14 MAY 1965
(1015 - 1531)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
NINEMILE POINT
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Knickerbocker Road
Lake Road
BOOTLEGGERS POINT
FAIRBANKS POINT
PULTNEYVILLE
BOILER POINT
North Centenary Avenue
Maple Avenue
Lake Avenue

none

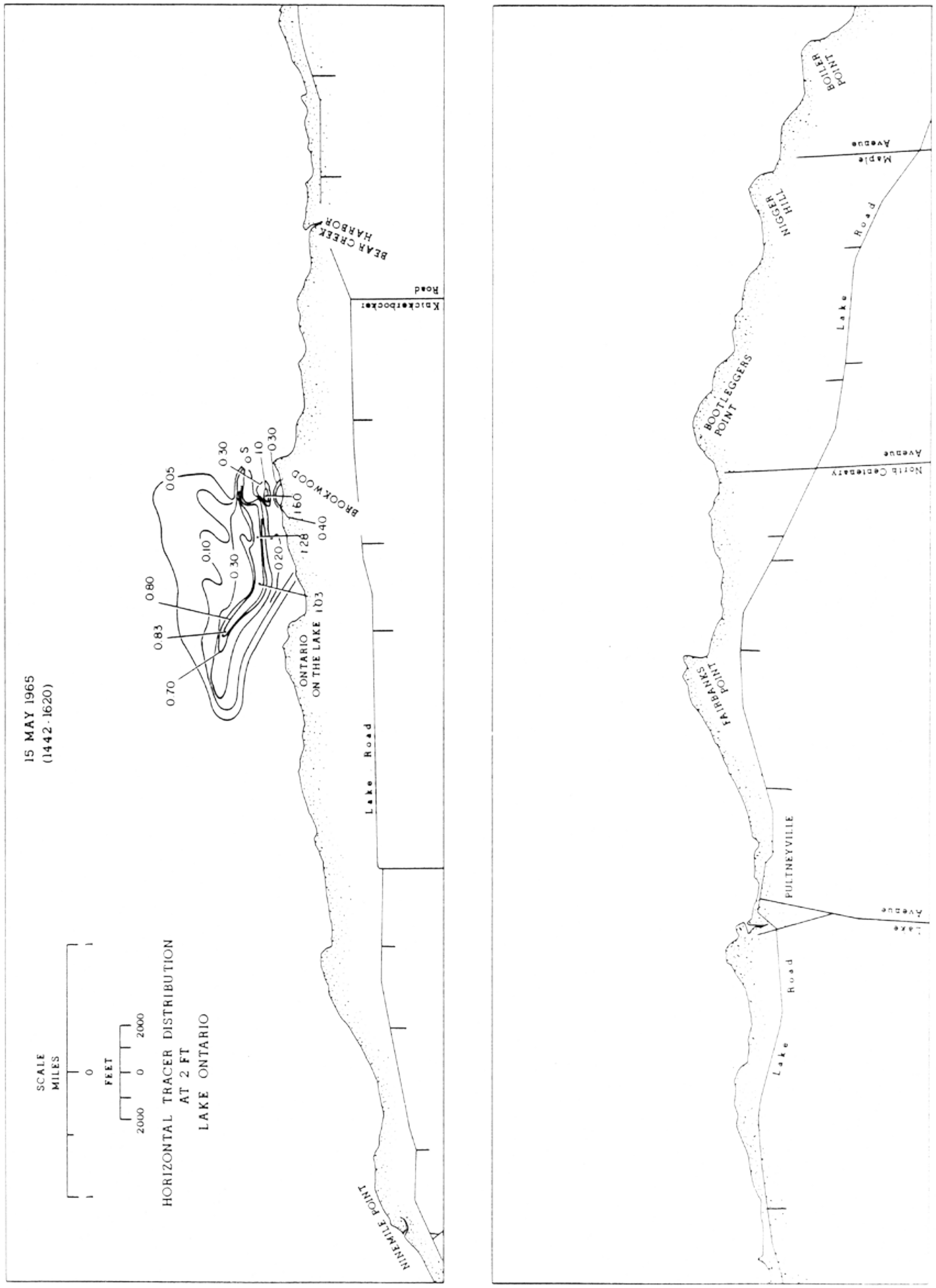

15 MAY 1965
(0904 - 1142)
SCALE
MILES
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
1.0
0.5
0.90
0.05
0.40
0.30
0.50
0.83
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

2B-55

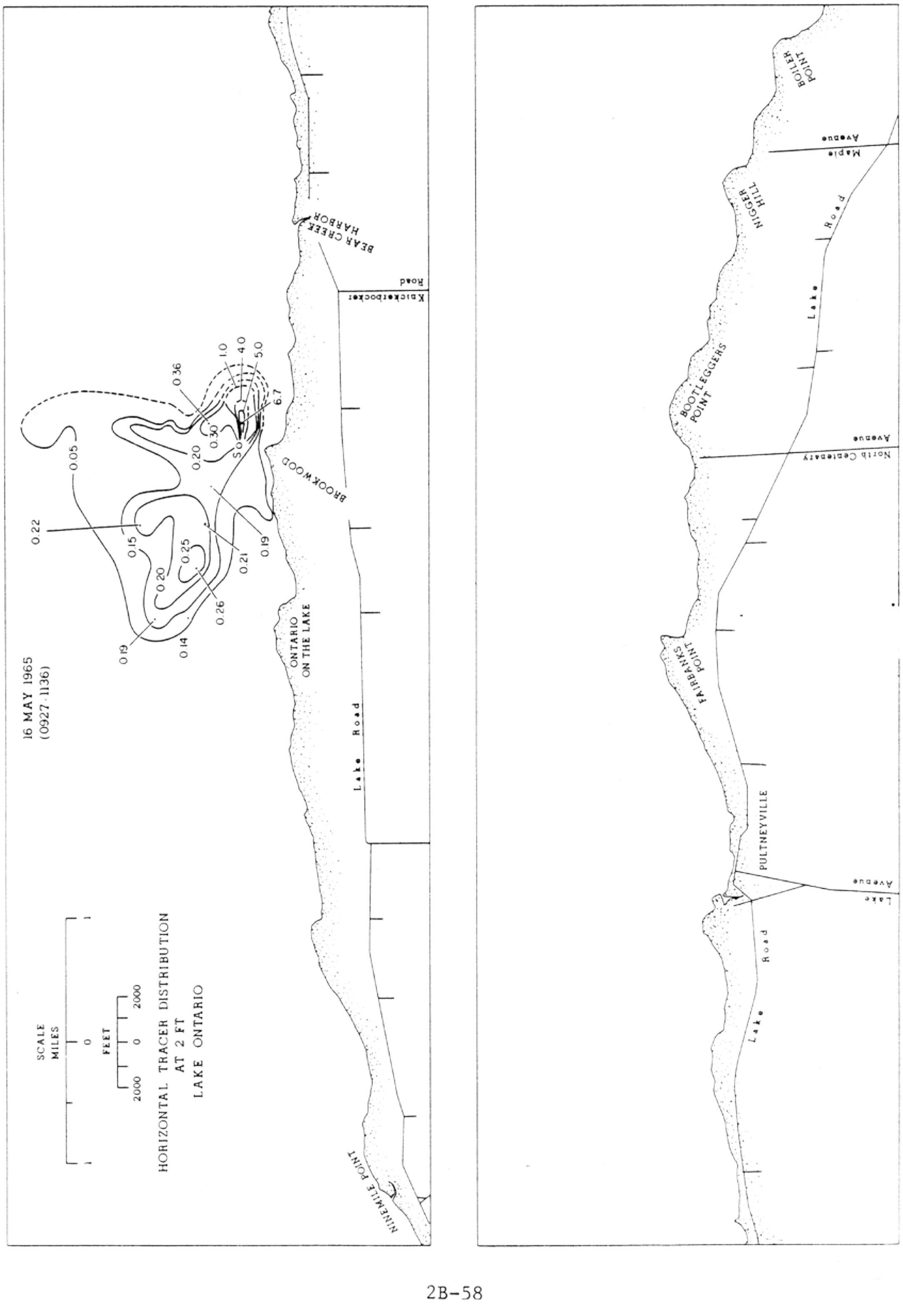
none


SCALE
MILES
0
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
15 MAY 1965
(1158-1213)
NINEMILE POINT
ONTARIO ON THE LAKE
Lake Road
BROOKWOOD
BEAR CREEK HARBOR
Knickerbocker Road
0.05
5.4
0.40
0.10
1.0
0.60
0.30
FAIRBANKS POINT
BOOTLEGGERS POINT
NIGGER HILL
BOILER POINT
PULTNEYVILLE
Lake Avenue
North Centenary Avenue
Maple Avenue
Lake Road

none


15 MAY 1965
(1442-1620)
SCALE
MILES
FEET
2000
0
2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none


HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
16 MAY 1965
(0927-1136)
SCALE
MILES
FEET
2000 0 2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

2B-58

none

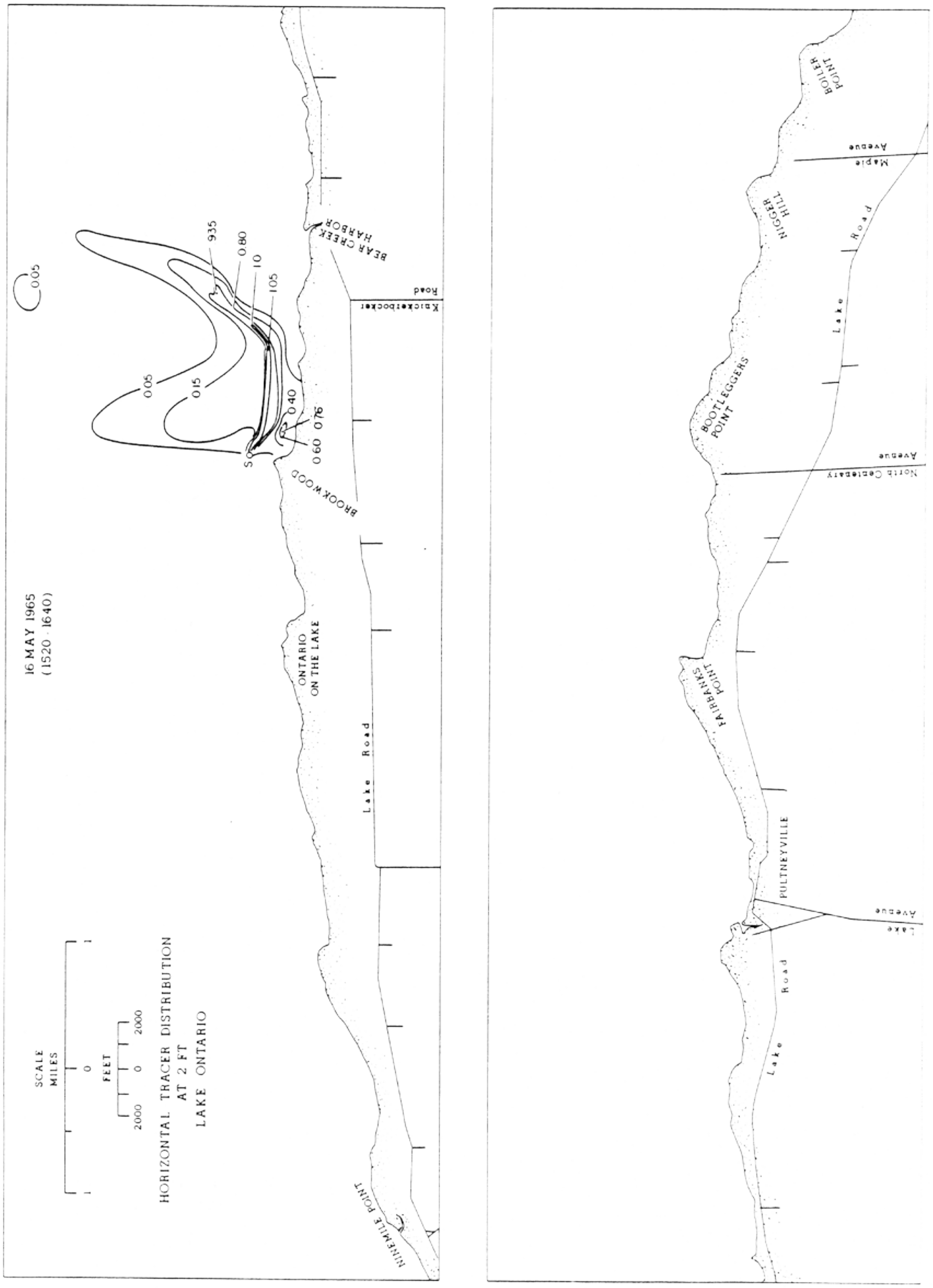
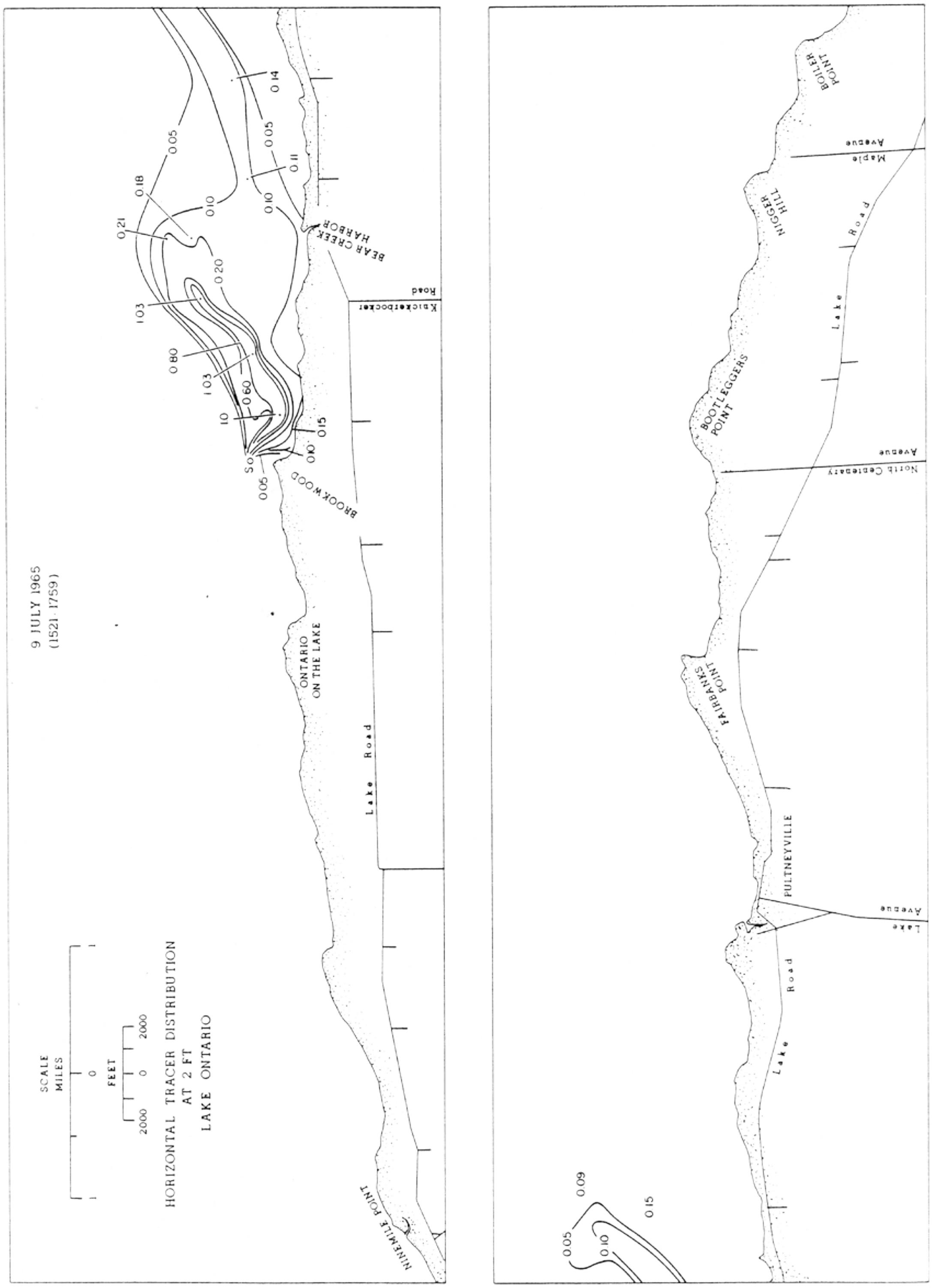

16 MAY 1965
(1520-1640)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

Figure 2


9 JULY 1965
(1521-1759)
SCALE
MILES
FEET
2000
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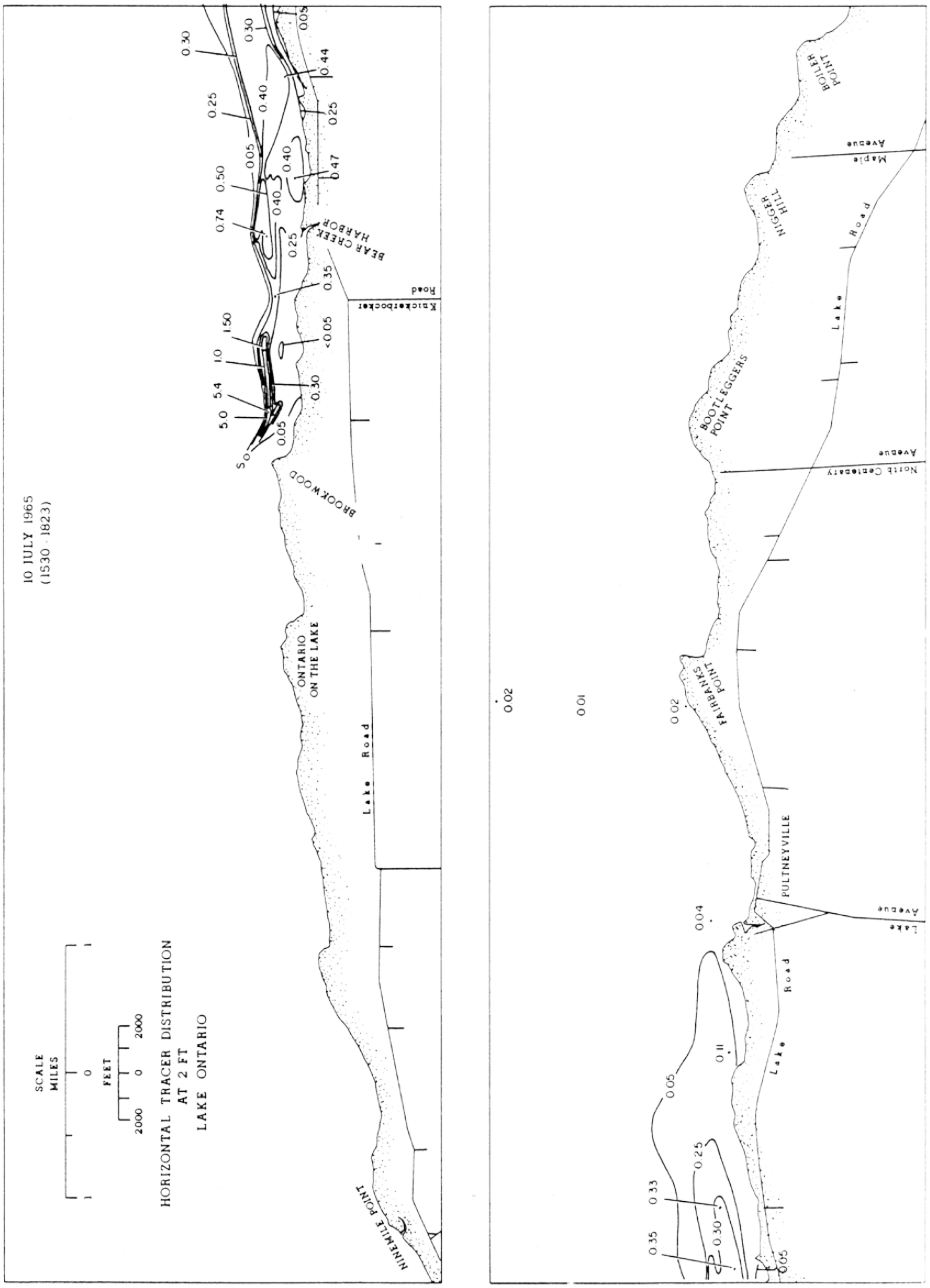
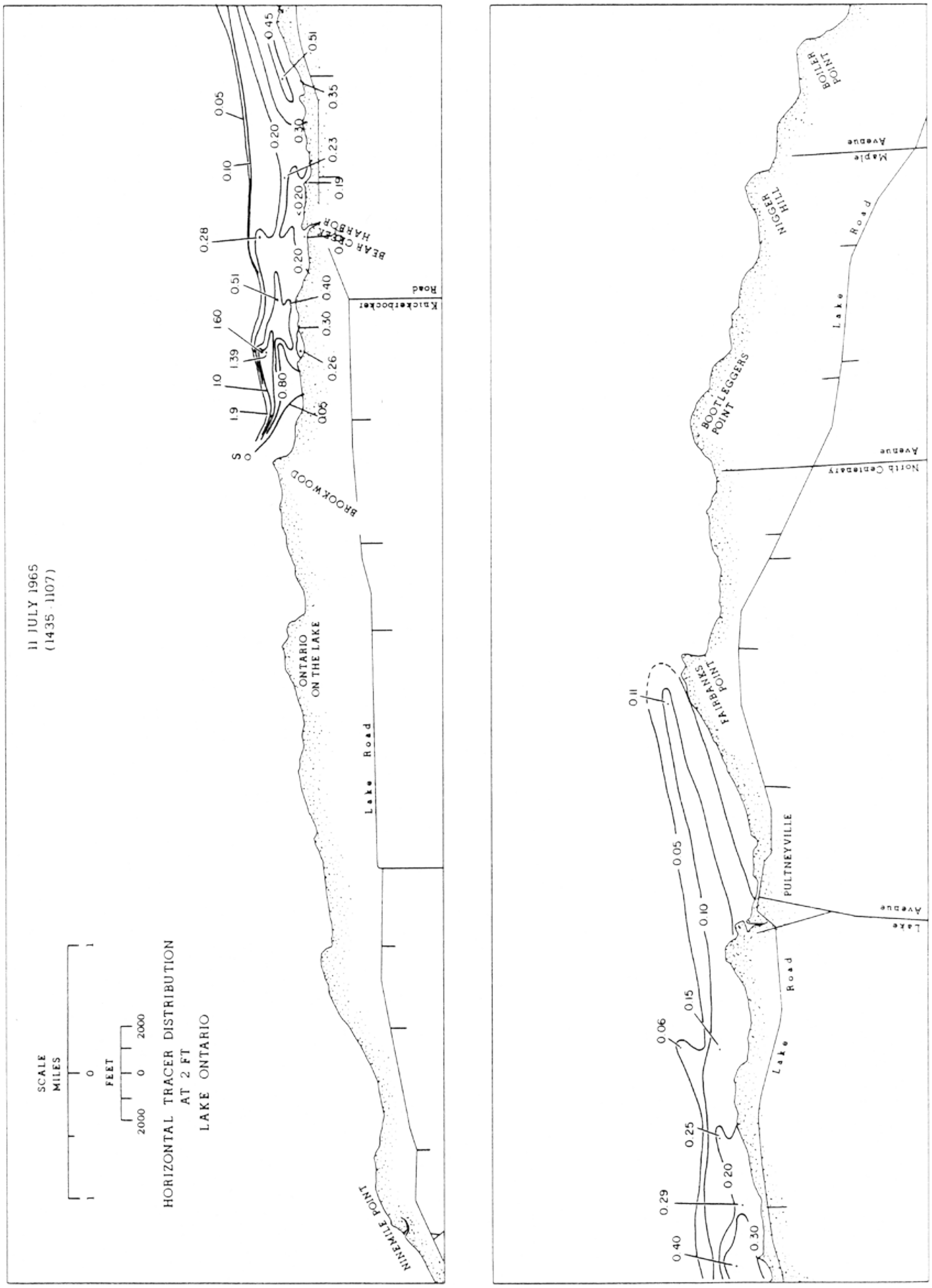
2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none

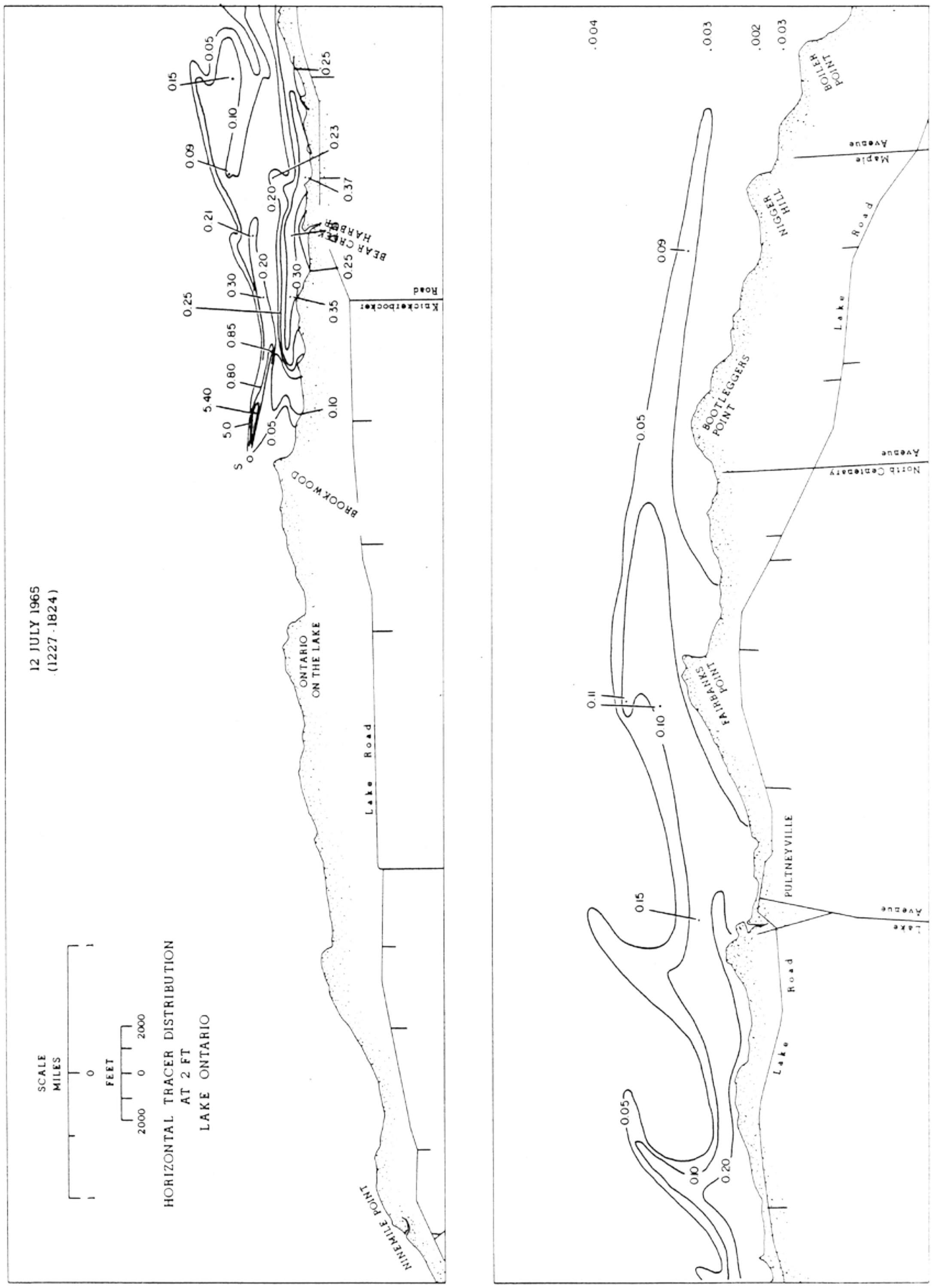

10 JULY 1965
(1530 - 1823)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
NINEMILE POINT
BROOKWOOD
ONTARIO
ON THE LAKE
Lake Road
Knickerbocker Road
BEAR CREEK
HARBOR
BOILER
POINT
Maple Avenue
NIGGER
HILL
BOOTLEGGERS
POINT
North Cattenary Avenue
FAIRBANKS
POINT
PULTNEYVILLE
Lake Avenue

none


11 JULY 1965
(1435-1107)
SCALE
MILES
FEET
2000
0
2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Road
Lake

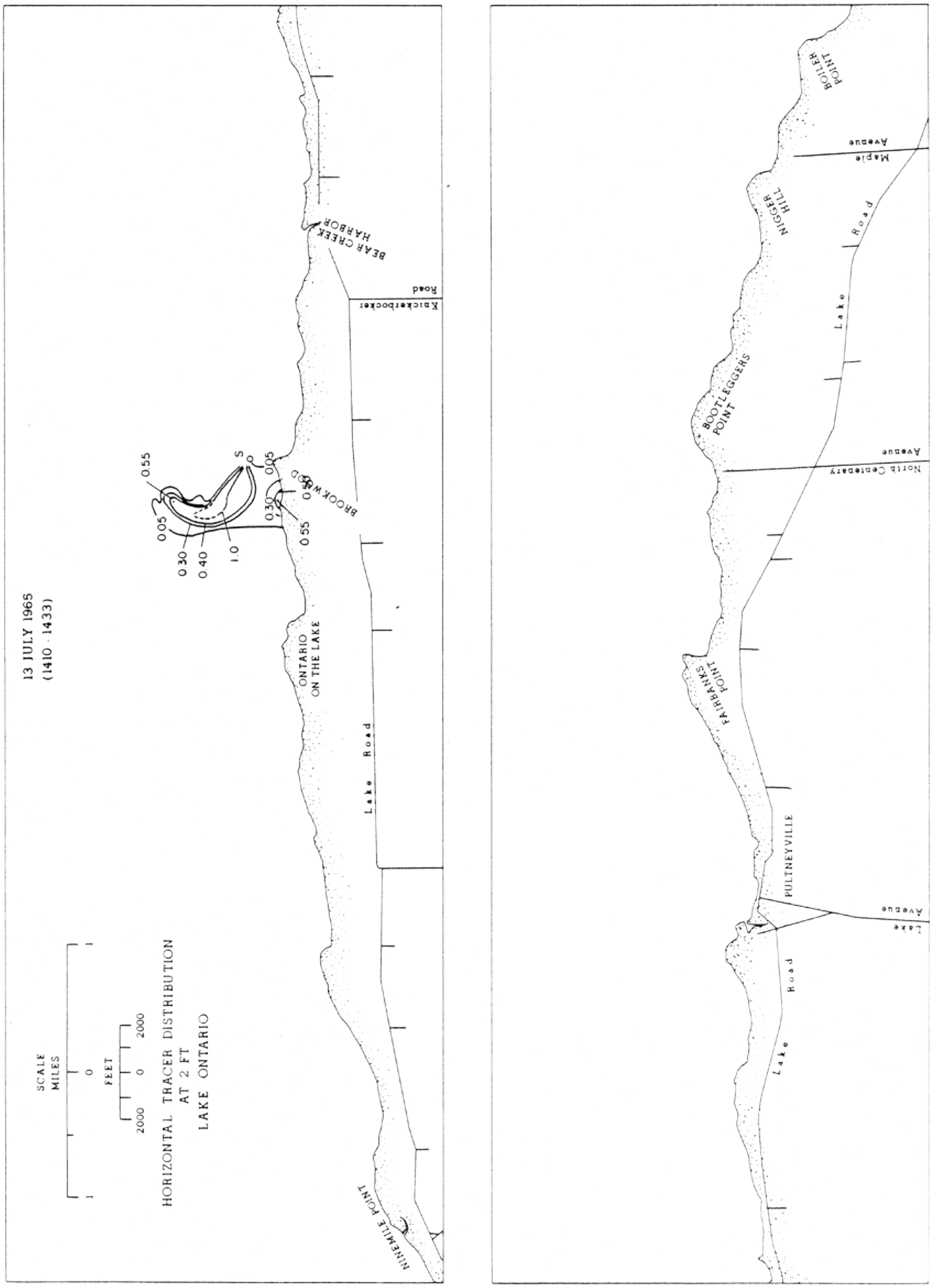
2B-62

none

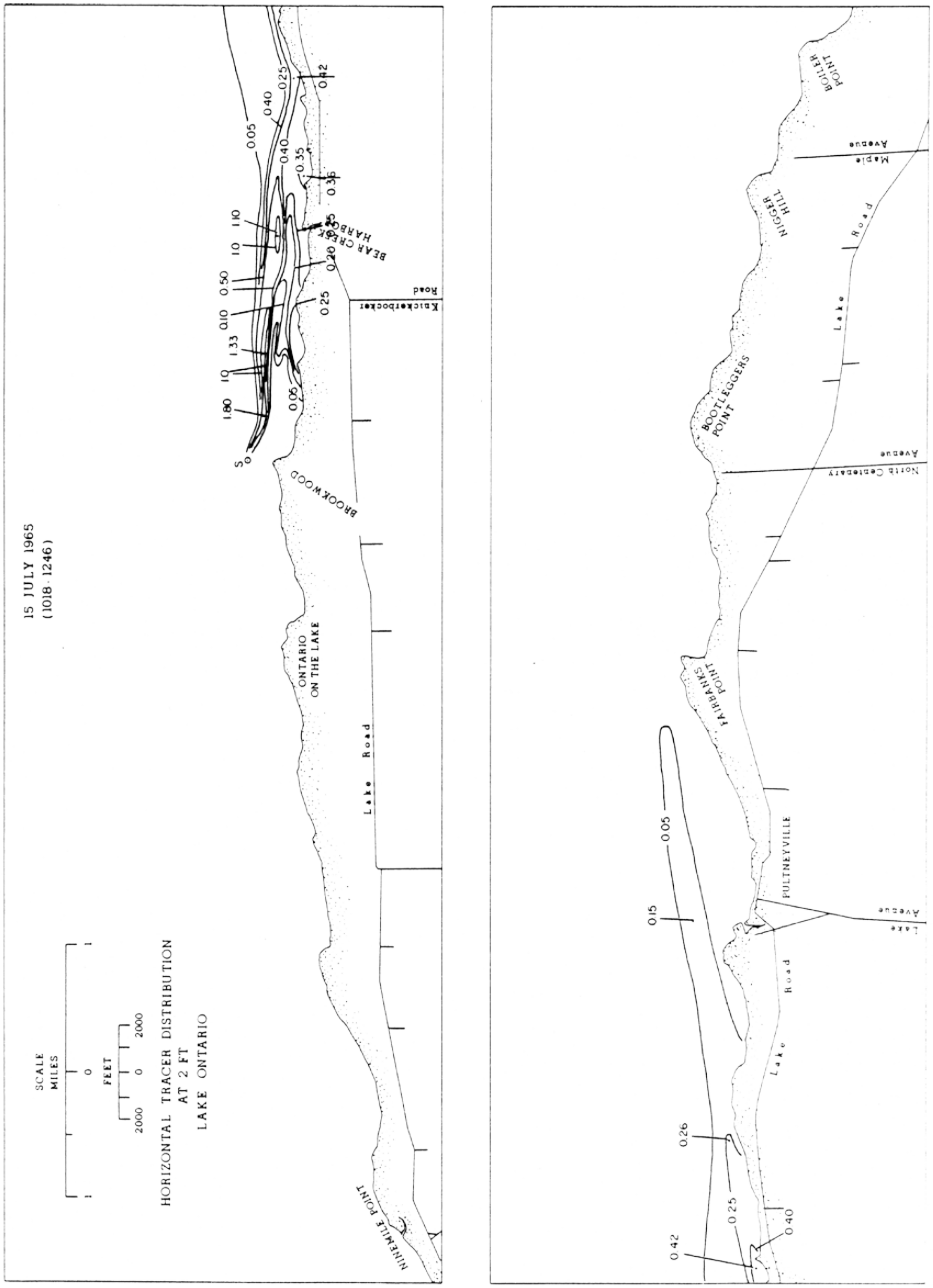

12 JULY 1965
(1227-1824)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000 0 2000
NINEMILE POINT
BROOKWOOD
ONTARIO ON THE LAKE
BEAR CREEK HARBOR
Knickerbocker Road
Lake Road
PULTNEYVILLE
FAIRBANKS POINT
BOOTLEGGERS POINT
NIGGER HILL
BOILER POINT
North Centenary Avenue
Maple Avenue
Lake Avenue

2B-63

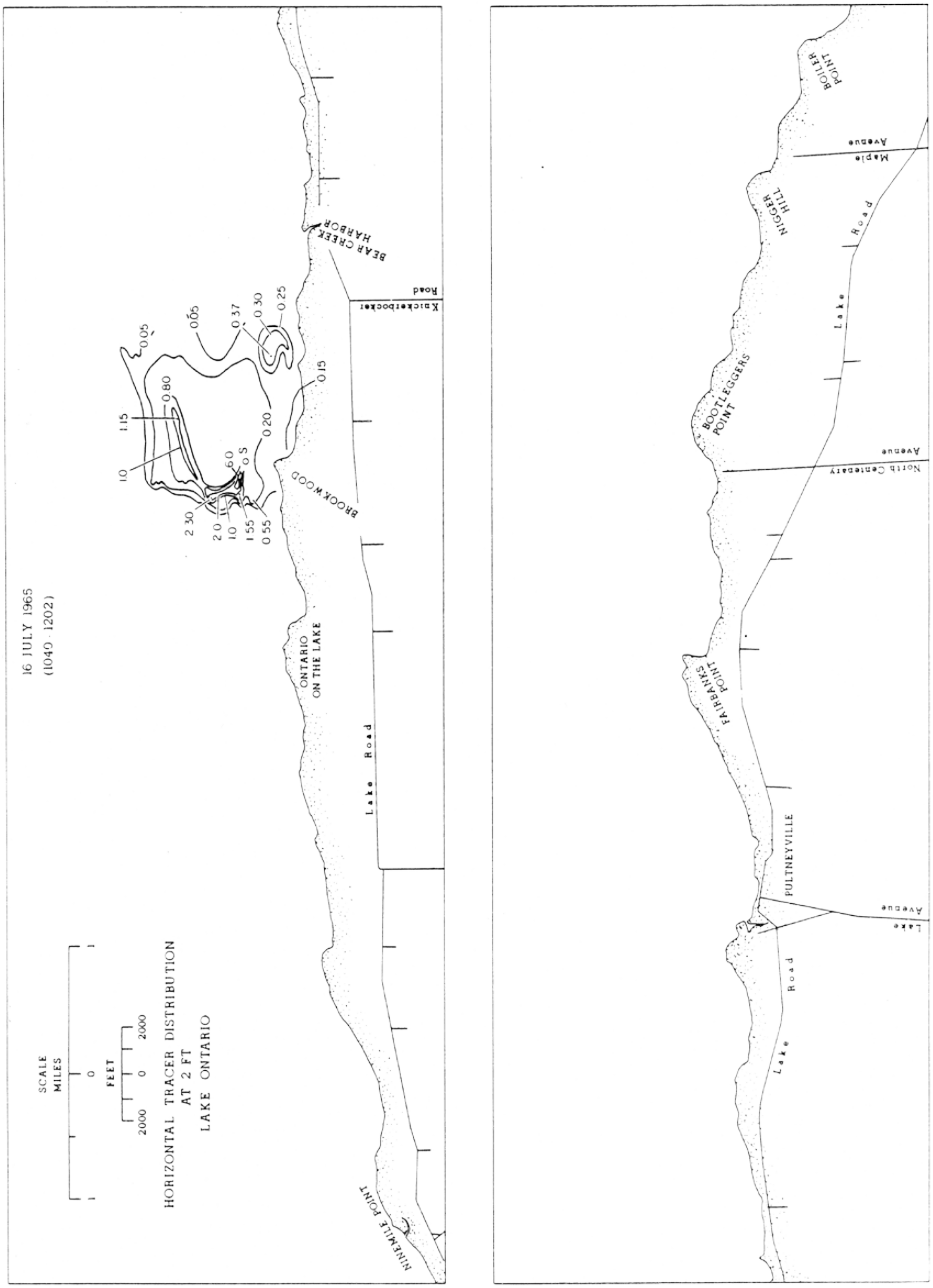
none


13 JULY 1965
(1410 - 1433)
SCALE
MILES
0
FEET
2000
0
2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
0.55
0.05
0.30
0.40
1.0
0.05
0.30
0.55
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOLLER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none


15 JULY 1965
(1018-1246)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none



HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO

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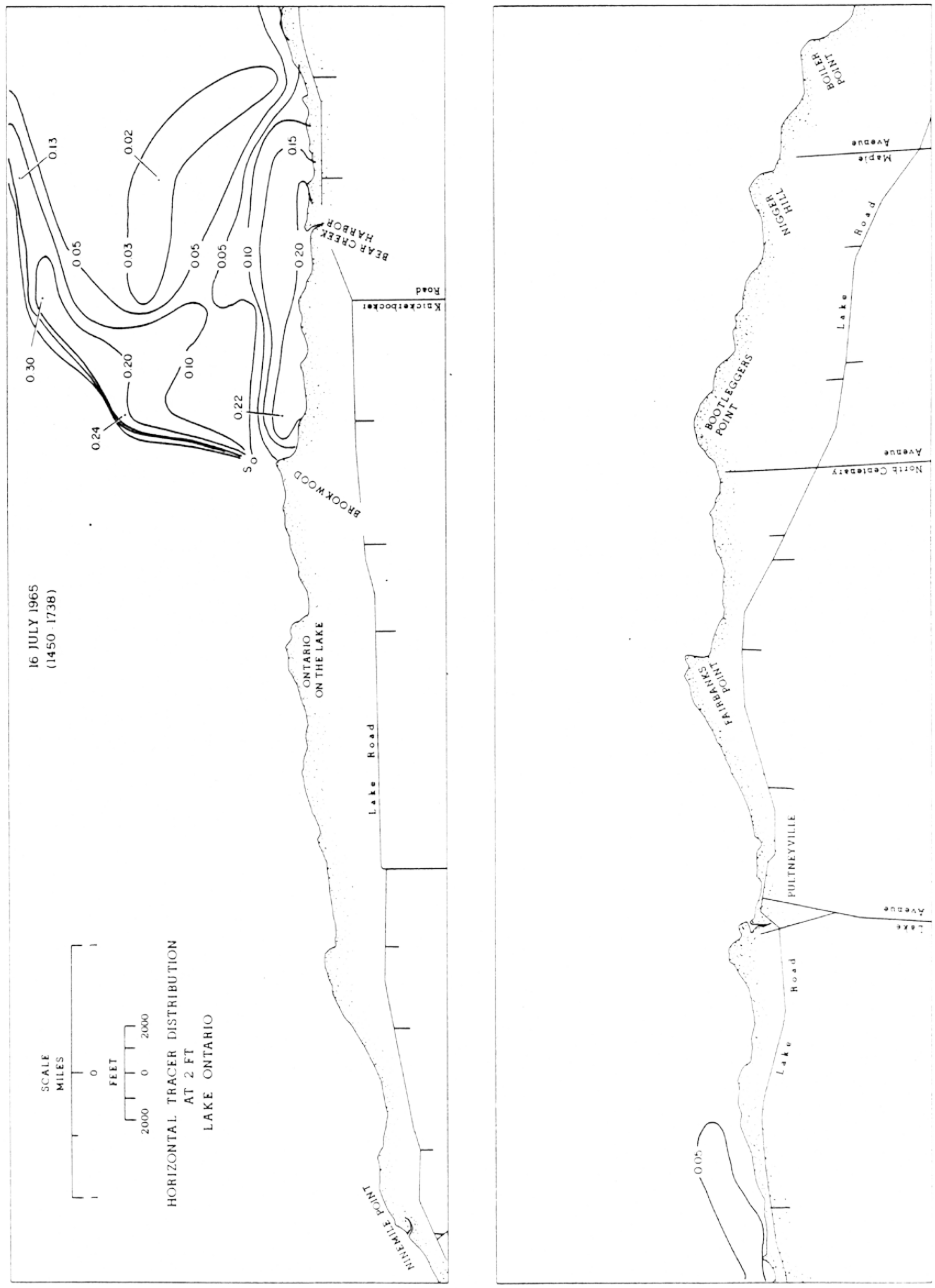
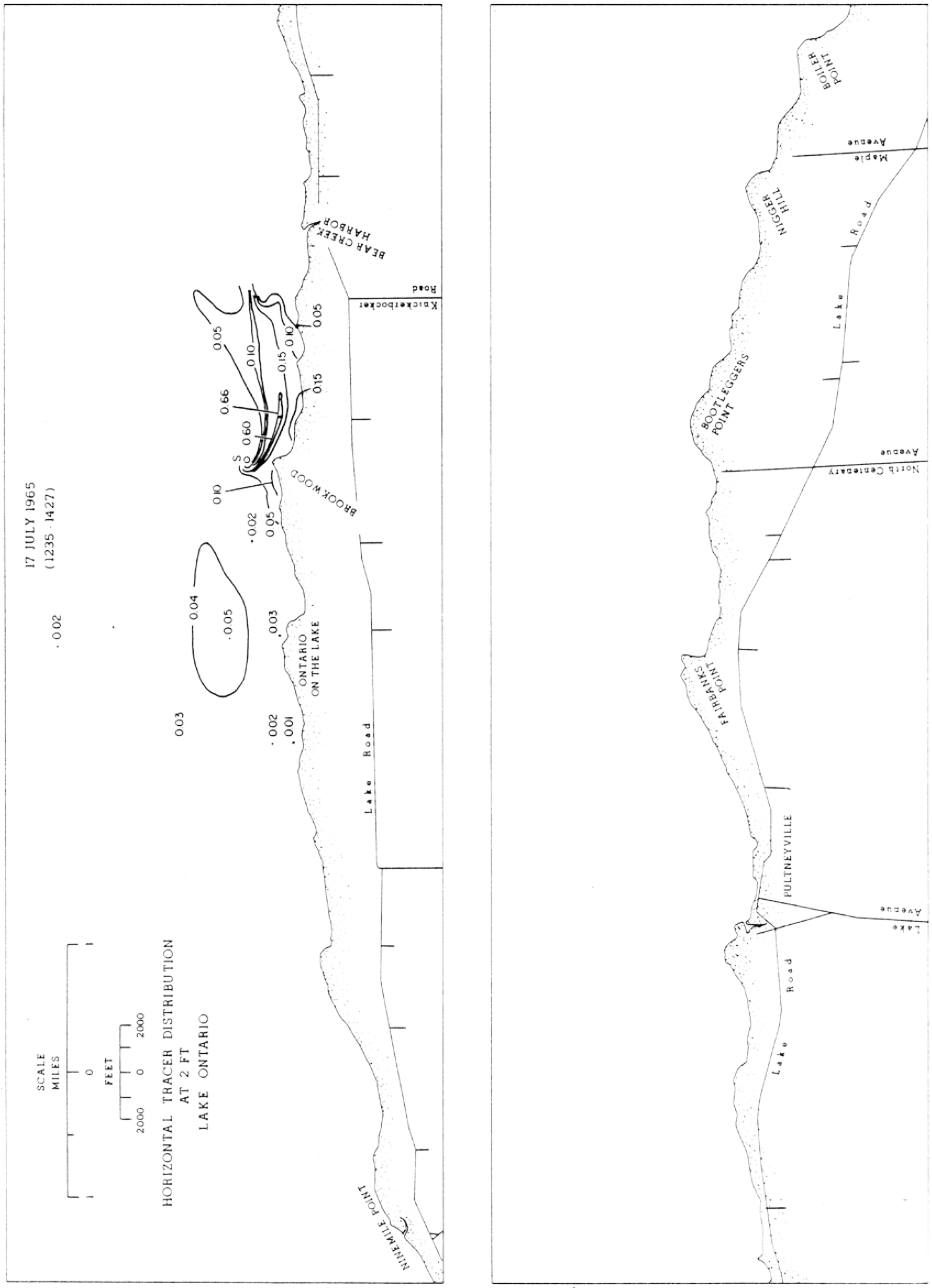
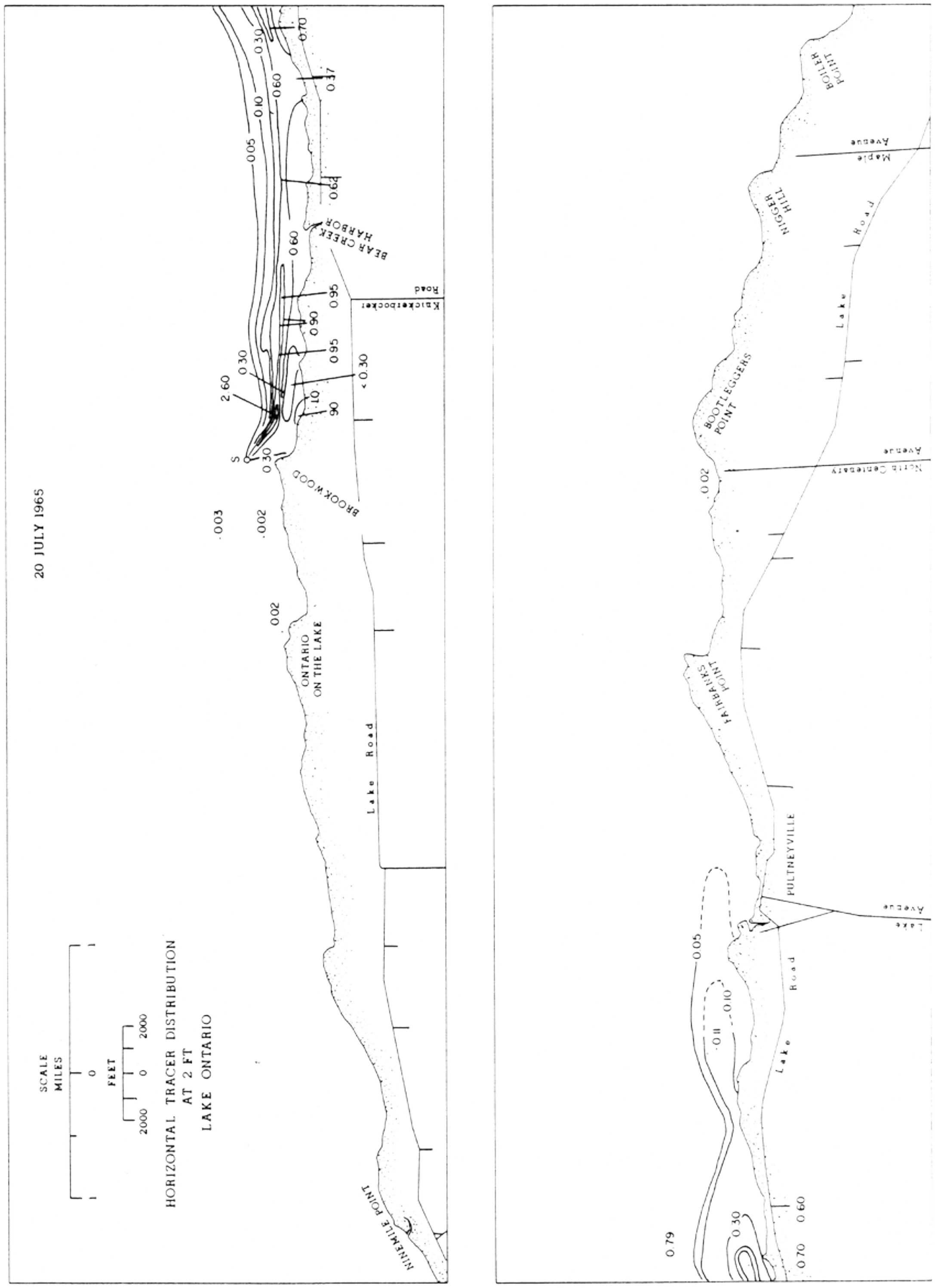

16 JULY 1965
(1450-1738)
SCALE
MILES
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
0.13
0.02
0.15
0.05
0.03
0.05
0.05
0.10
0.20
0.30
0.20
0.10
0.22
0.24
S
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Cemetary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road
0.05

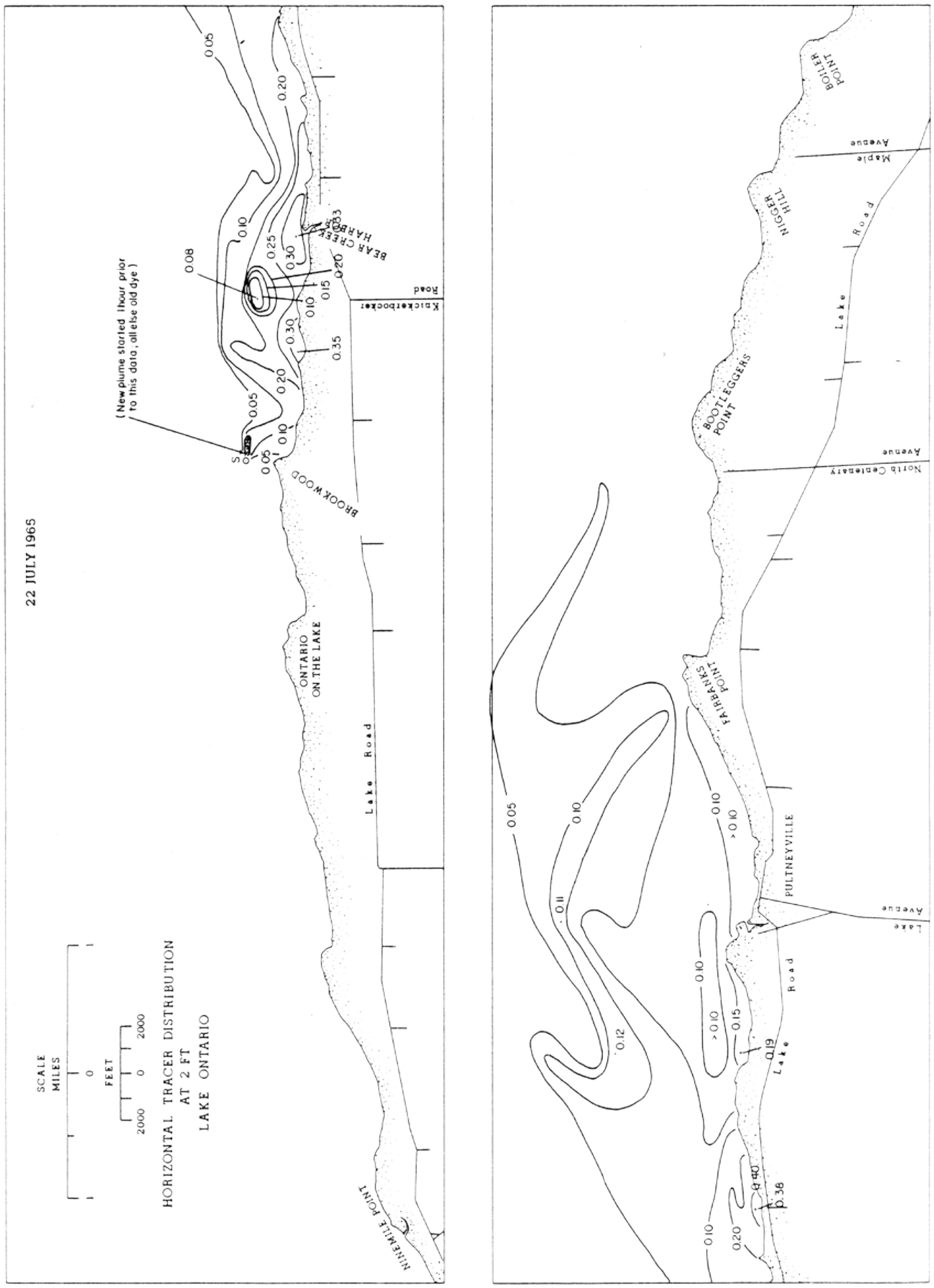
none


17 JULY 1965
(1235-1427)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000 0 2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

none


20 JULY 1965
SCALE
MILES
0
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Cemetary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

none


22 JULY 1965
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
(New plume started 1 hour prior to this data, all else old dye)
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

2B-70

none

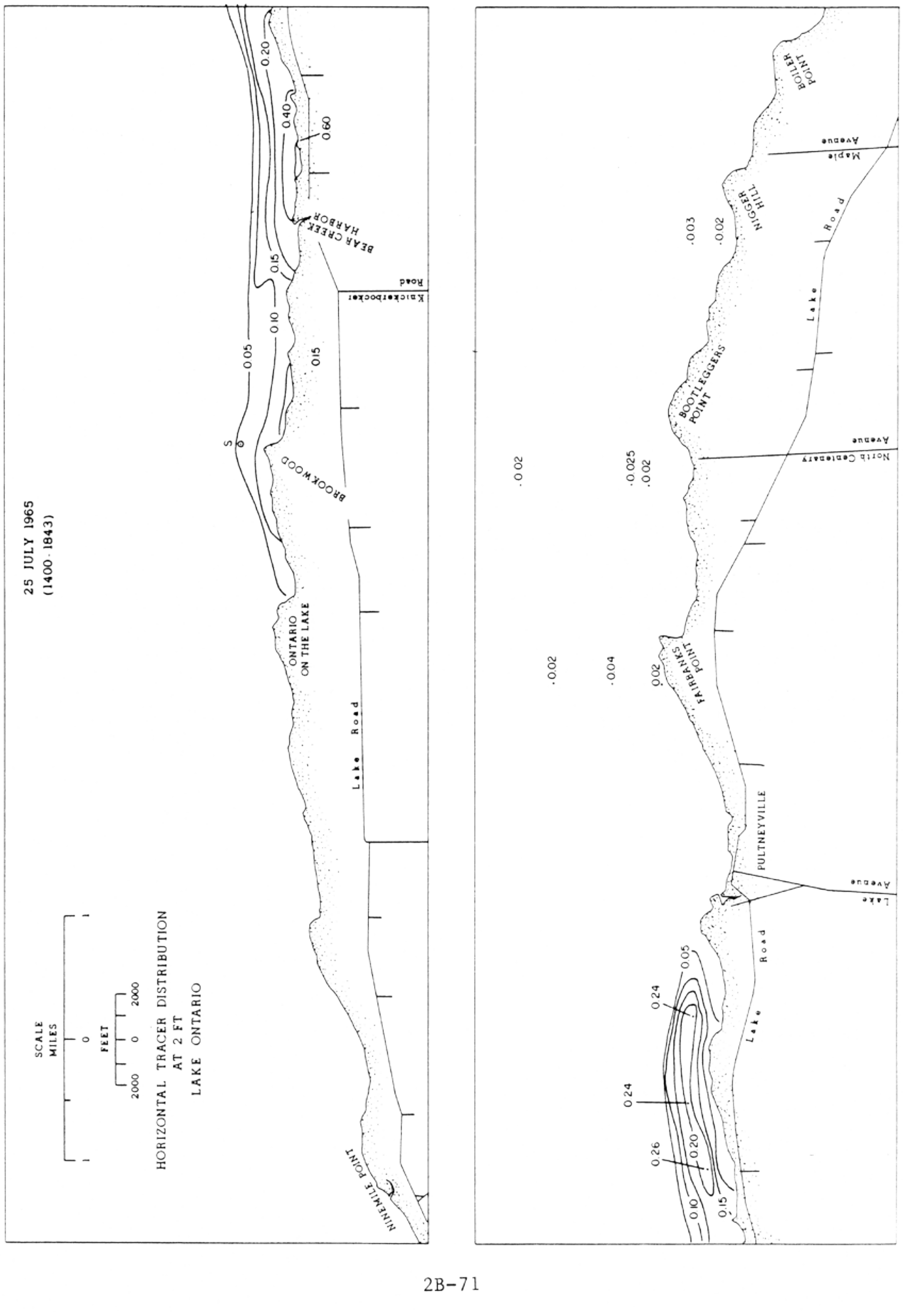
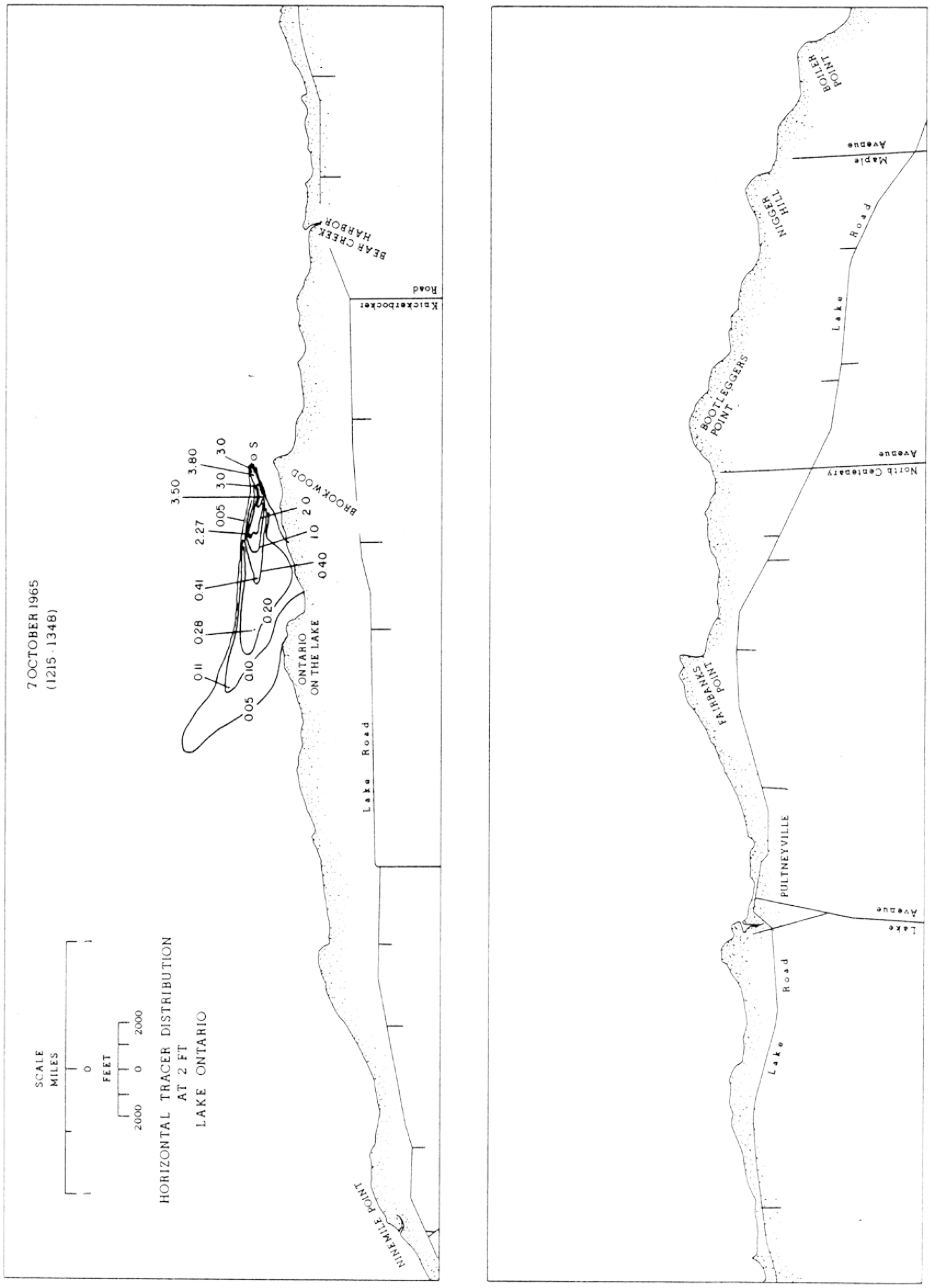
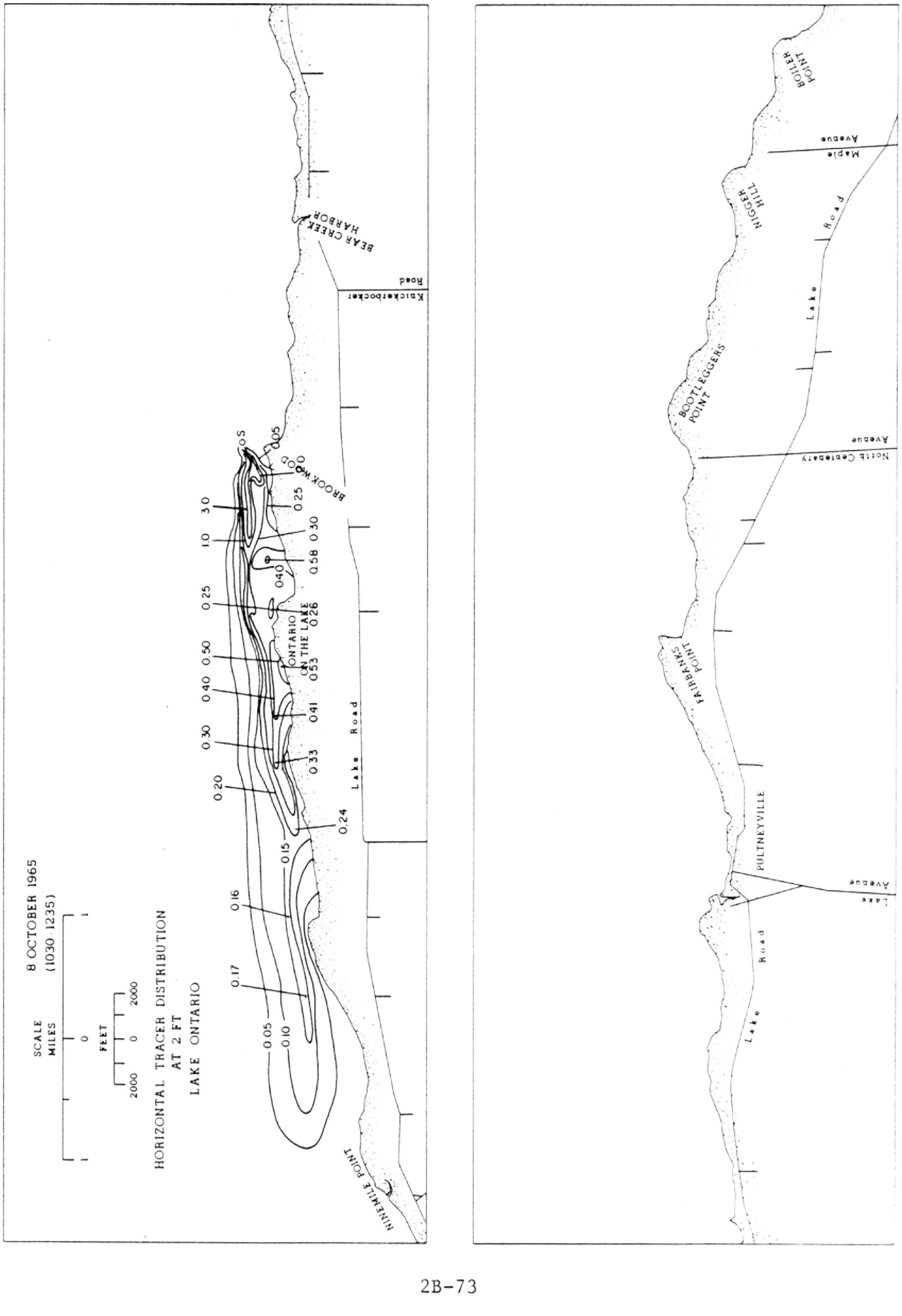

25 JULY 1965
(1400-1843)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

Figure 3

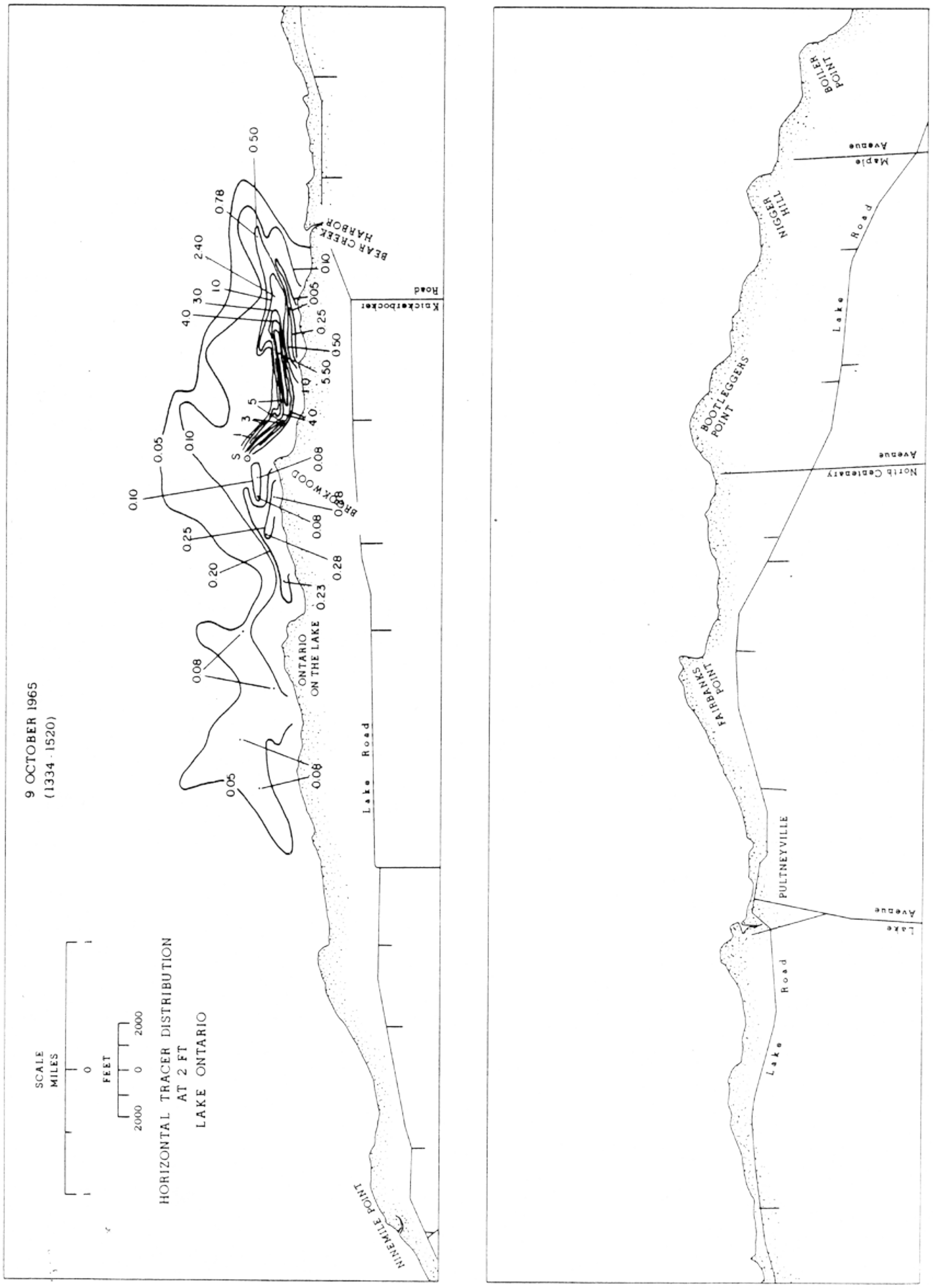

7 OCTOBER 1965
(1215-1348)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
0
FEET
2000
0
2000
3.50
3.80
3.0
30
0.5
005
2.27
2.0
1.0
0.41
0.40
0.28
0.20
0.11
0.10
0.05
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none

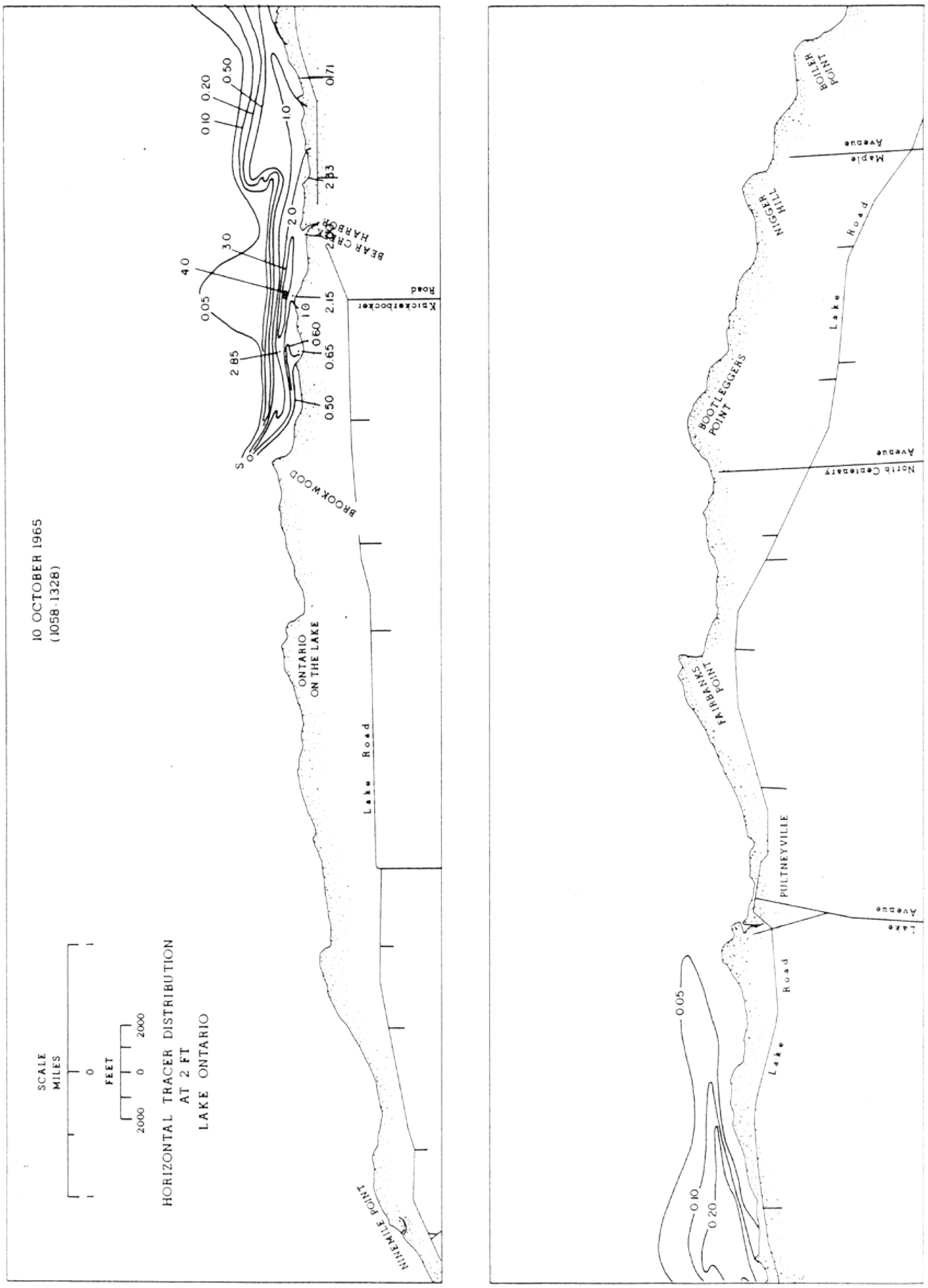

8 OCTOBER 1965
(1030-1235)
SCALE
MILES
FEET
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

2B-73

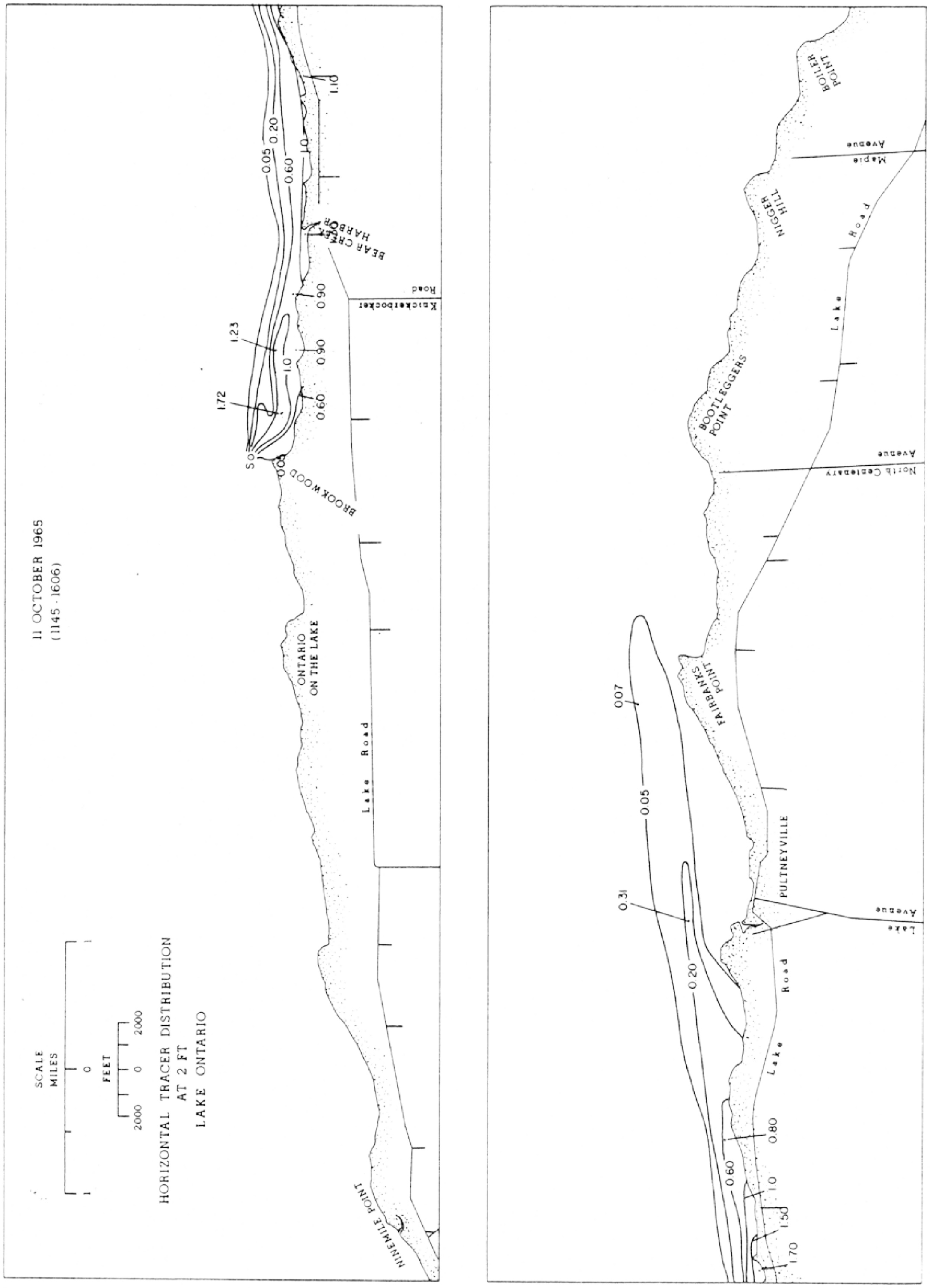
none


9 OCTOBER 1965
(1334-1520)
SCALE
MILES
FEET
2000 0 2000
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

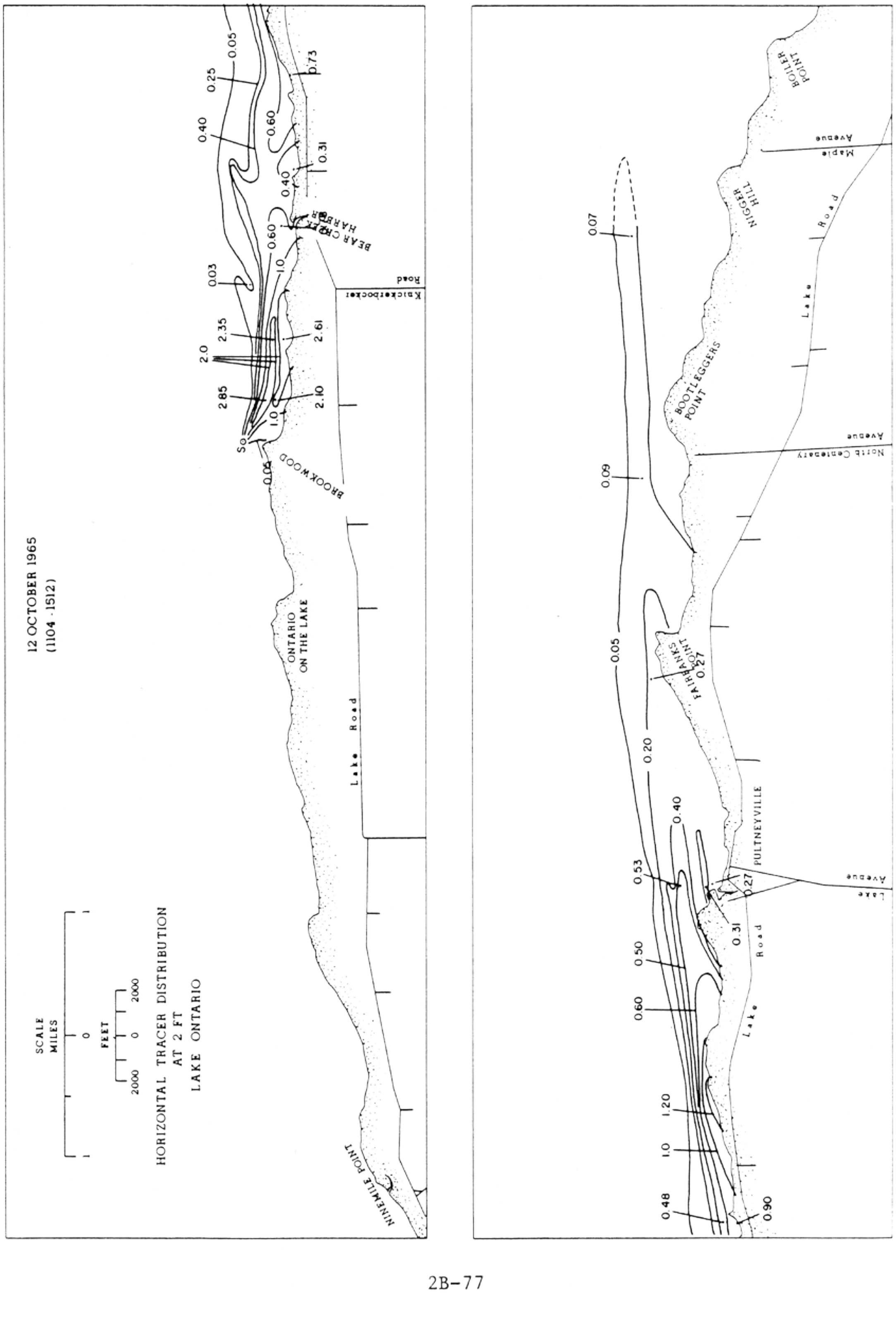
none


10 OCTOBER 1965
(1058-1328)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
NINEMILE POINT
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Lake Road
Knickerbocker Road
PULTNEYVILLE
FAIRBANKS POINT
BOOTLEGGERS POINT
NIGGER HILL
BOILER POINT
Maple Avenue
North Centenary Avenue
Lake Avenue
0.05
0.10
0.20
0.50
1.0
2.0
3.0
4.0
2.85
2.83
2.15
0.71
0.60
0.65
0.50

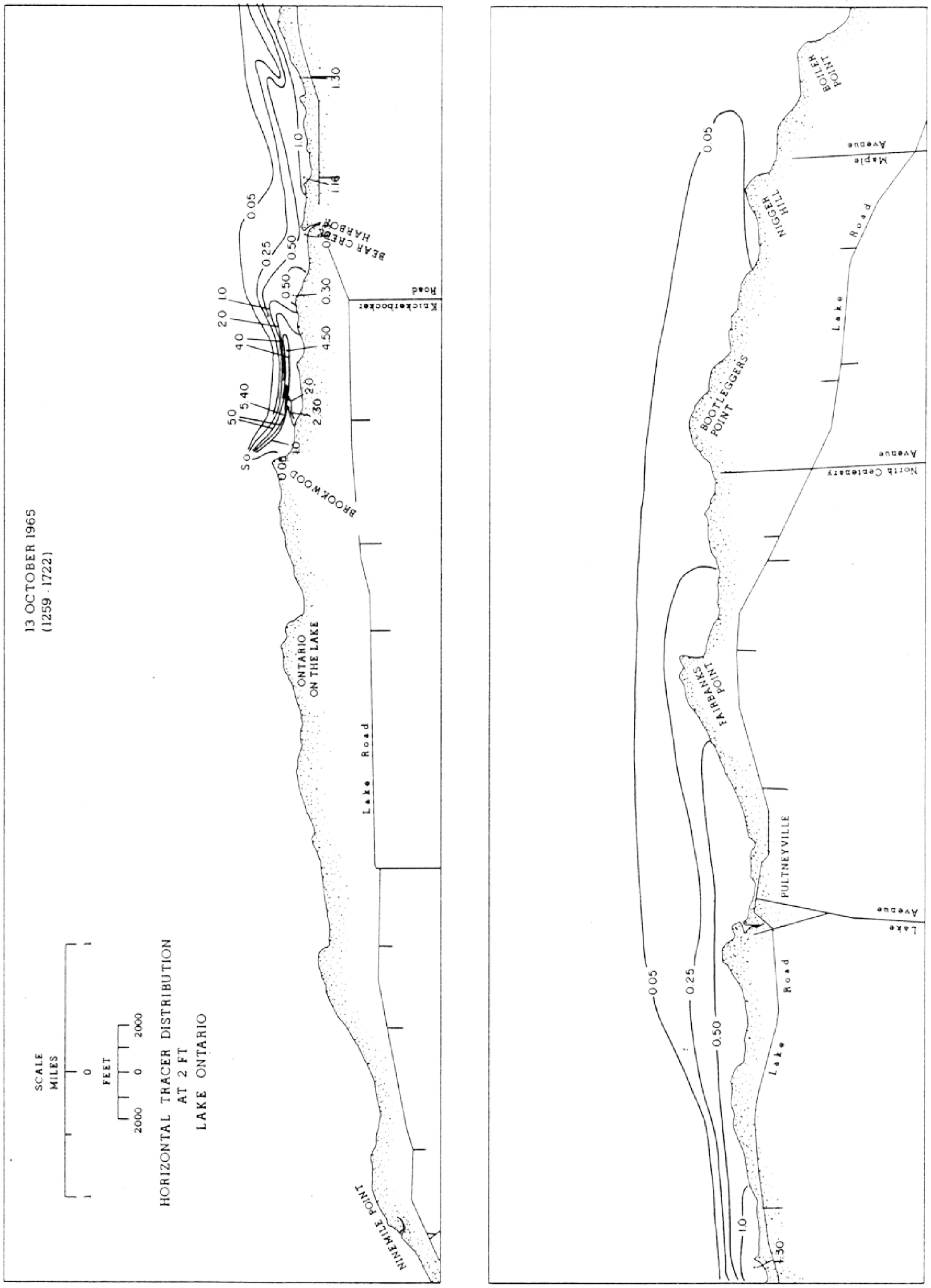
none


11 OCTOBER 1965
(1145-1606)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
NINEMILE POINT
ONTARIO ON THE LAKE
BROOKWOOD
BEAR CREEK HARBOR
Knickerbocker Road
Lake Road
1.72
1.23
0.05
0.20
0.60
0.90
1.0
1.10
BOILER POINT
Maple Avenue
NIGGER HILL
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
0.07
0.31
0.80
1.50
1.70

none


12 OCTOBER 1965
(1104 - 1512)
HORIZONTAL TRACER DISTRIBUTION
AT 2 FT
LAKE ONTARIO
SCALE
MILES
FEET
2000
0
2000
BEAR CREEK HARBOR
Knickerbocker Road
BROOKWOOD
ONTARIO ON THE LAKE
Lake Road
NINEMILE POINT
BOILER POINT
Maple Avenue
NIGGER HILL
Lake Road
BOOTLEGGERS POINT
North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue
Lake Road

none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BROOKWOOD
Knickerbocker Road
BEAR CREEK HARBOR
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North Centenary Avenue
FAIRBANKS POINT
PULTNEYVILLE
Lake Avenue

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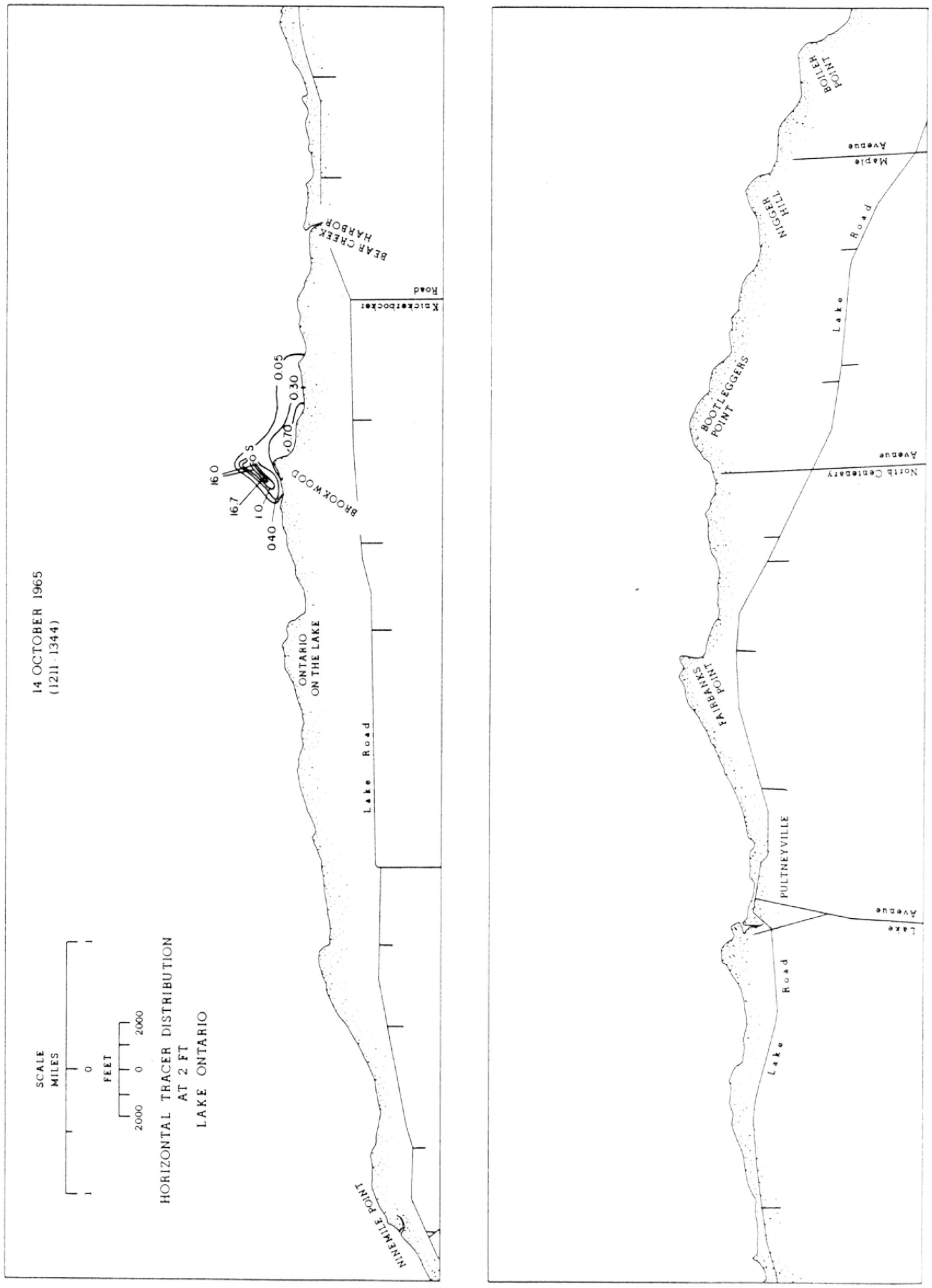
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0.30
0.70
0.40
16.0
16.7
1.0
BOILER POINT
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Lake Road

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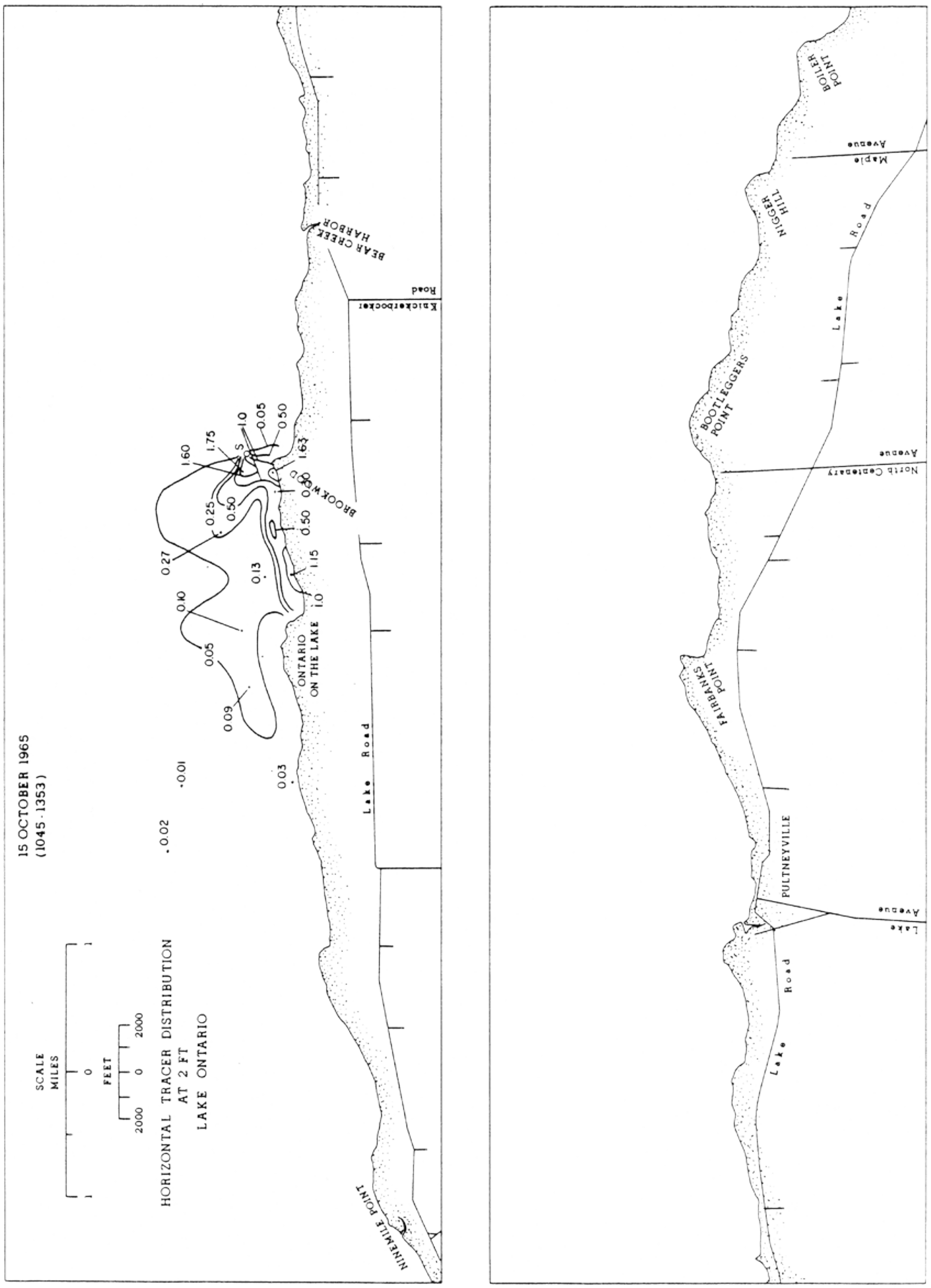
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BOILER POINT
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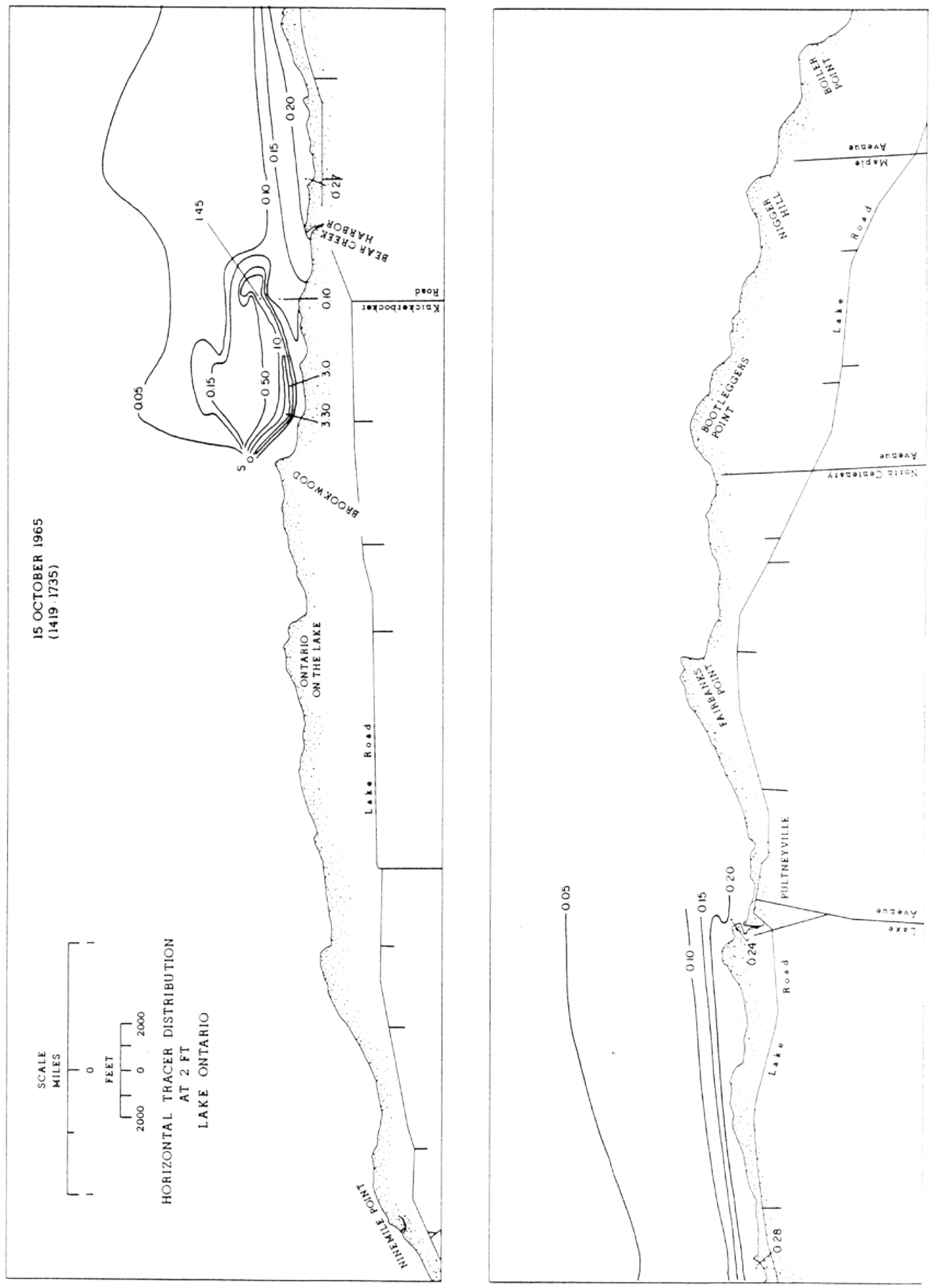
Lake Avenue
Lake Road

none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Lake Avenue

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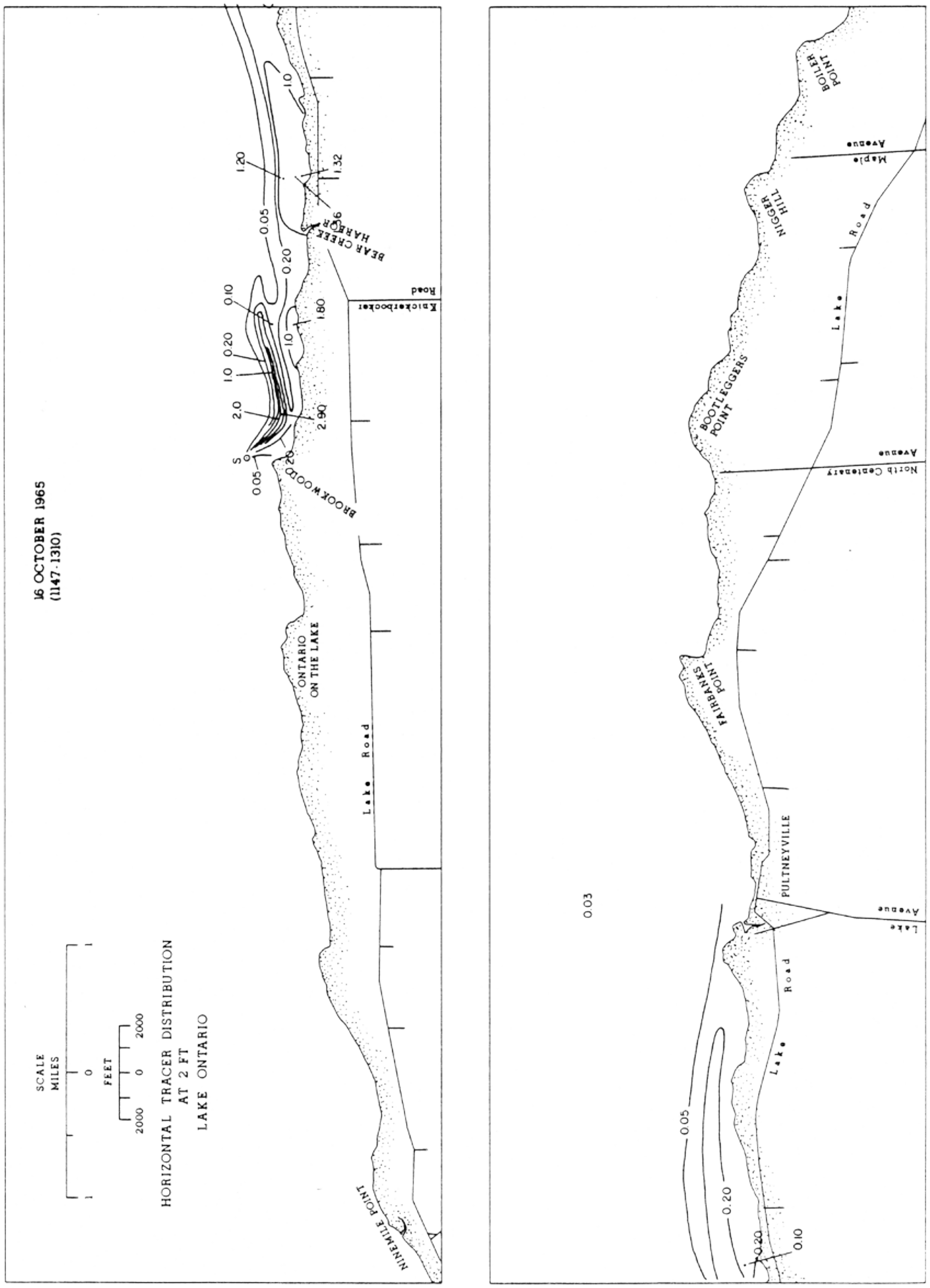
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ONTARIO ON THE LAKE
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Knickerbocker Road
Lake Road
BOILER POINT
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FAIRBANKS POINT
PULTNEYVILLE
Maple Avenue
North Centenary Avenue
Lake Avenue
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Figure 3

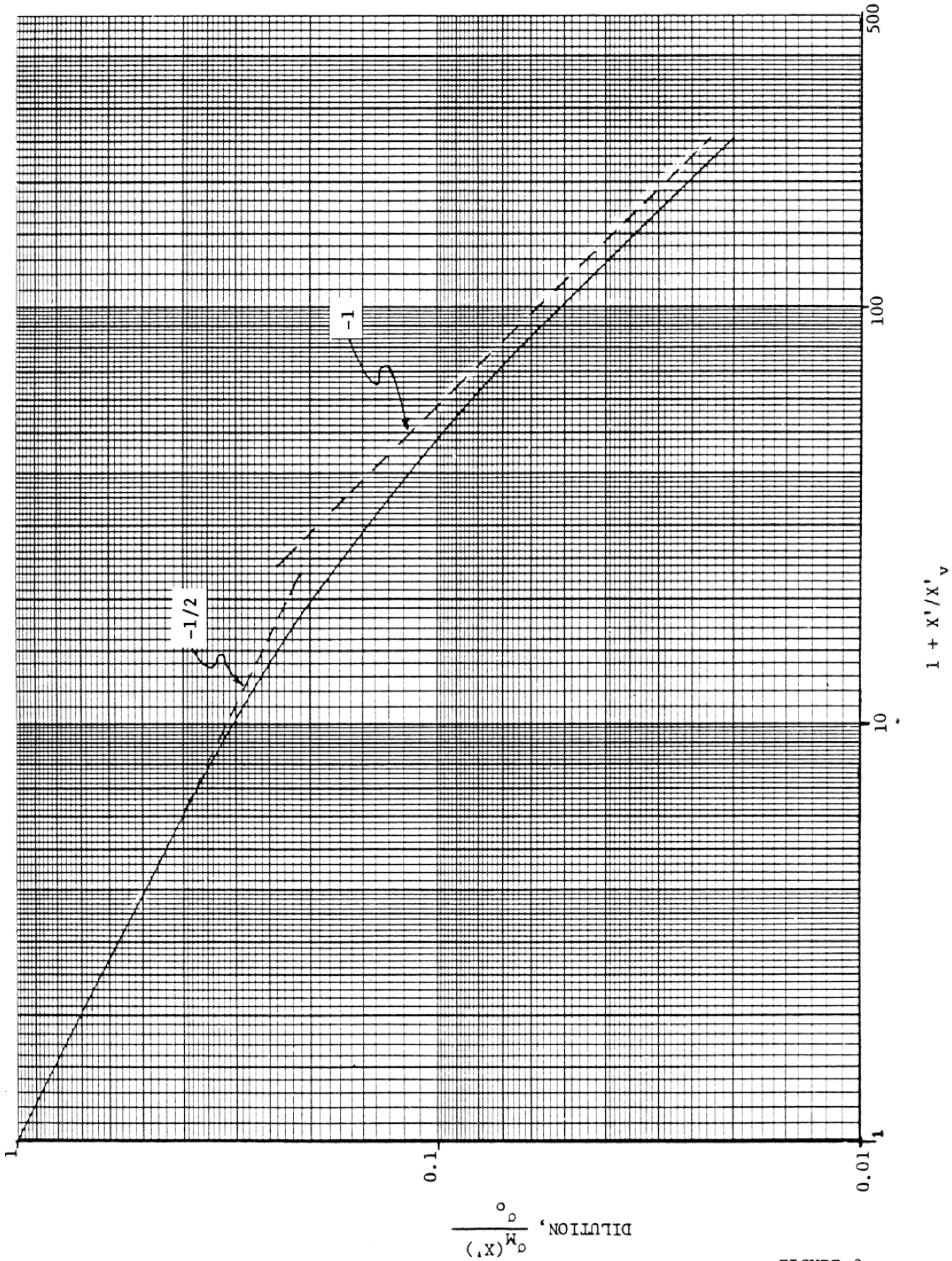


FIGURE 3

Figure 1 Plate 1 - Plot Plan

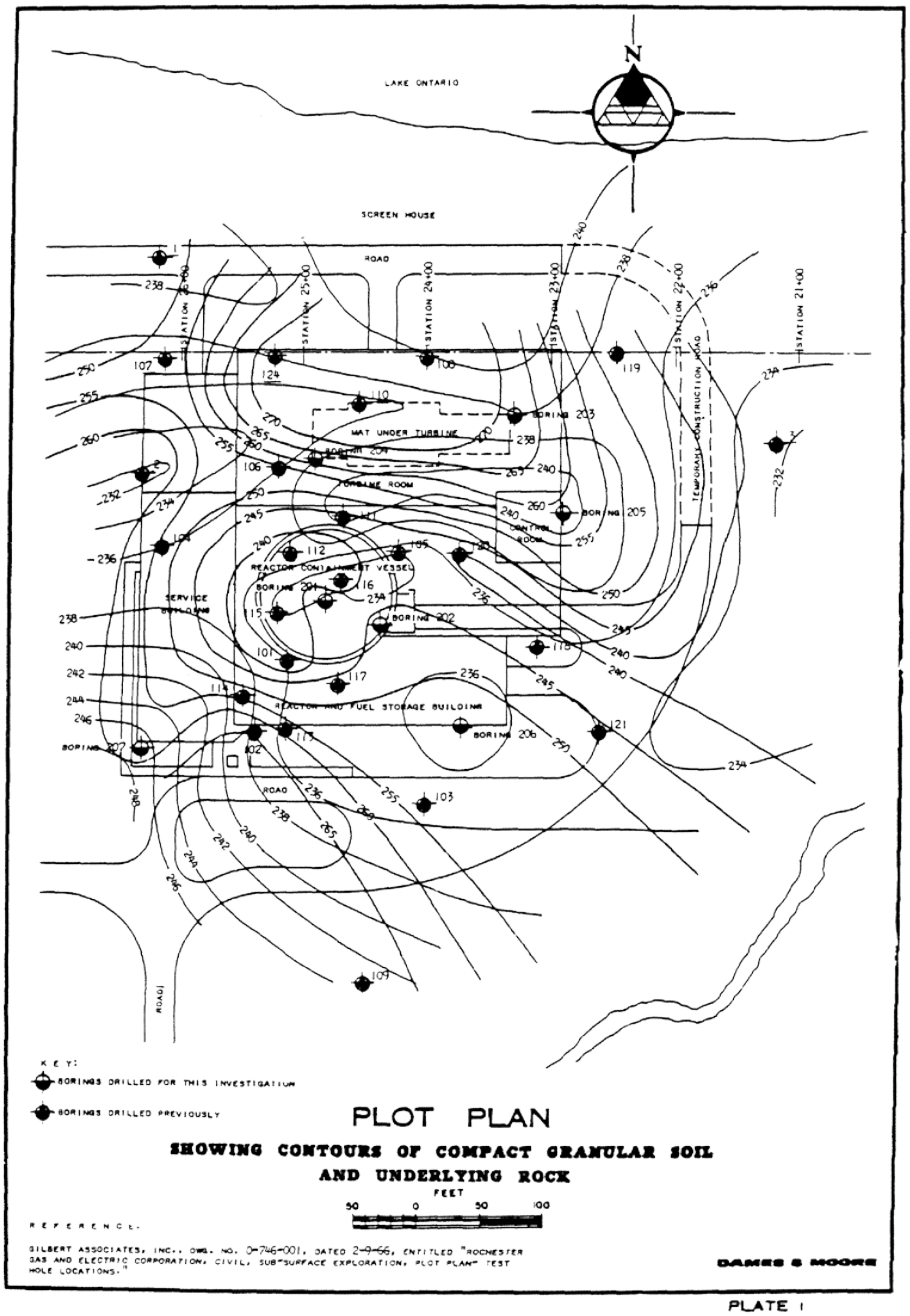
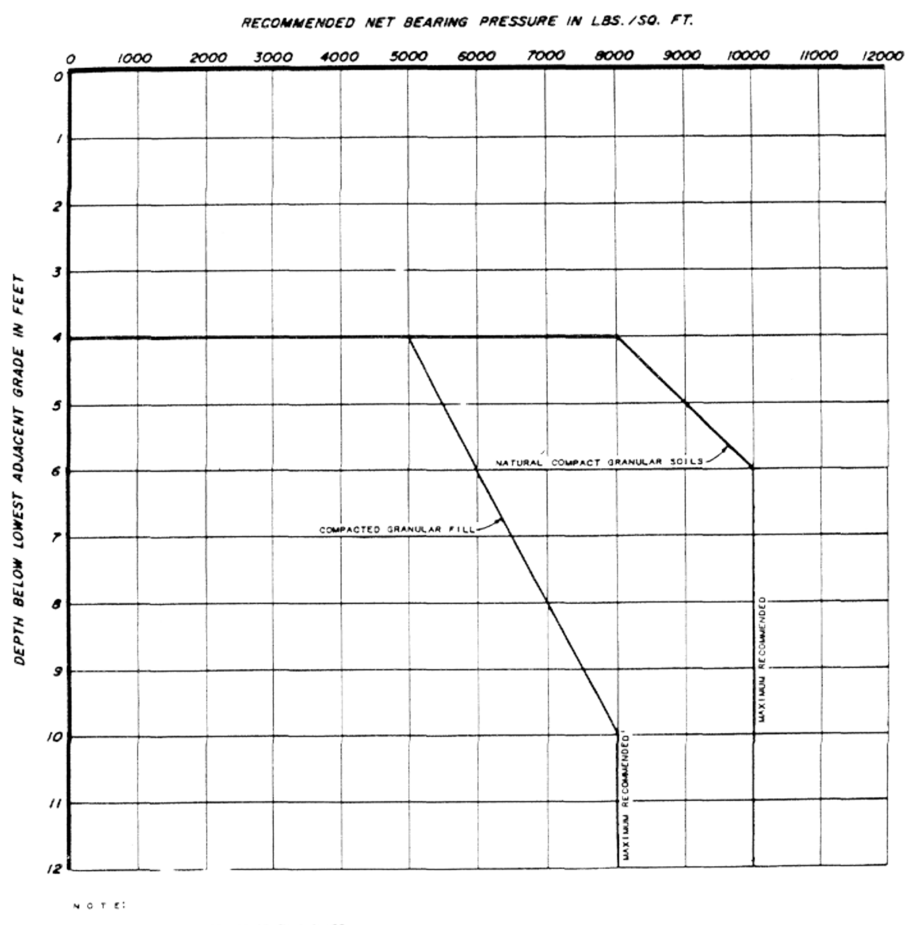


Figure 2 Plate 2 - Foundation Design Data



FOUNDATION DESIGN DATA

NATURAL AND FILL SOILS

DAMES & MOORE

PLATE 2

Figure 3 Soil Sampler Type U

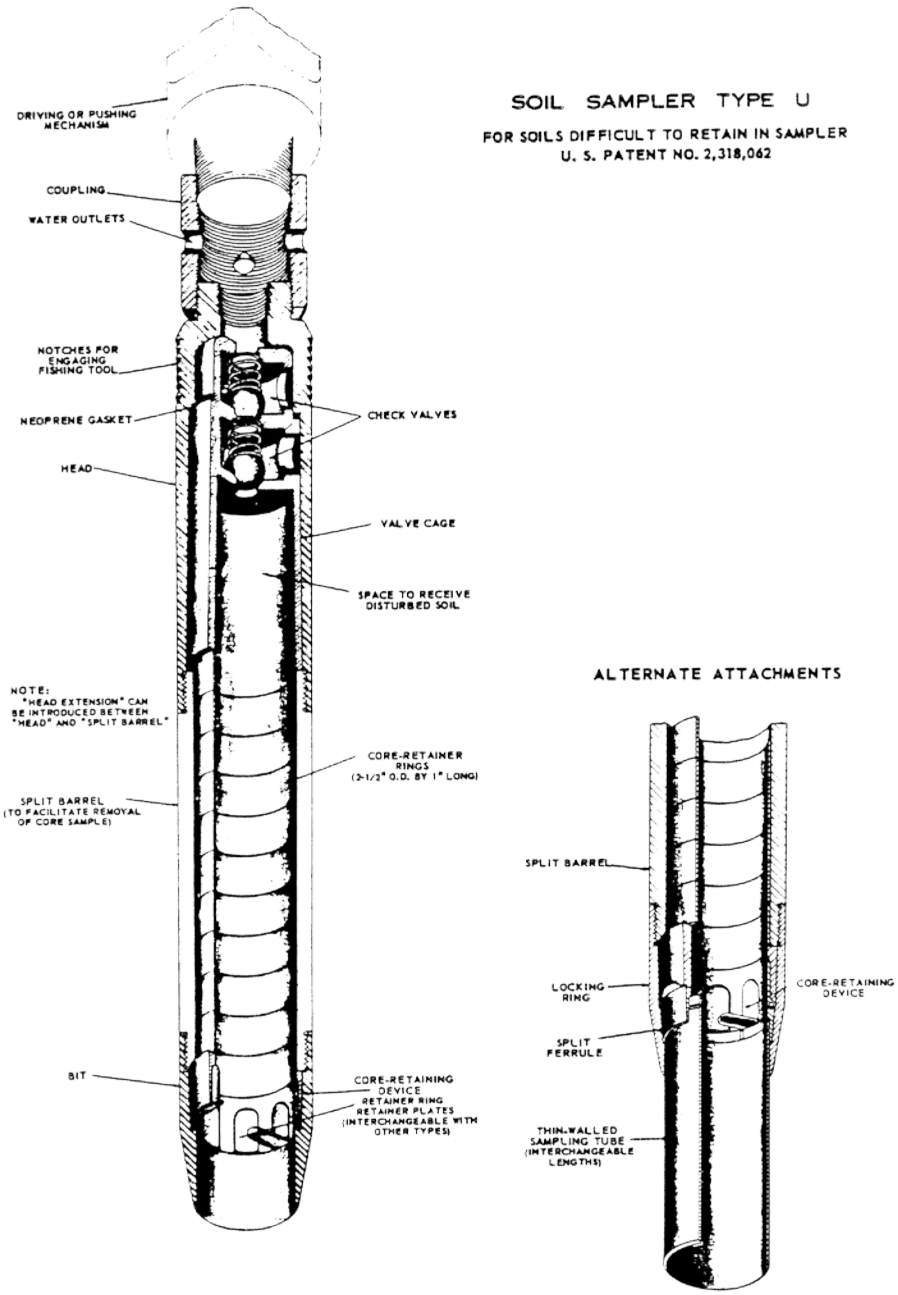
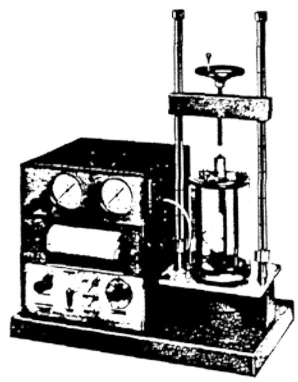


Figure 4 Methods of Performing Unconfined Compression and Triaxial Compression Tests

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.



TRIAXIAL COMPRESSION TEST UNIT

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

Figure 5 Plate A-1A - Log of Borings (Borings 201 through 202)

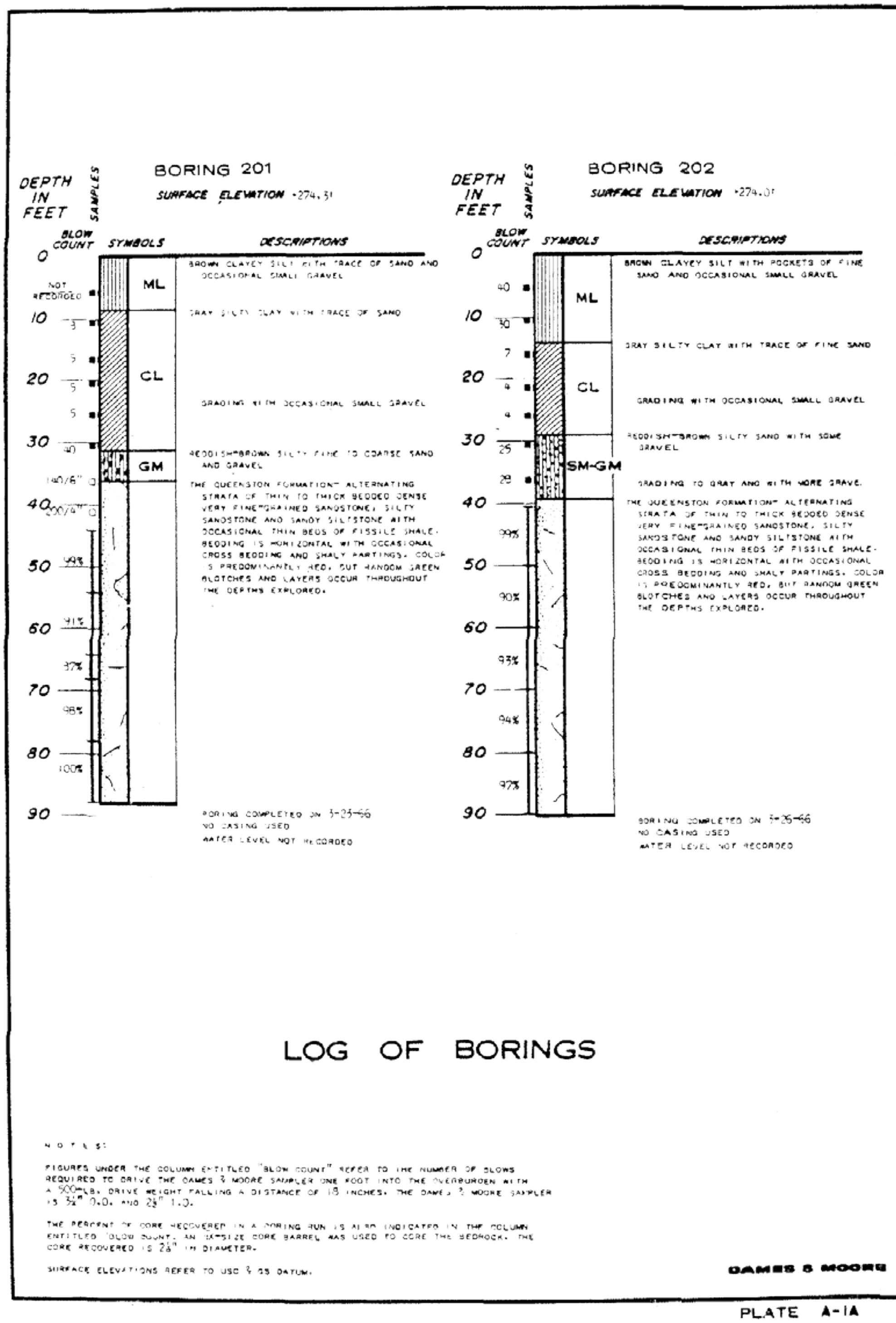


Figure 6 Plate A-1B - Log of Borings (Borings 203 through 207)

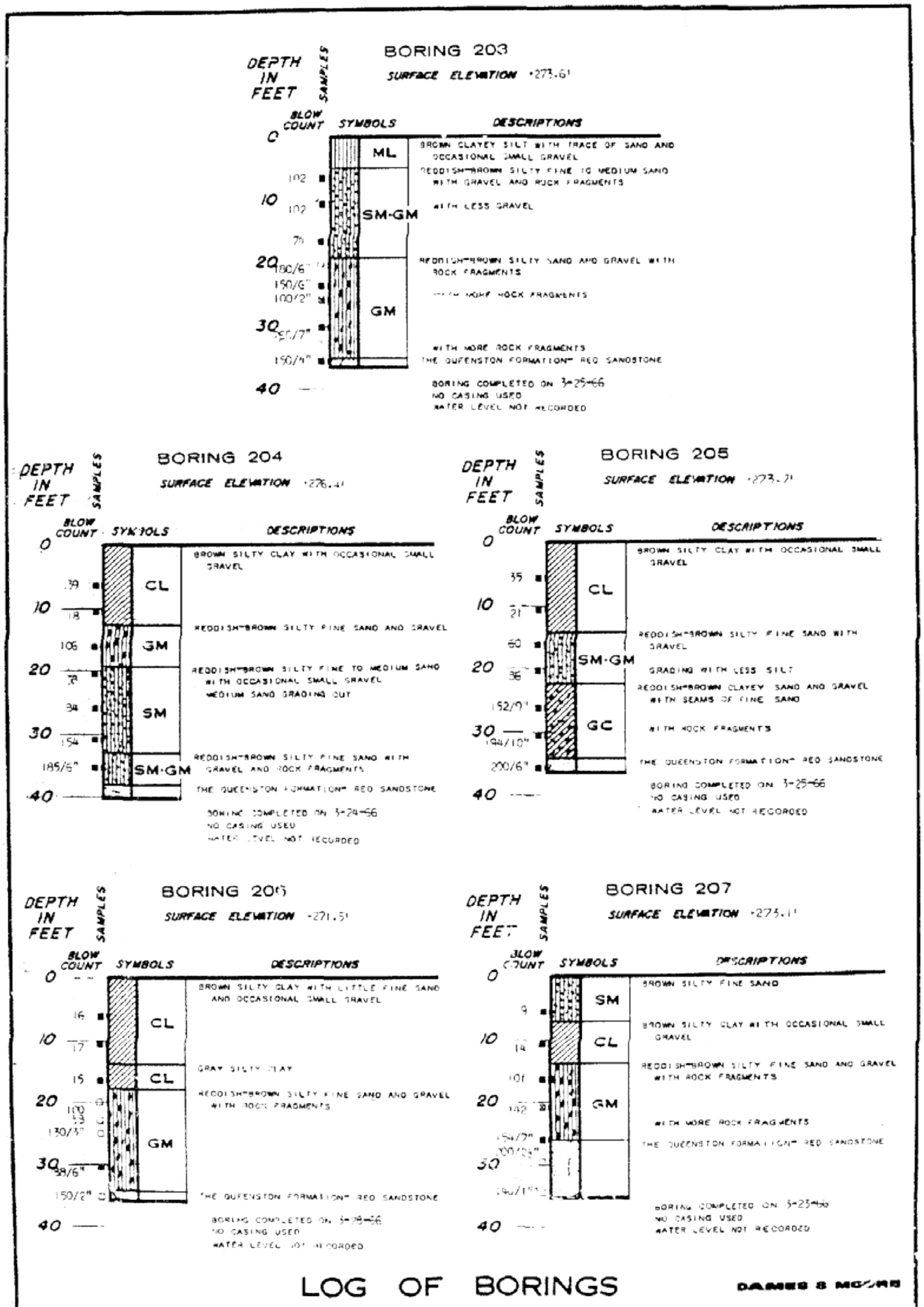
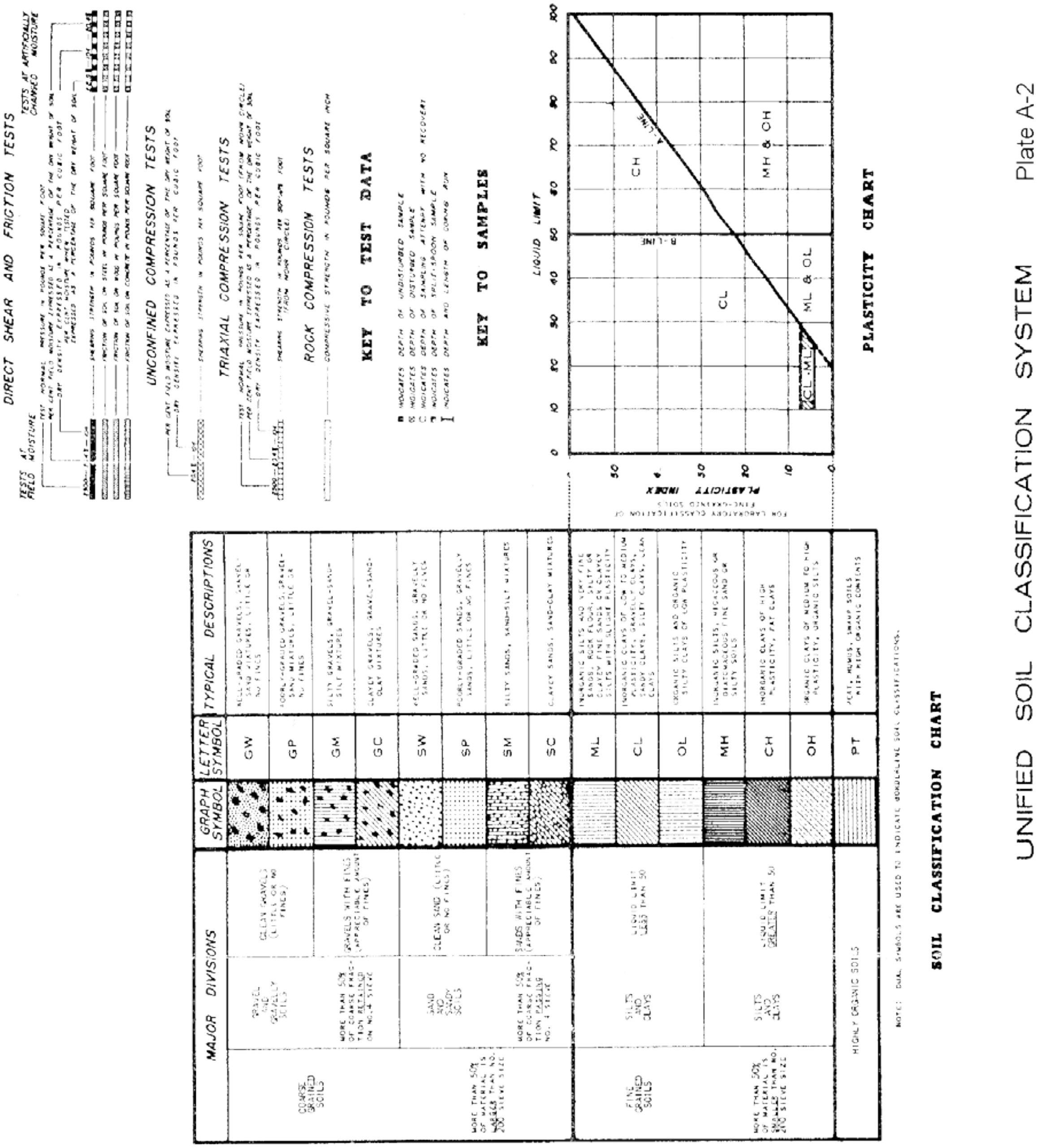


PLATE A-1B

Figure 7 Plate A-2 - Unified Soil Classification System and Key to Test Data



3 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

3.1 CONFORMANCE WITH NRC GENERAL DESIGN CRITERIA

The discussion of general design criteria is divided into two parts. 04/2013Section 3.1.1 discusses the general design criteria used during the licensing of Ginna Station. Section 3.1.2 discusses the adequacy of the Ginna design relative to the 1972 version of the General Design Criteria in 10 CFR 50, Appendix A.

3.1.1 ATOMIC INDUSTRIAL FORUM DESIGN CRITERIA

The following general design criteria comprise the proposed Atomic Industrial Forum (AIF) versions of the criteria issued for comment by the AEC on July 10, 1967. These criteria define or describe safety objectives and approaches incorporated in the design of this plant. Each criterion is followed by a brief description of related plant features which are provided to meet the design objectives reflected in the criterion. The description is developed more fully in succeeding sections of the updated FSAR. The criteria are identified as AIF-GDC plus their identification numbers to distinguish them from the later 10 CFR 50, Appendix A, criteria which are identified as GDC plus their identification numbers.

3.1.1.1 Overall Plant Requirements

3.1.1.1.1 Quality Standards

CRITERION: Those systems and components of reactor facilities which are essential to the prevention, or the mitigation of the consequences, of nuclear accidents which could cause undue risk to the health and safety of the public shall be identified and then designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed. Where generally recognized codes and standards pertaining to design, materials, fabrication, and inspection are used, they shall be identified. Where adherence to such codes or standards does not suffice to assure a quality product in keeping with the safety function, they shall be supplemented or modified as necessary. Quality assurance programs, test procedures, and inspection acceptance criteria to be used shall be identified. An indication of the applicability of codes, standards, quality assurance programs, test procedures, and inspection acceptance criteria used is required. Where such items are not covered by applicable codes and standards, a showing of adequacy is required (AIF-GDC 1).

All structures, systems, and components of the facility were classified according to their safety importance. Those items vital to safe shutdown and isolation of the reactor or whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity were designated Class I. Those items important to reactor operation but not essential to safe shutdown and isolation of the reactor or control of the release of substantial amounts of radioactivity were designated Class II. Those items not related to reactor operation or safety were designated Class III.

Class I systems and components were designated as essential to the protection of the health and safety of the public. Consequently, they were designed, fabricated, inspected, and erected and the materials selected to the applicable provisions of recognized codes, good nuclear

practice and to quality standards that reflect their importance. Discussions of applicable codes and standards, quality assurance programs, test provisions, etc., are given in the sections of the UFSAR describing each system. It should be noted that Ginna Station no longer uses the Class I, II, and III classification scheme. The classification and codes applicable to Ginna Station structures, systems, and components are discussed in Section 3.2 and in the applicable UFSAR sections.

Reference chapters are as follows:

Chapter Title	Chapter
Reactor	Chapter 4
Reactor Coolant System and Connected Systems	Chapter 5
Engineered Safety Features	Chapter 6
Instrumentation and Controls	Chapter 7
Electric Power	Chapter 8
Auxiliary Systems	Chapter 9
Steam and Power Conversion System	Chapter 10

3.1.1.1.2 Performance Standards

CRITERION: Those systems and components of reactor facilities which are essential to the prevention or to the mitigation of the consequences of nuclear accidents which could cause undue risk to the health and safety of the public shall be designed, fabricated, and erected to performance standards that enable such systems and components to withstand, without undue risk to the health and safety of the public the forces that might reasonably be imposed by the occurrence of an extraordinary natural phenomenon such as earthquake, tornado, flooding condition, high wind or heavy ice. The design bases so established shall reflect: (a) appropriate consideration of the most severe of these natural phenomena that have been officially recorded for the site and the surrounding area and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design (AIF-GDC 2).

All systems and components designated Class I were designed so that no loss of function in the event of the maximum potential ground acceleration acting in the horizontal and vertical directions simultaneously would occur. Similarly, measures were taken in the plant design to protect against high winds, sudden barometric pressure changes, seiches, and other natural phenomena.

Reference chapters are as follows:

Chapter Title	Chapter
Site Characteristics	Chapter 2
Design of Structures, Components, and Systems	Chapter 3
Reactor	Chapter 4
Reactor Coolant System and Connected Systems	Chapter 5
Engineered Safety Features	Chapter 6
Instrumentation and Controls	Chapter 7
Electrical Power	Chapter 8
Auxiliary Systems	Chapter 9
Steam and Power Conversion System	Chapter 10

3.1.1.1.3 Fire Protection

CRITERION: A reactor facility shall be designed such that the probability of events such as fires and explosions and the potential consequences of such events does not result in undue risk to the health and safety of the public. Noncombustible and fire resistant materials shall be used throughout the facility wherever necessary to preclude such risk, particularly in areas containing critical portions of the facility such as containment, control room, and components of engineered safety features (AIF-GDC 3).

Fire prevention in all areas of the nuclear-electric plant is provided by structure and component design which optimizes the containment of combustible materials and maintains exposed combustible materials below their ignition temperature in the design atmosphere. Fire control requires the capability to isolate or remove fuel from an igniting source, or to reduce the combustible's temperature below the ignition point, or to exclude the oxidant, and preferably, to provide a combination of the three basic control means. The latter two means are fulfilled by providing fixed or portable fire fighting equipment of capacities proportional to the energy that might credibly be released by fire.

This station is designed on the basis of limiting the use of combustible materials in construction by using fire-resistant materials to the greatest extent possible.

The fire protection system has the design capability to extinguish any probable combination of simultaneous fires which might occur at the station. The system is designed in accordance with the standards of the National Fire Protection Association and is based generally on the recommendations of the Nuclear Energy Property Insurance Association.

Fire protection systems for Ginna Station are discussed in Section 9.5.1.

Refer to Section 9.5.1.1.2 and Section 9.5.1.1.3 for updated design information.

3.1.1.1.4 Sharing of Systems

CRITERION: Reactor facilities may share systems or components if it can be shown that such sharing will not result in undue risk to the health and safety of the public (AIF-GDC 4).

Analyses confirm that the sharing of components among systems does not result in interference with the basic function and operability of these systems and hence there is no undue risk to the health and safety of the public.

3.1.1.1.5 Records Requirements

CRITERION: The reactor licensee shall be responsible for assuring the maintenance throughout the life of the reactor of records of the design, fabrication, and construction of major components of the plant essential to avoid undue risk to the health and safety of the public (AIF-GDC 5).

A complete set of as-built facility plant and system diagrams including arrangement plans and structural plans are maintained throughout the life of the reactor.

A set of completed test procedures for all plant testing is maintained as outlined in Chapter 14.

A set of all the quality assurance data generated during fabrication and erection of the essential components of the plant, as defined by the quality assurance program, is retained.

3.1.1.2 Protection by Multiple Fission Product Barriers

3.1.1.2.1 Reactor Core Design

CRITERION: The reactor core with its related controls and protection systems, shall be designed to function throughout its design lifetime without exceeding acceptable fuel damage limits which have been stipulated and justified. The core and related auxiliary system design shall provide this integrity under all expected conditions of MODES 1 and 2 with appropriate margins for uncertainties and for specified transient situations which can be anticipated (AIF-GDC 6).

The reactor core, with its related control and protection system, is designed to function throughout its design lifetime without exceeding acceptable fuel damage limits. The core design, together with reliable process and decay heat removal systems, provides for this capability under all expected conditions of MODES 1 and 2 with appropriate margins for uncertainties and anticipated transient situations, including the effects of the loss of reactor coolant flow (Section 15.3), loss of electrical load (Section 15.2.2), loss of normal feedwater (Section 15.2.6), and loss of all offsite power (Section 15.2.5).

The reactor control and protection instrumentation is designed to actuate a reactor trip for any anticipated combination of plant conditions, when necessary to ensure a minimum departure from nucleate boiling ratio (DNBR) equal to or greater than the safety limit and fuel center temperature below the melting point of uranium dioxide.

Referenced chapters are:

Chapter Title	Chapter
Reactor	Chapter 4
Instrumentation and Controls	Chapter 7
Accident Analyses	Chapter 15

3.1.1.2.2 Suppression of Power Oscillations

CRITERION: The design of the reactor core with its related controls and protection systems shall ensure that power oscillations, the magnitude of which could cause damage in excess of acceptable fuel damage limits, are not possible or can be readily suppressed (AIF-GDC 7).

The design of the reactor core and related protection systems ensures that power oscillations which could cause fuel damage in excess of acceptable limits are not possible or can be readily suppressed.

The potential for possible spatial oscillations of power distribution for this core has been reviewed. In summary it is concluded that the only potential spatial instability of a magnitude which could cause damage in excess of acceptable fuel damage limits is the xenon-induced axial instability which may be a nearly free running oscillation with little or no inherent damping. Part-length control rods were originally provided to suppress these oscillations if they occurred. They have since been removed. Operating control strategies have been devised that do not require insertion of the part-length rods and eliminate the potential for axial xenon instabilities. Out-of-core instrumentation is provided to obtain necessary information concerning axial distributions. This instrumentation is adequate to enable the operator to monitor and control xenon induced oscillations. In-core instrumentation is used to periodically calibrate and verify the information provided by the out-of-core instrumentation.

The temperature coefficient in the power operating range was maintained zero or negative by inclusion of burnable poison shims in the first core loading. The burnable poison shims have since been removed.

3.1.1.2.3 Overall Power Coefficient

CRITERION: The reactor shall be designed so that the overall power coefficient in the power operating range shall not be positive (AIF-GDC 8).

The overall power coefficient in the power operating range is maintained nonpositive. The nuclear design of the reactor is discussed in Section 4.2.4.2.7.

3.1.1.2.4 Reactor Coolant Pressure Boundary

CRITERION: The reactor coolant pressure boundary shall be designed, fabricated and constructed so as to have an exceedingly low probability of gross rupture or significant uncontrolled leakage throughout its design lifetime (AIF-GDC 9).

The reactor coolant system, in conjunction with its control and protective provisions, is designed to accommodate the system pressures and temperatures attained under all expected modes of plant operation or anticipated system interactions, and maintain the stresses within applicable code stress limits.

Fabrication of the components which constitute the pressure retaining boundary of the reactor coolant system is carried out in strict accordance with the applicable codes. In addition, there

are areas where equipment specifications for reactor coolant system components go beyond the applicable codes. Materials of construction were chosen to lessen the probability of gross leakage or failure. Details are given in Section 5.2.3.

The materials of construction of the pressure retaining boundary of the reactor coolant system are protected by control of coolant chemistry from corrosion phenomena which might otherwise reduce the system structural integrity during its service lifetime.

System conditions resulting from anticipated transients or malfunctions are monitored and appropriate action is automatically initiated to maintain the required cooling capability and to limit system conditions so that continued safe operation is possible.

The system is protected from overpressure by means of pressure relieving devices, as required by Section III of the ASME Code. Low temperature over-pressure protection is also provided, together with operating precautions to minimize operation under undesirable conditions (see Section 5.2.2).

Isolable sections of the system are provided with overpressure relieving devices discharging to closed systems such that the system code allowable relief pressure within the protected section is not exceeded.

3.1.1.2.5 Reactor Containment

CRITERION: Reactor containment shall be provided. The containment structure shall be designed (a) to sustain without undue risk to the health and safety of the public the initial effects of gross equipment failures, such as a large reactor coolant pipe break, without loss of required integrity and (b) together with other engineered safety features as may be necessary, to retain for as long as the situation requires the functional capability of the containment to the extent necessary to avoid undue risk to the health and safety of the public (AIF-GDC 10).

The reactor containment structure is a reinforced-concrete vertical cylinder with pre-stressed tendons in the vertical wall, a reinforced-concrete ring anchored to bedrock and a reinforced hemispherical dome. See Section 3.8.1.

The design pressure of the containment exceeds the peak pressure occurring as the result of the complete blowdown of the reactor coolant through any pipe rupture of the reactor coolant system up to and including the hypothetical severance of a reactor coolant pipe, as well as a postulated main steam line break. The containment structure and all penetrations are designed to withstand within design limits the combined loadings of the design-basis accident and design seismic conditions.

All piping systems which penetrate the containment are anchored in the penetration sleeve or the structural concrete of the Containment Building. The penetrations for the main steam, feedwater, blowdown, and sample lines are designed so that the penetration is stronger than the piping system and that the containment will not be breached due to a postulated pipe rupture. The lines connected to the primary coolant system that penetrate the containment and pass through the secondary shield walls (i.e., walls surrounding the steam generators and reactor coolant pumps) are also anchored in the primary shield walls and are each provided

with at least one valve between the anchor and the coolant system. These anchors are designed to withstand the thrust moment and torque resulting from a postulated rupture of the attached pipe.

All isolation valves are supported to withstand, without impairment of valve operability, the combined loadings of the design-basis accident and design seismic conditions.

3.1.1.3 Nuclear and Radiation Controls

3.1.1.3.1 Control Room

CRITERION: The facility shall be provided with a control room from which actions to maintain safe operational status of the plant can be controlled. Adequate radiation protection shall be provided to permit continuous occupancy of the control room under any credible postaccident condition or as an alternative access to other areas of the facility as necessary to shut down and maintain safe control of the facility without excessive radiation exposures of personnel (AIF-GDC 11).

The plant is equipped with a control room which contains all controls and instrumentation necessary for operation of the reactor and turbine generator under normal and accident conditions.

The control room is capable of continuous occupancy by the operating personnel under all operating and accident conditions.

Sufficient shielding, ventilation, and habitability provisions exist to ensure that control room personnel can perform all required safety functions from the control room, under all credible postulated accident conditions (see Section 6.4.1).

3.1.1.3.2 Instrumentation and Controls Systems

CRITERION: Instrumentation and controls shall be provided as required to monitor and maintain within prescribed operating ranges essential reactor facility operating variables (AIF-GDC 12).

Instrumentation and controls essential to avoid undue risk to the health and safety of the public are provided to monitor and maintain neutron flux, primary coolant pressure, flow rate, temperature, and control rod positions within prescribed operating ranges.

The non-nuclear regulating, process, and containment instrumentation measures temperature, pressure, flow, and levels in the reactor coolant system, steam systems, containment and other auxiliary systems. Process variables required on a continuous basis for the startup, operation, and shutdown of the plant are indicated, recorded, and controlled from the control room into which access is supervised. The quantity and types of process instrumentation provided ensures safe and orderly operation of all systems and processes over the full operating range of the plant.

The instrumentation and controls systems are discussed in Chapter 7.

3.1.1.3.3 Fission Process Monitors and Controls

CRITERION: Means shall be provided for monitoring or otherwise measuring and maintaining control over the fission process throughout core life under all conditions that can reasonably be anticipated to cause variations in reactivity of the core (AIF-GDC 13).

The nuclear instrumentation system is provided to monitor the reactor power from source range through the intermediate range and power range up to 120% of full power. The system provides indication, control, and alarm signals for reactor operation and protection.

The operational status of the reactor is monitored from the control room. When the reactor is sub-critical and during approach to criticality (i.e., during MODE 6, "Refueling" through MODE 3 "Hot Shutdown", and during MODE 2 "Startup"), the relative reactivity status (neutron source multiplication) is continuously monitored by two Source Range proportional counter detectors located in instrument wells within the primary shield and adjacent to the reactor vessel. Two source range detector channels are provided to supply neutron source multiplication information during the above-mentioned plant modes. A reactor trip is actuated from either channel if the neutron flux level becomes excessive.

The source range channels are checked prior to operations in which criticality may be approached. A source of neutrons is necessary to provide at least the minimum count rate (> 5 cps) required for startup operations. The discrete (Sb-Be) secondary sources initially installed were removed from the core during the refueling outage at the end of cycle 20. The neutron emissions which occur naturally in burnt fuel are now utilized as the neutron source. These neutron emissions are produced primarily by spontaneous fission of Cm-242 and Cm-244.

Any appreciable increase in the neutron source multiplication, including that caused by the maximum physical boron dilution rate, is slow enough to give ample time to start corrective action (boron dilution stop and/or emergency boron injection) to prevent the core from becoming critical.

When the reactor is critical, means for showing the relative reactivity status of the reactor is provided by control bank positions displayed in the control room. The position of the control banks is directly related to the reactivity status of the reactor when at power and any unexpected change in the position of the control banks under automatic control or change in the coolant temperature under manual control provides a direct and immediate indication of a change in the reactivity status of the reactor. Periodic samples of the coolant boron concentration are taken. The variation in concentration during core life provides a further check on the reactivity status of the reactor including core depletion.

High nuclear flux protection is provided both in the power and intermediate ranges by reactor trips actuated from either range if the neutron flux level exceeds trip setpoints. When the reactor is critical, the best indications of the reactivity status in the core (in relation to the power level and average coolant temperature) is the control room display of the rod control group position.

Reactor Trip System (RTS) instrumentation and controls are discussed in Section 7.2.1.

3.1.1.3.4 Core Protection Systems

CRITERION: Core protection systems, together with associated equipment, shall be designed to prevent or to suppress conditions that could result in exceeding acceptable fuel damage limits (AIF-GDC 14).

Instrumentation and controls provided for the protective systems are designed to trip the reactor when necessary to prevent or limit fission product release from the core; to limit energy release; to signal closure of containment isolation valves; and to control the operation of engineered safety features equipment.

During reactor operation in the startup and power modes, redundant safety limit signals will automatically actuate two reactor trip breakers which are in series with the rod drive mechanism coils. This action would interrupt power and initiate reactor trip. This criterion, as applied to the Reactor Trip System (RTS), is discussed more fully in Sections 3.1.1.4.8, 7.2.1, and 7.2.3.

3.1.1.3.5 Engineered Safety Features Protection Systems

CRITERION: Protection systems shall be provided for sensing accident situations and initiating the operation of necessary engineered safety features (AIF-GDC 15).

The Engineered Safety Features Actuation System (ESFAS) provides actuation of the following functions: safety injection, containment isolation, steam line isolation, containment spray and feedwater isolation, automatic diesel start-up, and preferred auxiliary feedwater pump startup.

The safety injection systems deliver water to the reactor core following a loss-of-coolant accident. The principal components of the safety injection system are two passive accumulators (one for each loop), three high-head safety injection pumps, two low-head safety injection (residual heat removal) pumps, and the essential piping and valves. A safety injection accumulator makeup pump is available to fill the accumulator tanks when there is a need, due to miscellaneous system leakage. The accumulators are passive devices which discharge into the cold leg of each loop.

The safety injection system may be actuated by two-out-of-three low-pressurizer-pressure signals, two-out-of-three low-steam-line-pressure signals, two-out-of-three high-containment-pressure signals; or the system can be actuated manually. Any of the safety injection system signals will open the system isolation valves, start the high-head safety injection pumps and the low-head (residual heat removal) pumps (see Section 6.3).

The steam line isolation valves are closed upon receipt of high steam line flow in conjunction with a safety injection system signal, by containment pressure, or by manual initiation. See Section 6.2.4.3 and Section 3.2.2.1 for a more current and detailed description of steam line isolation.

The containment spray system consists of two pumps, one spray additive tank, valves, piping, and spray nozzles. Containment spray is initiated by coincident signals from two sets of two-out-of-three containment pressure signals monitoring containment high-high pressure. The actuation signal starts the pumps and opens the discharge valves to the spray header. Valves for the spray additive tank open after a very short time delay.

Containment isolation is initiated by an automatic safety injection system signal or manually. Actuation of containment isolation trips the containment sump pumps, closes containment isolation valves (as discussed in Section 6.2.4 and listed in Tables 6.2-15 and 6.2-16), and trips the purge supply and exhaust fans. Containment ventilation isolation and depressurization valves are also isolated on high containment activity (R-11 and R-12), any safety injection signal, or from a manual containment spray signal. See Section 6.2.4.3 for a more current and detailed description of containment isolation and containment ventilation isolation.

The feedwater isolation system consists of the two main feedwater regulating valves, two main feedwater regulating valve bypass valves, and two main feedwater isolation valves. The main feedwater regulating valves and the main feedwater regulating bypass valves close when they receive a safety injection system signal or an engineered safety feature sequence initiation signal. They fail closed if power or air is lost. The two main feedwater isolation valves close when they receive a safety injection signal. They fail close if power or instrument air is lost. See Section 7.3.2.2.2 for a more detailed description of feedwater isolation.

As part of the plant uprate to 1775 MWt two manual feedwater isolation valves were upgraded to automatic isolation valves by the installation of an air actuator on each valve. The modifications provided an additional automatic feedwater isolation valve for each SG. These new automatic isolation valves provide redundancy to the automatic isolation function provided by the two main feedwater regulating valves and two main feedwater by-pass valves.

Automatic diesel startup will be caused by undervoltage at the engineered safety features buses in addition to being caused by the safety injection signal.

The motor-driven auxiliary feedwater pumps (MDAFW) start upon a safety injection signal, either steam generator low-low level, loss of both main feedwater pumps or ATWS Mitigation System Actuation Circuitry (AMSAC). The turbine-driven auxiliary feedwater pump (TDAFW) will start on low-low level in both steam generators and loss of bus voltage on 11A and 11B. See Section 7.3.2.2.2 and Section 7.2.6 for a more current and detailed description of auxiliary feedwater pump starts.

3.1.1.3.6 Monitoring Reactor Coolant Leakage

CRITERION: Means shall be provided to detect significant uncontrolled leakage from the reactor coolant pressure boundary (AIF-GDC 16).

Positive indications in the control room of leakage of coolant from the reactor coolant system to the containment are provided by equipment which permits continuous monitoring of containment air activity (R-11 and R-12) and humidity, containment sump A level (LT-2039 and LT-2044), and of runoff from the condensate collection system under the cooling coils of

The containment recirculation fan cooler (CRFC) units. This equipment provides indication of normal background which is indicative of a basic level of leakage from primary systems and components. Any increase in the observed parameters is an indication of change within the containment, and the equipment provided is capable of monitoring this change. The basic design criterion is the detection of deviations from normal containment environmental conditions including air particulate activity, radiogas activity, humidity, condensate runoff, and the liquid inventory in the process systems and containment sump A. Further details are supplied in Section 5.2.5.

3.1.1.3.7 Monitoring Radioactivity Releases

CRITERION: Means shall be provided for monitoring the containment atmosphere and the facility effluent discharge paths for radioactivity released from MODES 1 and 2, from anticipated transients, and from accident conditions. An environmental monitoring program shall be maintained to confirm that radioactivity releases to the environs of the plant have not been excessive (AIF-GDC 17).

The containment atmosphere, the containment purge, the plant vent, the containment fan-coolers service water (SW) discharge, the waste disposal system liquid effluent, and the spent fuel pool (SFP) heat exchanger raw water discharge are monitored for radioactivity concentration during MODES 1 and 2, from anticipated transients, and from accident conditions.

All gaseous effluent from possible sources of accidental releases of radioactivity external to the reactor containment (e.g., the spent fuel pool (SFP) and waste handling equipment) will be exhausted from the plant vent which is monitored. All accidental spills of liquids are maintained within the auxiliary building and collected in a drain tank. Any contaminated liquid effluent discharged to the condenser circulating water canal is monitored.

Process radiation monitoring and area radiation monitoring are described in Sections 11.5.2.2 and12.3.4, respectively.

Additional details of offsite radiological monitoring are provided in the Offsite Dose Calculation Manual (ODCM).

3.1.1.3.8 Monitoring Fuel and Waste Storage

CRITERION: Monitoring and alarm instrumentation shall be provided for fuel and waste storage and associated handling areas for conditions that might result in loss of capability to remove decay heat and to detect excessive radiation levels (AIF-GDC 18).

Monitoring and alarm instrumentation is provided for fuel and waste storage and handling areas to detect inadequate cooling and to detect excessive radiation levels. Radiation monitors are provided to maintain surveillance over the release operation.

The spent fuel pool (SFP) cooling system flow is monitored to ensure proper operation as described in Section 9.1.3.

A controlled ventilation system removes gaseous radioactivity from the atmosphere and fuel storage and waste treating areas of the auxiliary building and discharges it to the atmosphere via the plant vent. Radiation monitors are in continuous service in these areas to actuate high activity alarms on the control board annunciator, as described in Sections 11.5 and 12.3.

3.1.1.4 Reliability and Testability of Protection Systems

3.1.1.4.1 Protection Systems Reliability

CRITERION: Protection systems shall be designed for high functional reliability and inservice testability necessary to avoid undue risk to the health and safety of the public (AIF-GDC 19).

The reactor uses a higher speed version of the Westinghouse magnetic-type control rod drive mechanisms used in the San Onofre and Connecticut Yankee plants. The original control rod drive mechanisms (CRDM) supplied to Ginna Station were replaced with equivalent model L-106 during the 2003 refueling outage by modification PCR 2001-0042. Upon a loss of power to the coils, the rod cluster control assembly is released and falls by gravity into the core.

The reactor internals, fuel assemblies, control rods, and control rod drive system components (as required for trip) are designed as Seismic Category I equipment. The control rods are fully guided through the fuel assembly and for the maximum travel of the control rod into the guide tube. Furthermore, the control rods are never fully withdrawn from their guide thimbles in the fuel assembly. Due to this and the flexibility designed into the control rods, abnormal loadings and misalignments can be sustained without impairing operation of the control rods.

The control rod guide system throughout its length is locked together with pins, bolts and welds to ensure against misalignments which might impair control rod movement under normal operating conditions and credible accident conditions.

All reactor protection channels are supplied with sufficient redundancy to provide the capability for channel calibration and test at power. Bypass removal of one trip circuit is accomplished by placing that circuit in a half-tripped mode; i.e., a two-out-of-three circuit becomes a one-out-of-two circuit. Testing does not trip the system unless a trip condition exists in a concurrent channel.

Reliability and independence is obtained by redundancy within each tripping function. In a two-out-of-three circuit, for example, the three channels are equipped with separate primary sensors. Each channel is continuously fed from its own independent electrical sources. Failure to deenergize a channel when required would be a mode of malfunction that would affect only that channel. The trip signal furnished by the two remaining channels would be unimpaired in this event.

Routing and separation standards applicable to existing cables are those that were invoked at the time of cable installation. For more information, see Section 8.3.1.4.

3.1.1.4.2 Protection Systems Redundancy and Independence

CRITERION: Redundancy and independence designed into protection systems shall be sufficient to assure that no single failure or removal from service of any component or channel of such a system will result in loss of the protection function. The redundancy provided shall include, as a minimum, two channels of protection for each protection function to be served (AIF-GDC 20).

3.1.1.4.2.1 Reactor Trip Circuits

Two reactor trip breakers are provided to interrupt power to the rod drive mechanisms. The breaker main contacts are connected in series with the power supply to the mechanism coils. Opening either breaker interrupts power to the magnetic latch mechanisms on each control rod drive causing them to release the rods to fall by gravity into the core. Each breaker is opened through an undervoltage trip coil. Each protection channel actuates two separate trip logic trains, one for each reactor trip breaker undervoltage trip coil. The protection system is thus inherently safe in the event of a loss of rod control power.

The coincident trip philosophy is carried out to provide a safe and reliable system since a single failure will not defeat the function of a redundant channel and will also not cause a spurious plant trip. Channel independence is carried throughout the system extending from the sensor to the relay providing the logic. In most cases, the safety and control functions when combined are combined only at the sensor (and power supply). Both functions are fully isolated in the remaining part of the channel, control being derived from the primary safety signal path through an isolation amplifier. As such, a failure in the control circuitry does not affect the safety channels. This approach is used for pressurizer pressure and water level channels, steam generator water level, T_{AVG} and delta T channels, steam flow, and nuclear power range channels.

The power supplies to the channels are fed from four instrument buses. Two of the buses are supplied by constant voltage transformers and two are supplied by inverters.

3.1.1.4.2.2 Engineered Safety Features Initiation Circuits

The initiation of the engineered safety features provided for loss-of-coolant accidents, e.g., high-head safety injection and residual heat removal pumps, and containment spray systems, is accomplished from several signals derived from reactor coolant system and containment instrumentation. Channel independence is carried throughout the system from the sensors to the signal output relays including the power supplies for the channels. The initiation signal for containment spray comes from coincidence of two sets of two-out-of-three high-high-containment-pressure signals. On loss of voltage to the safeguards bus, the diesel generator will be automatically started and connected to the bus.

The signal for containment isolation of non-vital valves, i.e., the isolation valves trip signal, is derived from a coincidence of two-out-of-three containment high-pressure signals. This setpoint is below that for containment spray actuation. For this circuit also, the channels are independent from sensor to output relay and are supplied from independent power sources.

Redundancy is provided in that there are two diesel-generator sets capable of supplying the separate 480-V safeguards buses. One complete set of safety features equipment is therefore independently supplied from each diesel generator.

In the event that either diesel generator fails to start, a bus tie breaker may be manually closed by the operator to connect the 480-V safeguards bus to the second diesel-generator set. This would then allow a duplicate safety feature component from the bus associated with a failed diesel generator to be fed from the other bus in the event of a component failure. In the event of a fault on either bus, closing of the tie breaker is blocked.

Required continuous electrical power supply is discussed in Chapter 8.

3.1.1.4.3 Single-Failure Definition (Category B)

CRITERION: Multiple failures resulting from a single event shall be treated as a single failure (AIF-GDC 21).

The requirements of this criterion are included in Section 3.1.1.4.5.

3.1.1.4.4 Separation of Protection and Control Instrumentation Systems

CRITERION: Protection systems shall be separated from control instrumentation systems to the extent that failure or removal from service of any control instrumentation system component or channel, or of those common to control instrumentation and protection circuitry, leaves intact a system satisfying all requirements for the protection channels (AIF-GDC 22).

The requirements of this criterion are included in Section 3.1.1.4.2.

3.1.1.4.5 Protection Against Multiple Disability for Protection Systems

CRITERION: The effects of adverse conditions to which redundant channels or protection systems might be exposed in common, either under normal conditions or those of an accident, shall not result in loss of the protection function or shall be tolerable on some other basis (AIF-GDC 23).

The components of the protection system are qualified such that the mechanical and thermal adverse environment resulting from emergency situations during which the components are required to function does not prevent them from accomplishing their safety function.

3.1.1.4.6 Emergency Power for Protection Systems

CRITERION: In the event of loss of all offsite power, sufficient alternate sources of power shall be provided to permit the required functioning of the protection systems (AIF-GDC 24).

The requirements of this criterion are included in Section 3.1.1.7.3.

3.1.1.4.7 Demonstration of Functional Operability of Protection Systems

CRITERION: Means shall be included for suitable testing of the active components of protection systems while the reactor is in operation to determine if failure or loss of redundancy has occurred (AIF-GDC 25).

Each protection channel in service at power is capable of being calibrated and tripped independently by simulated signals for test purposes to verify its operation. This includes checking through to the trip breakers which necessarily involves the trip logic. Thus, the operability of each trip channel can be determined conveniently and without ambiguity.

Periodic testing of the diesel generators is routinely performed to ensure their operability. During power operation, surveillance testing verifies that the fuel transfer system is operational, the diesels start from normal standby conditions, the generators are properly synchronized and loaded, and that proper alignment is made so that the diesel generators could supply safeguards bus power. During shutdown conditions, the diesel generators are tested to ensure they can restore safeguards bus voltage in a timely manner by automatically actuating breakers in the time period required.

3.1.1.4.8 Protection Systems Failure Analysis Design

CRITERION: The protection systems shall be designed to fail into a safe state or into a state established as tolerable on a defined basis if conditions such as disconnection of the systems, loss of energy (e.g., electrical power, instrument air), or adverse environments (e.g., extreme heat or cold, fire, steam, or water) are experienced (AIF-GDC 26).

Each reactor trip circuit is designed so that trip occurs when the circuit is deenergized; an open circuit or loss of channel power therefore causes the system to go into its trip mode. In a two-out-of-three circuit, the three channels are equipped with separate primary sensors and each channel is energized from independent electrical buses. Failure to deenergize when required is a mode of malfunction that affects only one channel. The trip signal furnished by the two remaining channels is unimpaired in this event.

The signal for containment isolation of nonvital valves is developed from a two-out-of-three circuit in which each channel is separate and independent and which signals for containment isolation upon loss of power. The failure of any channel to deenergize when required does not interfere with the proper functioning of the isolation circuit.

Reactor trip is implemented by interrupting power to the magnetic latch mechanisms on each drive, allowing the rod clusters to insert by gravity. The protection system is thus inherently safe in the event of a loss of power.

Automatic starting of either emergency diesel generator is initiated by redundant undervoltage relays on the 480-V safeguards bus to which the diesel generator is connected or by the safety injection signal. Engine cranking is accomplished by a stored energy system supplied solely for the associated diesel generator. The undervoltage relay scheme is designed so that loss of 480-V power does not prevent the relay scheme from functioning properly.

3.1.1.5 Reactivity Control

3.1.1.5.1 Redundancy of Reactivity Control

CRITERION: Two independent reactivity control systems, preferably of different principles, shall be provided (AIF-GDC 27).

In addition to the reactivity control achieved by the control rods, reactivity control is provided by the chemical and volume control system which regulates the concentration of boric acid solution neutron absorber in the reactor coolant system. The system is designed to prevent, under anticipated system malfunction, uncontrolled or inadvertent reactivity changes which might stress the system beyond allowable limits.

3.1.1.5.2 Reactivity Hot Shutdown Capability

CRITERION: The reactivity control systems provided shall be capable of making and holding the core subcritical from any hot standby or hot operating condition (AIF-GDC 28).

The reactivity control systems provided are capable of making and holding the core subcritical from any hot standby condition, including those resulting from power changes. The maximum excess reactivity expected for the core occurs for the cold, clean condition at the beginning of each cycle.

The control rods are divided into two categories comprising a control group and shutdown groups. The control group, used in combination with chemical shim (soluble boron), provides control of the reactivity changes of the core throughout the life of the core at power conditions. This group of control rods is used to compensate for short-term reactivity changes at power that might be produced due to variations in reactor power requirements or in coolant temperature. The chemical shim control is used to compensate for the more slowly occurring changes in reactivity throughout core life such as those due to fuel depletion and fission product buildup and decay.

3.1.1.5.3 Reactivity Shutdown Capability

CRITERION: One of the reactivity control systems provided shall be capable of making the core subcritical under any anticipated operating condition (including anticipated operational transients) sufficiently fast to prevent exceeding acceptable fuel damage limits. Shutdown margin should assure subcriticality with the most reactive control rod fully withdrawn (AIF-GDC 29).

The shutdown groups are provided to supplement the control group of control rods to make the reactor subcritical with the required shutdown margin following trip from any credible operating condition to the hot, zero power condition assuming the most reactive rod cluster control assembly remains in the fully withdrawn position. Manually controlled boric acid addition is used to supplement the rod cluster control assemblies in maintaining the shutdown margin for the long-term conditions of xenon decay or plant cooldown. See Sections 4.2.1 and 9.3.4 concerning details of the control rods and chemical and volume control systems.

3.1.1.5.4 Reactivity Hold-Down Capability

CRITERION: The reactivity control systems provided shall be capable of making the core subcritical under credible accident conditions with appropriate margins for contingencies and limiting any subsequent return to power such that there will be no undue risk to the health and safety of the public (AIF-GDC 30).

Normal reactivity shutdown capability is provided by control rods with boric acid injection used to compensate for the long-term xenon decay transient and for plant cooldown. Any time that the plant is at power, the quantity of boric acid retained in the boric acid tanks or refueling water storage tank (RWST) and ready for injection will always exceed that quantity required for the normal MODE 5 (Cold Shutdown). This quantity will also exceed the quantity of boric acid required to bring the reactor to MODE 3 (Hot Shutdown) and to compensate for subsequent xenon decay.

The boric acid solution is transferred from the boric acid storage tanks by boric acid transfer pumps to the suction of the charging pumps which inject boric acid into the reactor coolant. Any charging pump and boric acid transfer pump can be operated from diesel-generator power on loss of primary power. Boric acid injection from the Boric Acid Storage Tanks (BAST) to the RCS by one charging pump operating at its nominal charging flow rate of 46 gpm is capable of shutting down the reactor with no rods inserted in approximately 81 minutes.

Sufficient boric acid from the BAST or the Refueling Water Storage Tank (RWST) can also be injected to compensate for xenon decay beyond the equilibrium level, with one charging pump operating at its minimum speed, and thereby delivering in excess of the required minimum flow of approximately 9 gpm into the reactor coolant system. This required flow rate is checked on a cycle specific basis. Additional boric acid is employed if it is desired to bring the reactor to MODE 5 (Cold Shutdown) conditions.

On the basis of the above, the injection of boric acid is shown to afford backup reactivity shutdown capability, independent of control rod clusters which normally serve this function in the short-term situation. Shutdown for long-term and reduced temperature conditions can be accomplished with boric acid injection using redundant components. Furthermore, boric acid from the refueling water storage tank (RWST) can also be transferred to the reactor coolant system via the charging pumps.

3.1.1.5.5 Reactivity Control Systems Malfunction

CRITERION: The Reactor Trip System (RTS) shall be capable of protecting against any single malfunction of the reactivity control system, such as unplanned continuous withdrawal (not ejection or dropout) of a control rod, by limiting reactivity transients to avoid exceeding acceptable fuel damage limits (AIF-GDC 31).

As described in Chapter 7, the Reactor Trip System (RTS) is designed to limit reactivity transients to DNBR greater than or equal to the safety limit due to any single malfunction in the deboration controls.

Reactor shutdown with control rods is completely independent of the normal control functions since the trip breakers completely interrupt the power to the rod mechanisms regardless of existing control signals.

Details of the effects of continuous withdrawal of a control rod and of continuous deboration are described in Sections 15.4.1 and 15.4.4.

3.1.1.5.6 Maximum Reactivity Worth of Control Rods

CRITERION: Limits, which include reasonable margin, shall be placed on the maximum reactivity worth of control rods or elements and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change or reactivity cannot (a) rupture the reactor coolant pressure boundary or (b) disrupt the core, its support structures, or other vessel internals sufficiently to lose capability of cooling the core (AIF-GDC 32).

Limits, which include considerable margin, are placed on the maximum reactivity worth of control rods or elements and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change of reactivity cannot (a) rupture the reactor coolant pressure boundary or (b) disrupt the core, its support structures, or other vessel internals so as to lose capability to cool the core.

The reactor coolant system employs control rods, less than half of which are fully withdrawn during power operation, serving as shutdown rods. The remaining rods comprise the controlling group which are used to control load and reactor coolant temperature. The control rod drive mechanisms are wired into preselected groups, and are therefore prevented from being withdrawn in other than their respective groups. The control rod drive mechanism is of the magnetic latch type and the coil actuation is sequenced to provide variable speed rod travel. The maximum reactivity insertion rate is analyzed in the detailed plant analysis described in Section 15.4.

No credible mechanical or electrical control system malfunction can cause a control rod to be withdrawn at a speed greater than 77 steps per minute.

3.1.1.6 Reactor Coolant Pressure Boundary

3.1.1.6.1 Reactor Coolant Pressure Boundary Capability

CRITERION: The reactor coolant pressure boundary shall be capable of accommodating without rupture the static and dynamic loads imposed on any boundary component as a result of an inadvertent and sudden release of energy to the coolant. As a design reference, this sudden release shall be taken as that which would result from a sudden reactivity insertion such as rod ejection (unless prevented by positive mechanical means), rod dropout, or cold water addition (AIF-GDC 33).

The reactor coolant boundary is shown to be capable of accommodating without further rupture the static and dynamic loads imposed as a result of a sudden reactivity insertion such as a rod ejection. Details of this analysis are provided in Section 15.4.5.

The operation of the reactor is such that the severity of an ejection accident is inherently limited. Since control rod clusters are used to control load variations only and core depletion is followed with boron dilution, only the rod cluster control assemblies in the controlling groups are inserted in the core at power, and at full power these rods are only partially inserted. A rod insertion limit monitor is provided as an administrative aid to the operator to ensure that this condition is met.

By using the flexibility in the selection of control rod groupings, radial locations and position as a function of load, the design limits the maximum fuel temperature for the highest worth ejected rod to a value which precludes any resultant damage to the primary system, pressure boundary, i.e., gross fuel dispersion in the coolant and possible excessive pressure surges.

The failure of a rod mechanism housing causing a control rod to be rapidly ejected from the core is evaluated as a theoretical, though not a credible, accident. While limited fuel damage could result from this hypothetical event, the fission products are confined to the reactor coolant system and the reactor containment. The environmental consequences of rod ejection are less severe than from the postulated loss-of-coolant accident, for which public health and safety are shown to be adequately protected.

3.1.1.6.2 Reactor Coolant Pressure Boundary Rapid Propagation Failure Prevention

CRITERION: The reactor coolant pressure boundary shall be designed and operated to reduce to an acceptable level the probability of rapidly propagating type failures. Consideration shall be given (a) to the provisions for control over service temperature and irradiation effects which may require operational restrictions, (b) to the design and construction of the reactor pressure vessel in accordance with applicable codes, including those which establish requirements for absorption of energy within the elastic strain energy range and for absorption of energy by plastic deformation and (c) to the design and construction of reactor coolant pressure boundary piping and equipment in accordance with applicable codes (AIF-GDC 34).

The reactor coolant pressure boundary is designed to reduce to an acceptable level the probability of a rapidly propagating type failure.

In the core region of the reactor vessel it is expected that the notch toughness of the material will change as a result of fast neutron exposure. This change is evidenced as a shift in the nil ductility transition temperature (NDTT) which is factored into the operating procedures in such a manner that full operating pressure is not obtained until the affected vessel material is above the now higher design transition temperature (DTT) and in the ductile material region. The pressure during startup and shutdown at the temperature below NDTT is maintained below the threshold of concern for safe operation.

The DTT is a minimum of NDTT plus 60°F and dictates the procedures to be followed in the hydrostatic test and in station operations to avoid excessive cold stress. The value of the DTT is increased during the life of the plant, as required by the expected shift in the NDTT and as confirmed by the experimental data obtained from irradiated specimens of reactor vessel material during the plant lifetime. Further details are given in Sections 5.2 and 5.3.

Low temperature reactor vessel overpressure protection is discussed in Section 5.2.2. Pressurized thermal shock of the reactor vessel is discussed in Section 5.3.3.5.

All pressure-containing components of the reactor coolant system are designed, fabricated, inspected, and tested in conformance with the applicable codes. Further details are given in Section 5.2.1.2.

3.1.1.6.3 Reactor Coolant Pressure Boundary Brittle Fracture Prevention

CRITERION: Under conditions where reactor coolant pressure boundary system components constructed of ferritic materials may be subjected to potential loadings, such as a reactivity-induced loading, service temperatures shall be at least 120°F above the nil ductility transition temperature (NDTT) of the component material if the resulting energy release is expected to be absorbed by plastic deformation or 60°F above the NDTT of the component material if the resulting energy release is expected to be absorbed within the elastic strain energy range (AIF-GDC 35).

The requirements of this criterion are included in Section 3.1.1.6.2.

3.1.1.6.4 Reactor Coolant Pressure Boundary Surveillance

CRITERION: Reactor coolant pressure boundary components shall have provisions for inspection, testing, and surveillance of critical areas by appropriate means to assess the structural and leaktight integrity of the boundary components during their service lifetime. For the reactor vessel, a material surveillance program conforming with current applicable codes shall be provided (AIF-GDC 36).

The design of the reactor vessel and its arrangement in the system provides the capability for accessibility during service life to the entire internal surfaces of the vessel and certain external zones of the vessel including the nozzle to reactor coolant piping welds and the top and bottom heads. The reactor arrangement within the containment provides sufficient space for inspection of the external surfaces of the reactor coolant piping, except for the area of pipe within the primary shielding concrete.

Monitoring of the NDTT properties of the core region plate forgings, weldments, and associated heat-treated zones are performed in accordance with ASTM E185, Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors. Samples of reactor vessel plate materials are retained and cataloged in case future engineering development shows the need for further testing.

The material properties surveillance program includes not only the conventional tensile and impact tests but also fracture mechanics specimens. The fracture mechanics specimens are the wedge-opening loading type specimens. The observed shifts in NDTT of the core region

materials with irradiation will be used to confirm the calculated limits to startup and shut-down transients.

To define permissible operating conditions below DTT, a pressure range is established which is bounded by a lower limit for pump operation and an upper limit which satisfies reactor vessel stress criteria. To allow for thermal stresses during heatup or cooldown of the reactor vessel, an equivalent pressure limit is defined to compensate for thermal stress as a function of rate of change of coolant temperature. The reactor coolant temperature and pressure and the system heatup and cooldown rates allowable are discussed in Section 5.1.3.9.

Since the normal operating temperature of the reactor vessel is well above the maximum expected DTT, brittle fracture during MODES 1 and 2 is not considered to be a credible mode of failure. The reactor vessel has been evaluated for potential damage due to "Pressurized Thermal Shock" (Unresolved Safety Issue A-49) and it was concluded that the potential for damage was acceptably small. A discussion of reactor vessel integrity under transient conditions is discussed in Sections 5.3.3.4 and 5.3.3.5.

3.1.1.7 Engineered Safety Features

3.1.1.7.1 Engineered Safety Features Basis for Design

CRITERION: Engineered safety features shall be provided in the facility to back up the safety provided by the core design, the reactor coolant pressure boundary, and their protection systems. Such engineered safety features shall be designed to cope with any size reactor coolant piping break up to and including the equivalent of a circumferential rupture of any pipe in that boundary assuming unobstructed discharge from both ends (AIF-GDC 37).

The design, fabrication, testing, and inspection of the core, reactor coolant pressure boundary, and their protection systems give assurance of safe and reliable operation under all anticipated normal, transient, and accident conditions. However, engineered safety features are provided in the facility to back up the safety provided by these components. These engineered safety features have been designed to cope with any size reactor coolant pipe break up to and including the circumferential rupture of any pipe in that boundary assuming unobstructed discharge from both ends, and to cope with any steam or feedwater line break up to and including the main steam or feedwater headers.

The release of fission products from the reactor fuel is limited by the safety injection system which, by cooling the core, keeps the fuel in place and substantially intact and limits the metal-water reaction.

The safety injection system consists of high and low-head centrifugal pumps driven by electric motors and passive accumulator tanks which are self-energized and which act independently of any actuation signal or power source.

The release of fission products from the containment is limited in three ways:

1. Blocking the potential leakage paths from the containment. This is accomplished by

a. A steel-lined concrete reactor containment with testable, double penetrations and liner weld channels which form a virtually leaktight barrier to the escape of fission products should a loss of coolant occur.

b. Isolation of process lines by the containment isolation system which imposes double barriers in each line that penetrates the containment.

2. Reducing the fission product concentration in the containment atmosphere. This is accomplished by

a. Air recirculation filters which provide for rapid removal of particles and iodine vapor from the containment atmosphere.

b. Chemically treated spray which removes elemental iodine vapor from the containment atmosphere by washing action.

3. Reducing the containment pressure and thereby limiting the driving potential for fission product leakage. This is accomplished by cooling the containment atmosphere by the following independent systems

a. Containment spray system.

b. Containment recirculation fan cooler (CRFC) and filtration system.

3.1.1.7.2 Reliability and Testability of Engineered Safety Features

CRITERION: All engineered safety features shall be designed to provide such functional reliability and ready testability as is necessary to avoid undue risk to the health and safety of the public (AIF-GDC 38).

A comprehensive program of plant testing is performed for all equipment systems and system controls vital to the functioning of engineered safety features. The program consists of performance tests of individual pieces of equipment in the manufacturer's shop, and integrated tests of the system as a whole, and periodic tests of the actuation circuitry and mechanical components to ensure reliable performance, upon demand, throughout the plant lifetime.

The initial tests of the individual components and the integrated test of the system as a whole complement each other to ensure performance of the system as designed and to prove proper operation of the actuation circuitry.

Routine periodic testing of the engineered safety features components is scheduled. In the event that one of the components should require maintenance as a result of failure to perform during the test according to prescribed limits, the necessary corrections or minor maintenance will be made as required by the Technical Specifications.

3.1.1.7.3 Emergency Power

CRITERION: An emergency power source shall be provided and designed with adequate independency, redundancy, capacity, and testability to permit the functioning of the engineered safety features and protection systems required to avoid undue risk to the health and safety of the public. This power source shall provide this capacity assuming a failure of a single active component (AIF-GDC 39).

Independent, redundant, alternate power systems are provided with adequate capacity and testability to supply the required engineered safety features.

The plant is supplied with normal, standby and emergency power sources as follows:

A. The normal source of auxiliary power during plant operation is the generator. Power is supplied via the unit auxiliary transformer 11 that is connected to the main leads of the generator, except for safeguards loads required during MODES 1 and 2, which are supplied from transformer 12A and the offsite source. See Section 8.2.1.2 for an updated description of the supply to the safeguards loads.

B. Standby power required during plant startup, shutdown, and after reactor trip is supplied from the high-tension transmission terminal which has multiple lines running to the interconnected system.

C. Two diesel-generator sets are connected to the engineered safety features buses to supply emergency shutdown power in the event of loss of all other ac auxiliary power.

D. Emergency power supply for vital instruments and control and for emergency lighting is supplied from the two 125-V dc station batteries.

Although the engineered safety features loads are arranged to operate from electrical buses supplied from normal outside ac power which is designed to remain functional following reactor trip, reliable onsite emergency power is provided. Thus, if normal ac power to the station is lost concurrent with a loss-of-coolant accident, power is available for the engineered safety features. Two diesel-generator sets, each capable of supplying the necessary engineered safety features or safe shutdown loads, are provided. Details are provided in Sections 8.1.4.2 and 8.3.1.1.

3.1.1.7.4 Missile Protection

CRITERION: Adequate protection for those engineered safety features, the failure of which could cause an undue risk to the health and safety of the public, shall be provided against dynamic effects and missiles that might result from plant equipment failures (AIF-GDC 40).

A loss-of-coolant accident or other plant equipment failure might result in dynamic effects or missiles. For such engineered safety features as are required to ensure safety in the event of such an accident or equipment failure, protection from these dynamic effects or missiles is considered in the layout of plant equipment and missile barriers. Fluid and mechanical driving forces are calculated and consideration is given to the possibility of damage due to fluid jets and missiles which might be produced by the action of such jets. Consideration is given during the design of the following potential sources of missiles: valve stems and bonnets, instrument thimbles including installed sensors, bolts, complete control rod drive shafts and/ or mechanisms, and rotating components. Consideration is also given to pipe whip effects.

Layout and structural design specifically protect injection paths leading to unbroken reactor coolant loops against damage as a result of the maximum reactor coolant pipe rupture. Injection lines penetrate the main missile barrier, and the injection headers are located in the missile-protected area between the missile barrier and the containment outside wall.

Individual injection lines, connected to the injection header, pass through the barrier and then connect to the loops. Separation of the individual injection lines is provided to the maximum extent practicable. Movement of the injection line, associated with rupture of a reactor coolant loop, is accommodated by line flexibility and by the design of the pipe supports.

All hangers, stops, and anchors are designed in accordance with USAS B31.1, Code for Pressure Piping, and ACI 318, Building Code Requirements for Reinforced Concrete, which provides minimum requirements on material, design, and fabrication with ample safety margins for both dead and dynamic loads over the life of the equipment. Additional information is provided in Sections 3.5 and 3.6.

3.1.1.7.5 Engineered Safety Features Performance Capability

CRITERION: Engineered safety features, such as the Emergency Core Cooling System (ECCS) and the containment heat removal system, shall provide sufficient performance capability to accommodate the failure of any single active component without resulting in undue risk to the health and safety of the public (AIF-GDC 41).

Each engineered safety feature provides sufficient performance capability to accommodate any single failure of an active component and still function in a manner to avoid undue risk to the health and safety of the public.

The extreme upper limit of public exposure is taken as the levels and time periods presently outlined in 10 CFR 100. The accident condition considered is the hypothetical case of a release of fission products per TID 14844. Also, the total loss of all offsite power is assumed concurrent with this accident. In *Reference 2*, the NRC approved the use of alternate source term (AST) methodology as defined in 10CFR50.67 for use by Ginna in determining offsite doses. The AST methodology was used during the power uprate to 1775 MWt to calculate offsite doses.

Under the above accident conditions, all engineered safety features equipment is designed to accomplish its safety function, assuming the worst case single failure.

3.1.1.7.6 Engineered Safety Features Components Capability

CRITERION: Engineered safety features shall be designed so that the capability of these features to perform their required function is not impaired by the effects of a loss of-coolant accident to the extent of causing undue risk to the health and safety of the public (AIF-GDC 42).

All active components of the safety injection system (with the exception of residual heat removal low-pressure safety injection line discharge valves) and the containment spray system are located outside the containment and are not subject to containment accident conditions.

Instrumentation, motors, cables, and penetrations located inside the containment are selected to meet the most adverse accident conditions to which they may be subjected. These items are either protected from containment accident conditions or are designed to withstand,

without failure, exposure to the worst combination of temperature, pressure, and humidity expected during the required operational period.

The piping and other components of the engineered safety features systems are designed and qualified to perform their safety function during and after the accident conditions, with concurrent seismic forces and accident operational loadings.

3.1.1.7.7 Accident Aggravation Prevention

CRITERION: Protection against any action of the engineered safety features which would accentuate significantly the adverse aftereffects of a loss of normal cooling shall be provided (AIF-GDC 43).

The reactor is maintained subcritical following a primary system pipe rupture accident. Introduction of borated cooling water into the core results in a net negative reactivity addition.

The delivery of cold safety injection water to the reactor vessel following a reactor coolant system break or secondary system break will not further adversely affect the integrity of the reactor coolant pressure boundary.

3.1.1.7.8 Emergency Core Cooling System (ECCS) Capability

CRITERION: An Emergency Core Cooling System (ECCS) with the capability for accomplishing adequate emergency core cooling shall be provided. This core cooling system and the core shall be designed to prevent fuel and clad damage that would interfere with the emergency core cooling function and to limit the clad metal-water reaction to acceptable amounts for all sizes of breaks in the reactor coolant piping up to the equivalent of a double-ended rupture of the largest pipe. The performance of such an Emergency Core Cooling System (ECCS) is evaluated conservatively in each area of uncertainty (AIF-GDC 44).

Adequate emergency core cooling is provided by the safety injection system which constitutes the Emergency Core Cooling System (ECCS) whose components include the passive accumulators, high-pressure safety injection, and residual heat removal low pressure safety injection and recirculation.

The primary purpose of the safety injection system is to automatically deliver cooling water to the reactor core to limit the fuel clad temperature and thereby ensure that the core will remain intact and in place, with its essential heat transfer geometry preserved. This protection is prescribed for all break sizes up to and including the hypothetical instantaneous double-ended rupture of the reactor coolant pipe, the rod ejection accident, a steam or feedwater line break, the steam generator tube rupture, and other accidents analyzed in Chapter 15.

The ability of the safety injection system to meet its capability objectives is presented in Section 6.3.3.

3.1.1.7.9 Inspection of Emergency Core Cooling System (ECCS)

CRITERION: Design provisions shall, where practical, be made to facilitate physical inspection of all critical parts of the Emergency Core Cooling System (ECCS), including reactor vessel internals and water injection nozzles (AIF-GDC 45).

Design provisions are made to the extent practical to facilitate access to the critical parts of the reactor vessel internals, injection nozzles, pipes, valves, and safety injection pumps for visual, boroscopic, and ultrasonic inspection for erosion, corrosion, and vibration wear evidence, and for nondestructive test inspection where such techniques are desirable and appropriate.

3.1.1.7.10 Testing of Emergency Core Cooling System (ECCS) Components

CRITERION: Design provisions shall be made so that components of the Emergency Core Cooling System (ECCS) can be tested periodically for operability and functional performance (AIF-GDC 46).

Design provisions are made so that active components of the safety injection system can be tested periodically for operability and functional performance.

Each active component can be individually actuated on the normal power source at any time during plant operation.

The safety injection pumps can be tested periodically during plant operation using the full flow test lines in accordance with the inservice pump and valve testing program. The residual heat removal pumps are used every time the residual heat removal loop is put into operation, as well as being periodically tested. All remote-operated valves are exercised and actuation circuits are tested during routine maintenance.

The accumulators are tested for flow during startup after a MODE 6 (Refueling) shutdown. Accumulator flow is measured when valves in the accumulator test line are opened during the test. This flow is recirculated to the refueling water storage tank (RWST).

See Section 6.3.5 for a more detailed description of current testing provisions.

3.1.1.7.11 Testing of Emergency Core Cooling System (ECCS)

CRITERION: Capability shall be provided to test periodically the operability of the Emergency Core Cooling System (ECCS) up to a location as close to the core as is practical (AIF-GDC 47).

This information is included in Section 3.1.1.7.10.

3.1.1.7.12 Testing of Operational Sequence of Emergency Core Cooling System (ECCS)

CRITERION: Capability shall be provided to test initially, under conditions as close as practical to design, the full operational sequence that would bring the Emergency Core Cooling System (ECCS) into action, including the transfer to alternate power sources (AIF-GDC 48).

The design provides for capability to test initially, to the extent practical, the full operational sequence up to the design conditions for the safety injection system to demonstrate the state of readiness and capability of the system. Details of the operational sequence testing are presented in Section 6.3.5, Tests and Inspections.

The functional test that was performed during startup is described in Section 5.4.5.5 and Section 14.6.1. (See also Section 6.3.1.4.)

3.1.1.7.13 Containment Design Basis

CRITERION: The reactor containment structure, including access openings and penetrations, and any necessary containment heat removal systems shall be designed so that the leakage of radioactive materials from the containment structure under conditions of pressure and temperature resulting from the largest credible energy release following a loss-of-coolant accident, including the calculated energy from metal-water or other chemical reactions that could occur as a consequence of failure of any single active component in the Emergency Core Cooling System (ECCS), will not result in undue risk to the health and safety of the public (AIF-GDC 49).

The following general criteria are followed to ensure conservatism in computing the required structural load capacity:

1. In calculating the containment pressure, rupture sizes up to and including a double-ended severance of reactor coolant pipes and steam lines are considered.
2. In considering postaccident pressure effects, various malfunctions of the emergency systems are evaluated consistent with the single-failure criteria.
3. The pressure and temperature loadings obtained by analyzing various accidents, when combined with operating loads and maximum wind or seismic forces, do not exceed the load-carrying capacity of the structure, its access openings, or penetrations.

Details of the containment evaluation are provided in Section 6.2.

3.1.1.7.14 Nil Ductility Transition Temperature Requirement for Containment Material

CRITERION: The selection and use of containment materials shall be in accordance with applicable engineering codes (AIF-GDC 50).

The selection and use of containment materials comply with the applicable codes and standards tabulated in Section 3.8.1.2.5.

The concrete containment is not susceptible to low-temperature brittle fracture.

The containment liner is enclosed within the containment and thus is not exposed to the outside temperature extremes. The containment ambient temperature during operation is between 50°F and 125°F which is expected to be well above the NDTT + 30°F for the liner material. Containment penetrations which can be exposed to the environment are also designed to the NDTT + 30°F criterion. The containment liner evaluation is discussed in Sections 3.8.1 and 3.8.2.

3.1.1.7.15 Reactor Coolant Pressure Boundary Outside Containment

CRITERION: If part of the reactor coolant pressure boundary is outside the containment, features shall be provided to avoid undue risk to the health and safety of the public in case of an accidental rupture in that part (AIF-GDC 51).

The reactor coolant pressure boundary does not extend outside of the containment.

3.1.1.7.16 Containment Heat Removal Systems

CRITERION: Where an active heat removal system is needed under accident conditions to prevent exceeding containment design pressure, this system shall perform its required function, assuming failure of any single active component (AIF-GDC 52).

Two means of removing heat from the containment atmosphere are provided: the containment recirculation fan cooler (CRFC)units and the containment spray system. Sections 6.2.2 and 6.5 and Chapter 15 describe the operability and capability of the containment spray system, the residual heat removal loop part of the containment heat removal system, and the containment recirculation fan cooler (CRFC) and filtration system.

3.1.1.7.17 Containment Isolation Valves

CRITERION: Penetrations that require closure for the containment function shall be protected by redundant valving and associated apparatus (AIF-GDC 53).

Isolation valves for all fluid system lines penetrating the containment provide at least two barriers for redundancy against leakage of radioactive fluids to the environment in the event of a loss-of-coolant accident. These barriers, in the form of isolation valves or closed systems, are defined on an individual line basis. In addition to satisfying containment isolation criteria, the valving is designed to facilitate normal operation and maintenance of the systems and to ensure reliable operation of other engineered safety features.

With respect to numbers and locations of isolation valves, the criteria applied are generally those outlined by the five classes described in Section 6.2.4.4.

3.1.1.7.18 Initial Leakage Rate Testing of Containment

CRITERION: Containment shall be designed so that integrated leakage rate testing can be conducted at the peak pressure calculated to result from the design-basis accident on completion and installation of all penetrations, and the leakage rate shall be measured over a sufficient period of time to verify its conformance with required performance (AIF-GDC 54).

After completion of the containment structure and installation of all penetration and weld channels, an initial integrated leakage rate test was conducted at the peak calculated accident pressure, maintained for a minimum of 24 hours, to verify that the leakage rate is not greater than 0.1% by weight of the containment volume per day.

The absolute method was used, and the test continued at a reduced pressure to provide a leak rate versus pressure characteristic curve. Weld channels and double penetrations were not pressurized during this test. Containment recirculation units operated continuously throughout the test to ensure good air mixing and temperature control.

3.1.1.7.19 Periodic Containment Leakage Rate Testing

CRITERION: The containment shall be designed so that an integrated leakage rate can be periodically determined by test during plant lifetime (AIF-GDC 55).

A leak rate test at the peak calculated accident pressure using the same method as the initial leak rate test can be performed at any time during the operational life of the plant, provided the plant is not in operation and precautions are taken to protect instruments and equipment from damage. However, in accordance with 10 CFR 50, Appendix J, subsequent containment integrated leak rate tests were conducted at reduced pressure, with appropriate compensatory modifications to the leakage acceptance criteria. See Section 6.2.6 for the latest criteria.

3.1.1.7.20 Provisions for Testing of Penetrations

CRITERION: Provisions shall be made to the extent practical for periodically testing penetrations which have resilient seals or expansion bellows to permit leak tightness to be demonstrated at the peak pressure calculated to result from occurrence of the design-basis accident (AIF-GDC 56).

A permanently piped monitoring system is provided such that all penetrations may be checked for leaktight integrity at any time throughout the operating life of the plant.

Penetrations are designed with double seals so as to permit pressurization of the interior of the penetration whenever a leak test is required. The large access openings such as the equipment hatch and personnel air locks are equipped with double seals with the space between the seals connected to the pressurizing system. The system utilizes a supply of clean, dry, compressed air which places all the penetrations under an internal pressure as required for the test.

Leakage from the system is checked by measurement of the integrated makeup air flow or change in internal pressure. In the event excessive leakage is discovered, each penetration can then be checked separately.

3.1.1.7.21 Provisions for Testing of Isolation Valves

CRITERION: Capability shall be provided to the extent practical for testing functional operability of valves and associated apparatus essential to the containment function for establishing that no failure has occurred and for determining that valve leakage does not exceed acceptable limits (AIF-GDC 57).

Capability is provided to the extent practical for testing the functional operability of valves and associated apparatus during periods of reactor shutdown. The type C tests for containment isolation valves are performed in accordance with 10 CFR 50, Appendix J. The results are documented in the Containment Integrated Leak Rate Test Report which is submitted

following the performance of each type A test. Containment leakage testing is discussed in Section 6.2.6.

3.1.1.7.22 Inspection of Containment Pressure-Reducing Systems

CRITERION: Design provisions shall be made to the extent practical to facilitate the periodic physical inspection of all important components of the containment pressure-reducing systems, such as pumps, valves, spray nozzles, torus, and sumps (AIF-GDC 58).

Design provisions are made to the extent practical to facilitate access for periodic visual inspection of all important components of the containment air recirculation and filtration and containment spray systems.

3.1.1.7.23 Testing of Containment Pressure-Reducing Systems Components

CRITERION: The containment pressure-reducing systems shall be designed to the extent practical so that components, such as pumps and valves, can be tested periodically for operability and required functional performance (AIF-GDC 59).

The containment pressure-reducing systems are designed to the extent practical so that the spray pumps, spray injection valves, spray nozzles and additive injection valves can be tested periodically and after any component maintenance action for operability and functional performance.

The air recirculating and cooling units, and the service water (SW) pumps that supply the cooling units are in operation on a relatively continuous schedule during plant operation, and no additional periodic tests are required.

3.1.1.7.24 Testing of Containment Spray Systems

CRITERION: A capability shall be provided to the extent practical to test periodically the operability of the containment spray system up to a position as close to the spray nozzles as is practical (AIF-GDC 60).

Permanent test lines for the containment spray loops are located so that all components up to the isolation valve at the spray nozzles may be tested. These isolation valves are checked separately. The spray nozzles are checked by blowing hot air (approximately 200°F) through the nozzles and observing the flow by use of thermography.

3.1.1.7.25 Testing of Operational Sequence of Containment Pressure-Reducing Systems

CRITERION: A capability shall be provided to test initially under conditions as close as practical to the design and the full operational sequence that would bring the containment pressure-reducing systems into action, including the transfer to alternate power sources (AIF-GDC 61).

Capability is provided to test initially to the extent practical the operational startup sequence beginning with transfer to alternate power sources and ending with near design conditions for

the containment spray and containment recirculation fan cooler (CRFC) and filtration systems.

3.1.1.7.26 Inspection of Air Cleanup Systems

CRITERION: Design provisions shall be made to the extent practical to facilitate physical inspection of all critical parts of containment air cleanup systems, such as, ducts, filters, fans, and dampers (AIF-GDC 62).

Access is available for visual inspection of the containment fan cooler and recirculation filtration components.

3.1.1.7.27 Testing of Air Cleanup Systems Components

CRITERION: Design provisions shall be made to the extent practical so that active components of the air cleanup systems, such as fans and dampers, can be tested periodically for operability and required functional performance (AIF-GDC 63).

Periodic tests of the dampers associated with the charcoal filter units of the containment air cleanup system are conducted. Each damper is stroked and its operation (including stroke time) is checked by personnel in the containment. An indicating light in the control room provides indication of damper movement. Periodic tests also verify that the dampers fail in a safe position upon loss of air, and that air flow and orientation for accident operation is acceptable.

3.1.1.7.28 Testing Air Cleanup System

CRITERION: A capability shall be provided to the extent practical for on site periodic testing and surveillance of the air cleanup systems to ensure (a) filter bypass paths have not developed and (b) filter and trapping materials have not deteriorated beyond acceptable limits (AIF-GDC 64).

Each containment recirculation fan unit is checked periodically for water in the filtration area. Also, charcoal filters are tested for bypass flow and pressure drop, and are visually inspected for damage and loss of charcoal. Further, a representative sample frame is removed during shutdown and tested periodically to verify its continued efficiency. After reinstallation the filter units are tested in place by aerosol injection to determine integrity of the flow path.

3.1.1.7.29 Testing of Operational Sequence of Air Cleanup Systems

CRITERION: Capability shall be provided to test initially under conditions as close to design as practical, the full operational sequence that would bring the air cleanup systems into action, including the transfer to alternate power sources and the design air flow delivery capability (AIF-GDC 65).

Means are provided to test initially under conditions as close to design and as near as is practical the full operational sequence that would bring the containment recirculation fan cooler (CRFC) and filtration system into action, including transfer to the emergency diesel-generator power source.

3.1.1.8 Fuel and Waste Storage Systems

3.1.1.8.1 Prevention of Fuel Storage Criticality

CRITERION: Criticality in new and spent fuel storage shall be prevented by physical systems or processes. Such means as geometrically safe configurations shall be emphasized over procedural controls (AIF-GDC 66).

During reactor vessel head removal and while loading and unloading fuel from the reactor, the boron concentration is maintained at not less than that required to shutdown the core to a $k_{EFF} = 0.90$. This shutdown margin maintains the core at k_{EFF} less than 0.99, even if all control rods are withdrawn from the core. Weekly checks of refueling water boron concentration ensure the proper shutdown margin.

The new and spent fuel storage racks are designed so that it is impossible to insert assemblies in other than the prescribed locations. Borated water is used to fill the spent fuel storage pool at a concentration to match that used in the reactor cavity and refueling canal during refueling operations. The fuel is stored vertically in an array with sufficient center-to-center distance between assemblies to ensure k_{EFF} less than or equal to 0.90 even if unborated water were used to fill the pool.

Detailed instructions are available for use by trained refueling personnel. Furthermore, interlocks are provided to limit the travel of heavy loads in areas where failure could result in unacceptable consequences.

Since initial criticality, changes have been made. Clarifications include:

1. Boron concentration ensures that k_{EFF} is maintained less than or equal to 0.95, vs. 0.90.
2. Checks of refueling water boron concentration are periodically conducted per the requirements of the Technical Specifications and Technical Requirements Manual, and not necessarily "weekly".
3. It is not impossible to insert assemblies into incorrect locations. Therefore, administrative controls have been established to ensure that assemblies are inserted into the proper locations.
4. The criticality methodology (Section 9.1.2.4) assumes a limited credit for borated water. The water can no longer be unborated, and this limited credit for borated water ensures that there are safe margins to an inadvertent criticality.

3.1.1.8.2 Fuel and Waste Storage Decay Heat

CRITERION: Reliable decay heat removal systems shall be designed to prevent damage to the fuel in storage facilities and to waste storage tanks that could result in radioactivity release which would result in undue risk to the health and safety of the public (AIF-GDC 67).

The refueling water provides a reliable and adequate cooling medium for spent fuel transfer. Heat removal is provided by auxiliary cooling systems, such as the spent fuel pool (SFP) cooling system (Section 9.1.2) and the service water (SW) system (Section 9.2.1).

3.1.1.8.3 Fuel and Waste Storage Radiation Shielding

CRITERION: Adequate shielding for radiation protection shall be provided in the design of spent fuel and waste storage facilities (AIF-GDC 68).

Adequate shielding for radiation protection is provided during refueling operations by conducting all spent fuel transfer and storage operations under water. This permits visual control of the operation at all times while maintaining low radiation levels. Shielding is provided for waste handling and storage facilities to permit operation within regulatory guidelines.

Gamma radiation is continuously monitored in the auxiliary building. A high level signal is alarmed locally and is annunciated in the control room.

Shielding for the waste disposal system and its storage components is designed to limit the dose rates as required by personnel access, testing, operation, and maintenance requirements.

3.1.1.8.4 Protection Against Radioactivity Release From Spent Fuel and Waste Storage

CRITERION: Provisions shall be made in the design of fuel and waste storage facilities such that no undue risk to the health and safety of the public could result from an accidental release of radioactivity (AIF-GDC 69).

The reactor cavity, refueling canal and spent fuel storage pool are reinforced concrete structures with a seam-welded stainless steel plate liner. These structures are designed to withstand the anticipated earthquake loadings as Seismic Category I structures so that the liner should prevent leakage even in the event the reinforced concrete develops cracks. Accident analyses described in Chapter 15 demonstrate that the postulated accidents result in exposures well within regulatory guidelines.

3.1.1.9 Control of Releases of Radioactivity to the Environment

CRITERION: The facility design shall include those means necessary to maintain control over the plant radioactivity effluents, whether gaseous, liquid, or solid. Appropriate holdup capacity shall be provided for retention of gaseous, liquid, or solid effluents, particularly where unfavorable environmental conditions can be expected to require operational limitations upon the release of radioactive effluents to the environment. In all cases, the design for radioactivity control must be justified
(a) on the basis of 10 CFR 20 requirements, for normal operations and for any transient situation that might reasonably be anticipated to occur and (b) on the basis of 10 CFR 100 dosage level guidelines for potential reactor accidents of exceedingly low probability of occurrence (AIF-GDC 70).

Liquid, gaseous, and solid waste disposal facilities are designed so that discharge of effluents and offsite shipments are in accordance with applicable NRC regulations and guidelines.

Radioactive fluids entering the waste disposal system are collected in sumps and tanks until determination of subsequent treatment can be made. They are sampled and analyzed to determine the quantity of radioactivity, with an isotopic breakdown if necessary. Before any attempt is made to discharge, they are processed as required and then released under controlled conditions. The system design and operation are characteristically directed toward minimizing releases to unrestricted areas. Discharge streams are appropriately monitored and safety features are incorporated to preclude excessive releases, in accordance with the Offsite Dose Calculation Manual (ODCM).

The bulk of the radioactive liquids discharged from the reactor coolant system are processed and retained inside the plant by the chemical and volume control system recycle train. This minimizes liquid input to the waste disposal system which processes relatively small quantities of generally low-activity level wastes. The processed water from waste disposal, from which most of the radioactive material has been removed, is discharged through a monitored line into the circulating water discharge.

Radioactive gases are pumped by compressors through a manifold to one of the gas decay tanks where they are held a suitable period of time for decay. Cover gases in the nitrogen blanketing system are reused to minimize gaseous wastes. During MODES 1 and 2, gases are discharged intermittently at a controlled rate from these tanks through the monitored plant vent. The system is provided with discharge controls so that environmental conditions do not restrict the release of radioactive effluents to the atmosphere.

Liquid wastes are processed to remove most of the radioactive materials. The spent resins from the demineralizers, the filter cartridges, and the concentrates from the evaporators are packaged and stored onsite until shipment offsite for disposal. Suitable containers are used to package these solids at the highest practical concentrations to minimize the number of containers shipped for burial.

All solid waste is placed in suitable containers and stored onsite until shipment offsite is made for disposal.

3.1.2 GENERAL DESIGN CRITERIA

General Design Criteria (GDC) are set forth in Appendix A of 10 CFR 50. The Ginna Station conformance to the 1972 version of the GDC is described in the following sections.

3.1.2.1 Overall Requirements

These criteria are intended to ensure that the quality control and quality assurance programs are identified, recorded, and justified in terms of their adequacy. The five criteria of this group are intended to apply to the design, fabrication, erection, and performance requirements of the facility’s essential components and systems to ensure that there is protection against natural phenomena and environmental conditions. In addition, these criteria are also intended to provide fire and explosion protection for all equipment important to safety.

3.1.2.1.1 General Design Criterion 1 Quality Standards and Records

CRITERION: Structures, systems, and components important to safety shall be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. Where generally recognized codes and standards are used, they shall be identified and evaluated to determine their applicability, adequacy, and sufficiency and shall be supplemented or modified as necessary to assure a quality product in keeping with the required safety function. A quality assurance program shall be established and implemented in order to provide adequate assurance that these structures, systems, and components will satisfactorily perform their safety functions. Appropriate records of the design, fabrication, erection, and testing of structures, systems, and components important to safety shall be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit (GDC 1).

All systems and components of the facility were classified according to their importance. Those items vital to safe shutdown and isolation of the reactor or whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity were designated Class I. Those items important to reactor operation but not essential to safe shutdown and isolation of the reactor or control of the release of substantial amounts of radioactivity were designated Class II. Those items not related to reactor operation or safety were designated Class III. Note that Ginna LLC no longer uses this classification scheme. The classification of structures and equipment is discussed in Section 3.2.

Safety-related structures, systems, and components are essential to the protection of the health and safety of the public. Consequently, they were designed, fabricated, inspected and erected, and the materials selected to the applicable provisions of the then recognized codes, good nuclear practice, and to quality standards that reflected their importance. Discussions of applicable codes and standards, quality assurance programs, test provisions, etc., that were used are given in the section describing each system.

A complete set of as-built facility plant and system diagrams are maintained throughout the life of the reactor. Records of modifications to the general arrangement and structural plans are also maintained throughout the life of the reactor.

A set of completed test procedures for all initial plant testing is maintained as outlined in Chapter 14.

A set of all the quality assurance data generated during fabrication and erection of the essential components of the plant, as defined by the Ginna Station construction quality assurance program, is retained. The quality control and quality assurance program for Ginna Station construction is described in Section 17.1. The current quality assurance program for Ginna Station is referenced in Section 17.2.

3.1.2.1.2 General Design Criterion 2 - Design Bases for Protection Against Natural Phenomena

CRITERION: Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these structures, systems, and components shall reflect: (1) Appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena and (3) the importance of the safety functions to be performed (GDC 2).

All systems and components designated Seismic Category I are designed so that there is no loss of function in the event of the safe shutdown earthquake. Measures were also taken in the plant design to protect against high winds, sudden barometric pressure changes, seiches, and other natural phenomena. Tornado and flood protection measures are discussed in Sections 3.3 and 3.4. Procedures have been written that will be followed in the event of such natural phenomena. The occurrence of such phenomena is discussed in Chapter 2.

On May 22, 1992, Generic Letter 87-02, Supplement 1, transmitted Supplemental Safety Evaluation Report No. 2 (SSER No. 2) on the Seismic Qualification Utility Group (SQUG) Generic Implementation Procedure, Revision 2, dated February 14, 1992 (GIP-2). Supplemental Safety Evaluation Report No. 2 approved the methodology in the Generic Implementation Procedure for use in verification of equipment seismic adequacy including equipment involved in future modifications and replacement equipment. In letters dated November 30, 1992, and June 8, 1993, the NRC accepted RG&E's response to Generic Letter 87-02, Supplement 1.

3.1.2.1.3 General Design Criterion 3 - Fire Protection

CRITERION: Structures, systems, and components important to safety shall be designed and located to minimize, consistent with other safety requirements, the probability and effect of fires and explosions. Noncombustible and heat resistant materials shall be used wherever practical throughout the unit, particularly in locations such as the containment and control room. Fire detection and fighting systems of appropriate capacity and capability shall be provided and designed to minimize the adverse effects of fires on structures, systems, and components important to safety. Fire-fighting systems shall be designed to assure that their rupture or inadvertent operation does not significantly impair the safety capability of these structures, systems, and components (GDC 3).

Fire detection and fighting systems of appropriate capacity and capability are provided to minimize the adverse effects of fire on structures, systems, and components important to safety. Sensing devices include both ionization chambers (smoke detectors) and temperature detectors. Fire-fighting equipment includes automatic water suppression in appropriate areas.

Automatically initiated Halon 1301 total flooding systems are provided in the relay room and computer room. Appropriate hoses and portable fire-fighting equipment are placed throughout the plant. The fire protection system and compliance with 10 CFR 50, Appendix R, are discussed in Section 9.5.1.

3.1.2.1.4 General Design Criterion 4 - Environmental and Missile Design Bases

CRITERION: Structures, systems, and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These structures, systems, and components shall be appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit (GDC 4).

A comprehensive review has been performed to ensure proper environmental qualification of safety-related electrical equipment, in accordance with 10 CFR 50.49. This is discussed in detail in Section 3.11. Also, a review of postulated pipe breaks inside and outside containment was conducted as part of the Systematic Evaluation Program (SEP) including dynamic effects such as pipe whip and jet impingement. This is discussed in Section 3.6. Finally, internally generated missiles, tornado missiles, and site proximity missiles, including aircraft, were reviewed as part of the SEP and are discussed in Section 3.5.

3.1.2.1.5 General Design Criterion 5 - Sharing of Structures, Systems, and Components

CRITERION: Structures, systems, and components important to safety shall not be shared among nuclear power units unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions, including, in the event of an accident in one unit, an orderly shutdown and cooldown of the remaining units (GDC 5).

The R. E. Ginna Nuclear Power Plant is a single unit installation.

3.1.2.2 Protection by Multiple Fission Product Barriers

These criteria are intended to ensure that designs provide the reactor unit with multiple barriers which remain intact during MODES 1 and 2 and all anticipated transients and that adequate barriers are available for design-basis accidents. In addition, these criteria are intended to identify and define the instrumentation and control systems, electrical power systems, and control room requirements required for MODES 1 and 2, anticipated operational occurrences, and for accident condition.

3.1.2.2.1 General Design Criterion 10 - Reactor Design

CRITERION: The reactor core and associated coolant, control, and protection systems shall be designed with appropriate margin to assure that specified acceptable fuel design limits are not exceeded during any condition of normal operation, including the effects of anticipated operational occurrences (GDC 10).

The reactor core design, in combination with coolant, control, and protection systems, provides margins to ensure that fuel is not damaged during MODES 1 and 2 or as a result of anticipated operational transients.

The DNB correlations have been used to predict the DNB flux and location of DNB for axially uniform and nonuniform heat flux distributions. For operation within the Technical Specification limits, the DNBR during steady-state operation and anticipated transients is limited to specific safety values.

The reactor control and protective system also prevents the power level or system temperature or pressure from exceeding limits that would result in a DNBR of less than the limiting values for anticipated transients (see Chapter 4).

3.1.2.2.2 General Design Criterion 11 - Reactor Inherent Protection

CRITERION: The reactor core and associated coolant systems shall be designed so that in the power operating range the net effect of the prompt inherent nuclear feedback characteristics tends to compensate for a rapid increase in reactivity (GDC 11).

The reactor core and associated coolant systems have been designed so that in the power operating range the net effect of the prompt nuclear feedback characteristics tends to compensate for a rapid increase in reactivity.

The moderator temperature coefficient is usually, though not always, negative. The moderator pressure and density coefficients are not usually negative; however, the overall power coefficient (due to the doppler coefficient) is negative and so provides a nuclear feedback characteristic to limit a rapid increase in reactivity.

3.1.2.2.3 General Design Criterion 12 - Suppression of Reactor Power Oscillations

CRITERION: The reactor core and associated coolant, control, and protection systems shall be designed to assure that power oscillations which can result in conditions exceeding specified acceptable fuel design limits are not possible or can be reliably and readily detected and suppressed (GDC 12).

The reactor core and the associated coolant, control, and protection systems, and operating strategies have been designed to prevent or easily suppress power oscillations that could result in exceeding fuel design limits.

3.1.2.2.4 General Design Criterion 13 - Instrumentation and Control

CRITERION: Instrumentation shall be provided to monitor variables and systems over their anticipated ranges for normal operation, for anticipated operational occurrences, and for accident conditions as appropriate to assure adequate safety, including those variables and systems that can affect the fission process, the integrity of the reactor core, the reactor coolant pressure boundary, and the containment and its associated systems. Appropriate controls shall be provided to maintain these variables and systems within prescribed operating ranges (GDC 13).

Instrumentation and controls essential to avoid undue risk to the health and safety of the public are provided to monitor and maintain containment pressure, neutron flux, primary coolant pressure, flow rate, temperature, and control rod positions within prescribed operating ranges.

The fission process is monitored and controlled for all conditions from the source range through the power range. The neutron monitoring system detects core conditions that could potentially threaten the overall integrity of the fuel barrier due to excess power generation and provides a corresponding signal to the Reactor Trip System (RTS). In addition to the ex-core neutron monitoring system, movable in-core instrumentation provides the capability of mapping the core.

The nonnuclear regulating, process, and containment instrumentation measures temperatures, pressure, flow, and levels in the reactor coolant system, steam systems, containment and other auxiliary systems. Process variables required on a continuous basis for the startup, operation, and shutdown of the plant are indicated, recorded, and controlled from the control room. The quantity and types of process instrumentation provided ensures safe and orderly operation of all systems and processes over the full operating range of the plant.

The instrumentation and control systems are discussed in Chapter 7.

3.1.2.2.5 General Design Criterion 14 - Reactor Coolant Pressure Boundary

CRITERION: The reactor coolant pressure boundary shall be designed, fabricated, erected, and tested so as to have an extremely low probability of abnormal leakage, of rapidly propagating failure, and of gross rupture (GDC 14).

All piping components and supporting structures of the reactor coolant system were designed as Class I and later reevaluated as Seismic Category I equipment as defined in Section 3.7. All pressure containing components of the reactor coolant system were designed, fabricated, inspected, and tested in conformance with the code requirements listed in Table 5.2-1. Therefore, the probability of abnormal leakage, of rapidly propagating failure and of gross rupture is very low.

3.1.2.2.6 General Design Criterion 15 - Reactor Coolant System Design

CRITERION: The reactor coolant system and associated auxiliary, control, and protection systems shall be designed with sufficient margin to assure that the design conditions of the reactor coolant pressure boundary are not exceeded during any condition of normal operation, including anticipated operational occurrences (GDC 15).

The reactor coolant system and associated auxiliary, control, and protection systems were designed with sufficient margins so that design conditions are not exceeded during MODES 1 and 2 including anticipated operational occurrences. The normal operating pressure is 2235 psig with design pressure being 2485 psig. This provides a reasonable range for maneuvering during operation with allowance for pressure transients without actuation of the safety valves. The analysis presented in Chapter 15 demonstrates the ability of the plant to safely undergo all anticipated transients with pressure peaks below 2485 psig.

Overpressurization is prevented by a combination of automatic control and pressure relief devices. The pressurizer safety valves (2485 psig setpoint) and pressurizer power operated relief valves (2335 psig setpoint) prevent overpressuring the reactor coolant system (RCS) during operation at rated power. Cold overpressure protection of the RCS is provided by the pressurizer power operated relief valves (PORV). The PORV lift setting is switched to Low Temperature Overpressure Protection (LTOP) control (lift setting 410 psig) prior to reducing RCS temperature below 330°F or placing the residual heat removal system in service.

3.1.2.2.7 General Design Criterion 16 - Containment Design

CRITERION: Reactor containment and associated systems shall be provided to establish an essentially leaktight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require (GDC 16).

The building containing the reactor and primary system is a reinforced-concrete structure prestressed in the vertical direction, with a welded steel liner on the inside. The structure contains a free volume of approximately 1,000,000 ft^3 and is designed for an internal pressure of 60 psig. Prior to initial operation, the containment was strength tested at 69 psig and then was leak tested. The acceptance criterion for the preoperational leakage test was established as 0.1% per 24 hours at 60 psig.

Reports on the Structural Integrity Test of Reactor Containment Structure and Pre-operational Integrated Leak Rate Test of the Reactor Containment Building were submitted to the AEC. The leakage rate at 60 psig was determined to be 0.0219 ± .0168% per 24 hours.

Periodic leak rate measurements as defined in the Technical Specifications ensure that the containment structure provides an essentially leaktight barrier against the uncontrolled release of radioactivity to the environment. Periodic inspection of prestressed tendons as well as periodic integrated leak rate tests, as defined in the Technical Specifications, ensure the continued structural integrity of the containment structure.

A containment spray system and fan coolers are provided to mitigate the consequences of a loss-of-coolant accident. More details on the containment system can be found in Sections 6.2 and 3.8.

3.1.2.2.8 General Design Criterion 17 - Electrical Power Systems

CRITERION: An onsite electric power system and an offsite electric power system shall be provided to permit functioning of structures, systems, and components important to safety. The safety function for each system (assuming the other system is not functioning) shall be to provide sufficient capacity and capability to assure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents.

> The onsite electric power supplies, including the batteries, and the onsite electric distribution system, shall have sufficient independence, redundancy, and testability to perform their safety functions assuming a single failure.
>
> Electric power from the transmission network to the onsite electric distribution system shall be supplied by two physically independent circuits (not necessarily on separate rights of way) designed and located so as to minimize to the extent practical the likelihood of their simultaneous failure under operating and postulated accident and environmental conditions. A switchyard common to both circuits is acceptable. Each of these circuits shall be designed to be available in sufficient time following a loss of all onsite alternating current power supplies and the other offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded. One of these circuits shall be designed to be available within a few seconds following a loss-of-coolant accident to assure that core cooling, containment integrity, and other vital safety functions are maintained.
>
> Provisions shall be included to minimize the probability of losing electric power from any of the remaining supplies as a result of, or coincident with, the loss of power generated by the nuclear power unit, the loss of power from the transmission network, or the loss of power from the onsite electric power supplies (GDC 17).

Onsite and offsite electrical power systems are provided to permit functioning of structures, systems, and components important to safety. Each system provides sufficient capacity and capability to ensure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences, and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents.

Two completely independent and redundant emergency diesel-generator systems are provided as well as two completely separate and independent station battery systems.

Offsite power is supplied by two separate sources. One source comes from the 115-kV system through a 115-kV to 34.5-kV step-down transformer and station auxiliary (startup) transformer 12A and the second from the 115-kV system through a 115-kV to 34.5-kV step-down transformer and station auxiliary (startup) transformer 12B. The station auxiliary transformers (12A and 12B) are the normal offsite power sources to the safeguards buses. In the event of a failure of both station auxiliary transformers, the unit auxiliary transformer (11) can be used as a backup supply. This transformer can be used by disconnecting a flexible connection on the isolated phase bus at the generator terminals and backfeeding from the 115-kV system through the main transformer.

Diesels and batteries are tested according to the requirements of the Technical Specifications. Both the onsite and offsite power systems would be available following a loss-of-coolant accident in time to ensure that core cooling, containment integrity, and other vital safety functions are maintained. More detailed information on the electrical systems can be found in Chapter 8.

3.1.2.2.9 General Design Criterion 18 - Inspection and Testing of Electrical Power Systems

CRITERION: Electric power systems important to safety shall be designed to permit appropriate periodic inspection and testing of important areas and features, such as wiring, insulation, connections, and switchboards, to assess the continuity of the systems and the condition of their components. The systems shall be designed with a capability to test periodically (1) the operability and functional performance of the components of the systems, such as onsite power sources, relays, switches, and buses, and (2) the operability of the systems as a whole and, under conditions as close to design as practical, the full operation sequence that brings the systems into operation, including operation of applicable portions of the protection system, and the transfer of power among the nuclear power unit, the offsite power system, and the onsite power system (GDC 18).

The electrical power systems are designed with the capability of periodic testing for operability. Components of the systems, i.e., onsite power sources, relays, and switches, are similarly capable of being periodically tested. Passive components such as wiring, connections, switchboards, and buses are capable of periodic inspection.

Verification of operability of the systems as a whole, including transfer of power, is described in Chapter 8. Operability of the systems in accordance with design conditions was verified by preoperational testing and periodic testing of the systems is required by the Technical Specifications.

3.1.2.2.10 General Design Criterion 19 - Control Room

CRITERION: A control room shall be provided from which actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain it in a safe condition under accident conditions, including loss-of-coolant accidents. Adequate radiation protection shall be provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent to any part of the body, for the duration of the accident. Equipment at appropriate locations outside the control room shall be provided (1) with a design capability for prompt hot shutdown of the reactor, including necessary instrumentation and controls to maintain the unit in a safe condition during hot shutdown, and (2) with a potential capability for subsequent cold shutdown of the reactor through the use of suitable procedures (GDC 19).

The station is equipped with a control room which contains controls and instrumentation as necessary for operation of the reactor and turbine generator under normal and accident conditions.

The control room is capable of continuous occupancy by the operating personnel under all operating and accident conditions, within specified dose limits. See Section 6.4.

Although the likelihood of conditions which could render the main control room inaccessible even for a short time is extremely small, provisions have been made so that plant operators can shut down and maintain the plant in a safe condition by means of controls located outside the control room. During such a period of control room inaccessibility, the reactor will be tripped and the plant maintained in a safe shutdown condition. This is described in Section 7.4.3.

3.1.2.3 Protection and Reactivity Control Systems

These criteria are intended to identify and establish requirements for functional reliability, inservice testability, redundancy, physical and electrical independence and separation, and fail-safe design of the systems that are essential to the reactor protection functions. In addition, these criteria are intended to establish (1) the reactor core reactivity insertion rate limit and (2) the means of control of the reactor within these limits.

3.1.2.3.1 General Design Criterion 20 - Protection Systems Functions

CRITERION: The protection system shall be designed (1) to initiate automatically the operation of appropriate systems including the reactivity control systems, to assure that specified acceptable fuel design limits are not exceeded as a result of anticipated operational occurrences and (2) to sense accident conditions and to initiate the operation of systems and components important to safety (GDC 20).

A plant protection system, as described in Section 7.2 is provided to automatically initiate appropriate action whenever specific plant conditions reach preestablished limits. These limits ensure that specified fuel design limits are not exceeded when anticipated operational occurrences happen. In addition, other protective instrumentation is provided to initiate actions which mitigate the consequences of an accident. The Ginna Station installation meets the requirements of Criterion 20.

3.1.2.3.2 General Design Criterion 21 - Protection System Reliability and Testability

CRITERION: The protection system shall be designed for high functional reliability and inservice testability commensurate with the safety functions to be performed. Redundancy and independence designed into the protection system shall be sufficient to assure that (1) no single failure results in loss of the protection function and (2) removal from service of any component or channel does not result in loss of the required minimum redundancy unless the acceptable reliability of operation of the protection system can be otherwise demonstrated. The protection system shall be designed to permit periodic testing of its functioning when the reactor is in operation, including a capability to test channels including a capability to test channels independently to determine failures and losses of redundancy that may have occurred (GDC 21).

Sufficient redundancy and independence are designed into the Reactor Trip System (RTS) to ensure that no single failure results in loss of protection function. The system is designed such that it will accommodate any single component failure and still perform its protective function.

Reliability and independence is obtained by redundancy within each tripping function. In a two-out-of-three circuit, for example, the three channels are equipped with separate primary sensors. Each channel is continuously fed from its own independent electrical sources. Failure to deenergize a channel when required would be a mode of malfunction that would affect only that channel. The trip signal furnished by the two remaining channels would be unimpaired in this event.

All reactor protection channels are supplied with sufficient redundancy to provide the capability for channel calibration and test at power. Bypass removal of one trip circuit is accomplished by placing that circuit in a half-tripped mode; i.e., a two-out-of-three circuit becomes a one-out-of-two circuit. Testing does not trip the system unless a trip condition exists in a concurrent channel.

Detailed information verifying compliance with this criterion is in Section 7.2 and in the Technical Specifications.

3.1.2.3.3 General Design Criterion 22 - Protection System Independence

CRITERION: The protection system shall be designed to assure that the effects of natural phenomena, and of normal operating, maintenance, testing, and postulated accident conditions on redundant channels do not result in loss of the protection function, or shall be demonstrated to be acceptable on some other defined basis. Design techniques, such as functional diversity or diversity in component design and principles of operation, shall be used to the extent practical to prevent loss of the protection function (GDC 22).

The Ginna Station protection system was designed so that the effects of natural phenomena and of normal operating, maintenance, testing, and postulated accident conditions do not result in the loss of the protective function. The design includes the techniques of functional diversity or diversity in components design and principles of operation to the extent practical in preventing the loss of the protection functions. Specific information about system independence is covered in Section 7.2.2.

3.1.2.3.4 General Design Criterion 23 - Protection System Failure Modes

CRITERION: The protection system shall be designed to fall into a safe state or into a state demonstrated to be acceptable on some other defined basis if conditions such as disconnection of the system, loss of energy (e.g., electric power, instrument air), or postulated adverse environments (e.g., extreme heat or cold, fire, pressure, steam, water, and radiation) are experienced (GDC 23).

The Reactor Trip System (RTS) is designed to fail-safe upon loss of power. Each reactor trip circuit is designed so that trip occurs when the circuit is deenergized; an open circuit or loss of channel power, therefore, causes the system to go into its trip mode. In a two-out-of-three circuit, the three channels are equipped with separate primary sensors and each channel is energized from independent electrical buses. Failure to deenergize when required is a mode of malfunction that affects only one channel. The trip signal furnished by the two remaining channels is unimpaired in this event.

Reactor trip is implemented by interrupting power to the magnetic latch mechanisms on each drive, allowing the rod clusters to insert by gravity. The protection system is thus inherently safe in the event of a loss of power. Automatic starting of either emergency diesel generator is initiated by redundant undervoltage relays on the 480-V safeguards bus with which the diesel generator is associated, or by the safety injection signal. Engine cranking is accomplished by a stored energy system supplied solely for the associated diesel generator. The undervoltage relay scheme is designed so that loss of 480-V power does not prevent the relay scheme from functioning properly.

Environmental and seismic qualification requirements are met as required for specified protection system equipment.

Chapters 7 and 8 discuss compliance with this criterion.

3.1.2.3.5 General Design Criterion 24 - Separation of Protection and Control Systems

CRITERION: The protection system shall be separated from control systems to the extent that failure of any single control system component or channel, or failure or removal from service of any single protection system component or channel which is common to the control and protection systems leaves intact a system satisfying all reliability, redundancy, and independence requirements of the protection system. Interconnection of the protection and control systems shall be limited so as to assure that safety is not significantly impaired (GDC 24).

The Reactor Trip System (RTS) is physically and electrically separate from the control systems such that failure of any single control component or channel, or removal from service, leaves the system satisfying the reliability, redundancy, and independence requirements of the Reactor Trip System (RTS). Information supporting compliance with this criterion is in Section 7.2.5.

3.1.2.3.6 General Design Criterion 25 - Protection System Requirements for Reactivity Control Malfunctions

CRITERION: The protection system shall be designed to assure that specified acceptable fuel design limits are not exceeded for any single malfunction of the reactivity control systems, such as accidental withdrawal (not ejection or dropout) of control rods (GDC 25).

The Reactor Trip System (RTS) is designed to ensure that the specified fuel design limits are not exceeded for any single malfunction of the reactivity control systems. Reactor shutdown with rods is completely independent of the normal control functions. The trip breakers interrupt the power to the rod mechanisms to trip the reactor regardless of existing control signals.

Details of the effects of continuous withdrawal of a control rod assembly and of continuous deboration are discussed in Sections 15.4.1 and 15.4.4.

3.1.2.3.7 General Design Criterion 26 - Reactivity Control System Redundancy and Capability

CRITERION: Two independent reactivity control systems of different design principles shall be provided. One of the systems shall use control rods, preferably including a positive means for inserting the rods, and shall be capable of reliably controlling reactivity changes to assure that under conditions of normal operation, including anticipated operational occurrences, and with appropriate margin for malfunctions such as stuck rods, specified acceptable fuel design limits are not exceeded. The second reactivity control system shall be capable of reliably controlling the rate of reactivity changes resulting from planned, normal power changes (including xenon burnout) to assure acceptable fuel design limits are not exceeded. One of the systems shall be capable of holding the reactor core subcritical under cold conditions (GDC 26).

One of the two reactivity control systems employs control rod drive mechanisms to regulate the position of silver-indium-cadmium neutron absorbers within the reactor core. The control rods are designed to shut down the reactor with adequate margin for all anticipated occurrences so that fuel design limits are not exceeded. The other reactivity control system employs the chemical and volume control system to regulate the concentration of boric acid neutron absorber in the reactor coolant system. The chemical and volume control system is capable of controlling the reactivity change resulting from planned normal power changes. Reactivity control system redundancy and capability are discussed in detail in Sections 4.3 and 9.3.4.

3.1.2.3.8 General Design Criterion 27 - Combined Reactivity Control System Capability

CRITERION: The reactivity control systems shall be designed to have a combined capability, in conjunction with poison addition by the Emergency Core Cooling System (ECCS), of reliably controlling reactivity changes to assure that under postulated accident conditions and with appropriate margin for stuck rods the capability to cool the core is maintained (GDC 27).

The reactivity control systems in conjunction with boron addition through the Emergency Core Cooling System (ECCS) has the capability of controlling reactivity changes under postulated accident conditions with appropriate margins for stuck rods.

Ginna Station is provided with the means of making and holding the core subcritical under any anticipated conditions and with appropriate margin for contingencies. Combined use of the rod cluster control system and the chemical shim control system permit the necessary shutdown margin to be maintained during long-term xenon decay and plant cooldown, even with the single highest worth control rod stuck out.

In a loss-of-coolant accident the safety injection system is actuated and concentrated boric acid is injected into the cold legs of the reactor coolant system. This is in addition to the boric acid content of the accumulators which is passively injected on a decrease in system pressure. See Section 6.3 and Section 4.2.1 for further details.

3.1.2.3.9 General Design Criterion 28 - Reactivity Limits

CRITERION: The reactivity control systems shall be designed with appropriate limits on the potential amount and rate of reactivity increase to assure that the effects of postulated reactivity accidents can neither (1) result in damage to the reactor coolant pressure boundary greater than limited local yielding nor (2) sufficiently disturb the core, its support structures or other reactor pressure vessel internals to impair significantly the capability to cool the core. These postulated reactivity accidents shall include consideration of rod ejection (unless prevented by positive means), rod dropout, steam line rupture, changes in reactor coolant temperature and pressure, and cold water addition (GDC 28).

The maximum reactivity worth of control rods and the maximum rates of reactivity insertion employing control rods are limited by the design of the facility to values which prevent failure of the coolant pressure boundary or disruptions of the core or vessel internals to a degree which could impair the effectiveness of emergency core cooling. Section 4.2.1 discusses the design basis in meeting this criterion, and Chapter 15 discusses the accident analyses and the relationship of the reactivity insertion rates to plant safety. The Core Operating Limits Report (COLR) includes appropriate graphs showing the maximum permissible insertion limits and overlap of rod cluster control assembly banks as a function of power.

3.1.2.3.10 General Design Criterion 29 - Protection Against Anticipated Operational Occurrences

CRITERION: The protection and reactivity control systems shall be designed to assure an extremely high probability of accomplishing their safety functions in the event of anticipated operational occurrences (GDC 29).

The protection and reactivity control systems are designed to ensure extremely high reliability in regard to their required safety functions in any anticipated operational occurrences. Anticipated failure modes of system components are designed to be safe modes. Equipment used in these systems is designed, constructed, operated, and maintained with a high level of reliability. Loss of power to the protection system will result in a reactor trip.

3.1.2.4 Fluid Systems

These criteria are intended to (1) identify those nuclear safety systems within the general category of fluid systems, (2) examine each one for capability, redundancy, testability, and inspectability, and (3) ensure that each safety feature capability encompasses all the anticipated and credible phenomena associated with the operational transients or design-basis accidents. In addition, these criteria are intended to establish the design requirements for the reactor coolant pressure boundary and to identify the means for satisfying these design requirements.

3.1.2.4.1 General Design Criterion 30 - Quality of Reactor Coolant Pressure Boundary

CRITERION: Components which are part of the reactor coolant pressure boundary shall be designed, fabricated, erected, and tested to the highest quality standards

practical. Means shall be provided for detecting and, to the extent practical, identifying the location of the source of reactor coolant leakage (GDC 30).

Quality standards of material selection, design, fabrication, and inspection for the Ginna reactor coolant system conformed to the applicable provisions of recognized codes and good nuclear practice of that period. Details of the quality assurance programs, test procedures, and inspection acceptance levels are given in Section 17.1. Particular emphasis was placed on the assurance of quality of the reactor vessel to obtain material whose properties are uniformly within tolerances appropriate to the application of the design methods of the code used. Table 3.2-1 gives the code requirements used for the reactor coolant system.

Leakage detection systems are described in Section 5.2.5.

3.1.2.4.2 General Design Criterion 31 - Fracture Prevention of Reactor Coolant Pressure Boundary

CRITERION: The reactor coolant pressure boundary shall be designed with sufficient margin to assure that when stressed under operating, maintenance, testing, and postulated accident conditions (1) the boundary behaves in a nonbrittle manner and
(2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the boundary material under operating, maintenance, testing, and postulated accident conditions and the uncertainties in determining (1) material properties, (2) the effects of irradiation on material properties, (3) residual, steady-state and transient stresses, and (4) size of flaws (GDC 31).

The reactor coolant pressure boundary was fabricated, inspected and tested in accordance with codes (i.e., ASME Boiler and Pressure Vessel Code and the ASA Code for Pressure Piping) that were applicable at the time of fabrication and installation. An evaluation of the Ginna reactor vessel concluded that the Ginna vessel met the ASME, Section III, fracture toughness requirements (see Section 5.3.1.2).

A maximum initial NDTT for the vessel shell material was established as 40°F. Curves for heatup and cooldown limitations are in the Pressure and Temperature Limits Report (PTLR) and are based upon an initial NDTT of 40°F. These curves are periodically updated to ensure operation within the required stress limits. Specimens of the vessel, weld material, and heat affected zone are located within the core region to permit periodic monitoring of exposure and material properties relative to control samples, as defined in the Pressure and Temperature Limits Report (PTLR).

Preservice ultrasonic inspection of the reactor vessel and primary system piping welds was performed and an inservice inspection program, as defined in the Technical Specifications, is maintained.

The heatup and cooldown rates during plant life are predicted using conservative values for the change in NDTT due to irradiation. Operating limitations during startup and shutdown of the reactor coolant systems were evaluated using Appendix G, Protection Against Non-Ductile Failure, of the ASME Code, Section III, fracture toughness rules (Code Case 1514)

Heatup and cooldown curves in accordance with the method of Appendix G of Section III ASME Code showed that the Pressure and Temperature Limits Report (PTLR) limits were very conservative.

Reactor vessel integrity has been evaluated as part of the SEP Topic V-6, Reactor Vessel Integrity (NUREG 0569), and unresolved safety issues A-49, Pressurized Thermal Shock, and A-11, Reactor Vessel Materials Toughness. Information on these evaluations is provided in Section 5.3.3.

Steady-state and transient analyses are presented in Chapter 15. These analyses demonstrate that the design of the vessel meets the necessary requirements. Inspections ensure that the probability of undetected and rapidly propagating fracture of the reactor coolant system is minimized.

3.1.2.4.3 General Design Criterion 32 - Inspection of Reactor Coolant Pressure Boundary

CRITERION: Components which are part of the reactor coolant pressure boundary shall be designed to permit (1) periodic inspection and testing of important areas and features to assess their structural and leak tight integrity, and (2) an appropriate material surveillance program for the reactor pressure vessel (GDC 32).

Inservice inspections of the reactor coolant pressure boundary and methods and frequencies for performing these inspections have been developed. The inspection program developed includes interpretation and analysis of the results employing the latest techniques available at the time of inspection. This program is described in the Technical Specifications and in Section 5.2.4.

3.1.2.4.4 General Design Criterion 33 - Reactor Coolant Makeup

CRITERION: A system to supply reactor coolant makeup for protection against small breaks in the reactor coolant pressure boundary shall be provided. The system safety function shall be to assure that specified acceptable fuel design limits are not exceeded as a result of reactor coolant loss due to leakage from the reactor coolant pressure boundary and rupture of small piping or other small components which are part of the boundary. The system shall be designed to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished using the piping, pumps, and valves used to maintain coolant inventory during normal reactor operation (GDC 33).

The chemical and volume control system provides a means of reactor coolant makeup and adjustment of the boric acid concentration. Normally, makeup is added automatically from the boric acid blend system to the suction of the positive displacement charging pumps when the volume control tank falls below a preset level. Further decrease in the level of the volume control tank requires a valve alignment to the refueling water storage tank (RWST). The charging pumps, of which there are three, are capable of injecting coolant into the reactor

coolant system at a rate of 60 gpm each when powered from either the onsite or offsite electric power systems.

Protection against small breaks in the reactor coolant system is afforded by low level in the pressurizer which initiates isolation of the normal letdown purification path of the chemical and volume control system. Charging flow should then be sufficient to compensate for break flow.

For larger breaks, the resultant loss of pressure will cause reactor trip and initiation of safety injection. These counter measures will limit the consequences of the accident in two ways:

1. Reactor trip and borated water injection will supplement void formation in causing rapid reduction of the nuclear power to a residual level corresponding to delayed fissions and fission product decay.
2. Injection of borated water ensures sufficient flooding of the core to prevent excessive temperatures.

3.1.2.4.5 General Design Criterion 34 - Residual Heat Removal

CRITERION: A system to remove residual heat shall be provided. The system safety function shall be to transfer fission product decay heat and other residual heat from the reactor core at a rate such that specified acceptable fuel design limits and the design conditions of the reactor coolant pressure boundary are not exceeded.

Suitable redundancy in components and features, and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure (GDC 34).

The residual heat removal system, in conjunction with the steam power conversion system, is designed to transfer the fission product decay heat and other residual heat from the reactor core at a rate such that design limits of the fuel and the primary system coolant boundary are not exceeded. Suitable redundancy is provided with two residual heat removal pumps and two heat exchangers. The residual heat removal system is able to operate on either onsite or offsite power systems. Details of the system design are given in Section 5.4.5.

3.1.2.4.6 General Design Criterion 35 - Emergency Core Cooling

CRITERION: A system to provide abundant emergency core cooling shall be provided. The system safety function shall be to transfer heat from the reactor core following any loss of reactor coolant at a rate such that (1) fuel and clad damage that could interfere with continued effective core cooling is prevented and (2) clad metal-water reaction is limited to negligible amounts.

Suitable redundancy in components and features, and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is

not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure (GDC 35).

The Emergency Core Cooling System (ECCS) are provided to cope with any loss-of-coolant accident due to a pipe rupture. Cooling water would be available in an emergency to transfer heat from the core at a rate sufficient to maintain the core in a coolable geometry and to ensure that the clad metal-water reaction is limited. The Emergency Core Cooling System (ECCS) are capable of meeting the requirements of 10 CFR 50.46 and 10 CFR 50, Appendix K. Adequate design provisions are made to ensure performance of the required safety functions even with a single failure, assuming that electrical power is available from either the offsite or the onsite electrical power system. Emergency core cooling is discussed in Section 6.3.

3.1.2.4.7 General Design Criterion 36 - Inspection of Emergency Core Cooling System (ECCS)

CRITERION: The Emergency Core Cooling System (ECCS) shall be designed to permit appropriate periodic inspection of important components, such as spray rings in the reactor pressure vessel, water injection nozzles, and piping, to assure the integrity and capability of the system (GDC 36).

Important components of the Emergency Core Cooling System (ECCS) are examined on a periodic basis as defined in the Inservice Inspection Program. Except for the low-head safety injection nozzles on the reactor vessel, all other connections are either directly or indirectly to the primary system piping, thus being more accessible for examination. Periodic ultrasonic and visual inspection using remote equipment is performed on the low-head safety injection nozzles.

Valves and piping are periodically inspected visually with nondestructive inspections being performed where appropriate. The components located outside containment are accessible for leaktightness inspection during operation.

3.1.2.4.8 General Design Criterion 37 - Testing of Emergency Core Cooling Systems (ECCS)

CRITERION: The Emergency Core Cooling System (ECCS) shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system (GDC 37).

Components of Emergency Core Cooling System (ECCS) located outside the containment are accessible for leaktightness inspection during periodic tests.

All of the pumps of the Emergency Core Cooling System (ECCS) are started at intervals as specified in the Inservice Testing Program. Valve operability as well as system operability tests are performed during the MODE 6 (Refueling) shutdowns to demonstrate proper automatic operation of the Emergency Core Cooling System (ECCS). The required surveillance tests are described in the Technical Specifications.

3.1.2.4.9 General Design Criterion 38 - Containment Heat Removal

CRITERION: A system to remove heat from the reactor containment shall be provided. The system safety function shall be to reduce rapidly, consistent with the functioning of other associated systems, the containment pressure and temperature following any loss-of-coolant accident and maintain them at acceptably low levels.

Suitable redundancy in components and features, and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure (GDC 38).

Two systems based on different principles are provided to remove heat from the containment following an accident in order to maintain the pressure below the containment design pressure. Containment spray is supplied from two pumps each being fed from a separate electrical bus. Two fan coolers are fed from one safeguards bus with the other two being fed from another safeguards bus. Power is supplied from either the normal supply or from the associated emergency diesel. These systems are discussed in Section 6.2.2.

3.1.2.4.10 General Design Criterion 39 - Inspection of Containment Heat Removal System

CRITERION: The containment heat removal system shall be designed to permit appropriate periodic inspection of important components, such as the torus, sumps, spray nozzles, and piping to assure the integrity and capability of the system (GDC 39).

The two containment heat removal systems can receive appropriate periodic inspection of important components. Containment spray nozzles are tested by blowing air or smoke into the spray rings and checking each nozzle for flow. Periodic testing of the pumps is also done. Besides their safeguards role, the containment fan coolers are routinely used during operation to maintain ambient temperature inside the containment at acceptable levels. The periodic testing is described in the Technical Specifications.

3.1.2.4.11 General Design Criterion 40 - Testing of Containment Heat Removal System

CRITERION: The containment heat removal system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole, and, under conditions as close to the design as practical, the performance of the full

operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system (GDC 40).

The containment heat removal systems have the capability of being periodically tested as follows:

1. Containment fan cooler system.

   a. The containment fan-cooler units are used during MODES 1 and 2 and by those means are continuously monitored.

   b. The service water (SW) pumps operate when the reactor is in operation and therefore are continuously monitored.

   c. Periodic system tests demonstrate proper automatic operation of the safety injection system. A test signal is applied to initiate automatic action and verify that the components receive the safety injection signal in the proper sequence. The test demonstrates the operability of the valves, circuit breakers, and automatic circuitry.

2. Containment spray system.

   a. Design provisions are made to the extent practical to facilitate access for periodic visual inspection of all important components of the containment spray system.

   b. Permanent test lines for the containment spray loops are located so that all components up to the isolation valves at the spray nozzles may be tested. These isolation valves are checked separately.

   c. The containment spray nozzles are tested by blowing air or smoke through the nozzles and observing the flow.

The required periodic tests are described in the Technical Specifications.

3.1.2.4.12 General Design Criterion 41 - Containment Atmosphere Cleanup

CRITERION: Systems to control fission products, hydrogen, oxygen, and other substances which may be released into the reactor containment shall be provided as necessary to reduce, consistent with the functioning of other associated systems, the concentration and quality of fission products released to the environment following postulated accidents, and to control the concentration of hydrogen or oxygen and other substances in the containment atmosphere following postulated accidents to assure that containment integrity is maintained.

Each system shall have suitable redundancy in components and features, and suitable interconnections, leak detection, isolation, and containment capabilities to assure that for onsite electrical power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) its safety function can be accomplished, assuming a single failure (GDC 41).

There are two systems which are designed to clean up the containment atmosphere after a postulated loss-of-coolant accident:

1. The containment spray system includes the injection of sodium hydroxide solution into the spray into the containment to remove elemental iodine. The system consists of redundant active components each supplied from separate electrical buses. No single active failure will cause both subsystems to fail to operate. This portion of the system is described in Section 6.5.
2. Charcoal filters are placed into the air stream flow of two of the four fan coolers to remove iodine. Each of the fan coolers is provided with a high efficiency particulate air filter bank. These are described in Section 6.5.

In addition, two recombiner units are installed in the containment. The purpose of these units is to prevent the uncontrolled postaccident buildup of hydrogen concentrations in the containment. These are described in Section 6.2.5 By *Reference 3*, the NRC removed from the Ginna Technical Specifications the requirements related to the hydrogen recombiners and hydrogen monitors.

3.1.2.4.13 General Design Criterion 42 - Inspection of Containment Atmosphere Cleanup Systems

CRITERION: The containment atmosphere cleanup systems shall be designed to permit appropriate periodic inspection of important components, such as filter frames, ducts, and piping to assure the integrity and capability of the systems (GDC 42).

The containment atmosphere cleanup systems, with the exception of the spray headers and nozzles, are designed and located such that they can be inspected periodically as required. The spray headers and nozzles can be tested as described in the response of Criterion 39 (Section 3.1.2.4.10).

The systems are described in Section 6.2.5 and the surveillance requirements are given in the Technical Specifications.

3.1.2.4.14 General Design Criterion 43 - Testing of Containment Atmosphere Cleanup Systems

CRITERION: The containment atmosphere cleanup systems shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the systems such as fans, filters, dampers, pumps, and valves and (3) the operability of the systems as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the systems into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of associated systems (GDC 43).

The containment atmosphere cleanup systems are tested as described in Criterion 40 (Section 3.1.2.4.11). In addition, the efficiency of the high efficiency particulate air and charcoal filters is checked periodically as required by the Technical Specifications.

3.1.2.4.15 General Design Criterion 44 - Cooling Water

CRITERION: A system to transfer heat from structures, systems, and components important to safety, to an ultimate heat sink shall be provided. The system safety function shall be to transfer the combined heat load of these structures, systems, and components under normal operating and accident conditions.

Suitable redundancy in components and features, and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure (GDC 44).

The systems provided to transfer heat from safety-related components to the ultimate heat sink of Lake Ontario consist of the service water (SW) and the component cooling water (CCW) systems described in Sections 9.2.1 and 9.2.2, respectively.

Component cooling water is supplied by two redundant pumps (one operating, one standby) which are supplied with power from separate buses. The service water (SW) is supplied by four pumps, two being fed power from one safeguards bus, the other two from another safeguards bus. Only one pump is needed during safe shutdown operation or during the injection phase of a postulated loss-of-coolant accident, and two are required during the recirculation phase of the accident.

The systems are operable from offsite power or from emergency onsite power (from the diesel generators).

No single active failure results in system loss of function for those functions important to safety.

3.1.2.4.16 General Design Criterion 45 - Inspection of Cooling Water System

CRITERION: The cooling water system shall be designed to permit appropriate periodic inspection of important components, such as heat exchangers and piping, to assure the integrity and capability of the system (GDC 45).

Important components of the component cooling water (CCW) system are located in areas which are accessible for periodic inspection.

Most of the service water (SW) system piping is buried reinforced concrete pipe which is not readily inspectable; however, there are two redundant service water (SW) supply headers and failure of one would not be expected to affect the operability of the other. The service water (SW) system consists of a single loop header supplied by two separate, 100% capacity, safety related pump trains as described in the Technical Specification Bases.

3.1.2.4.17 General Design Criterion 46 - Testing of Cooling Water System

CRITERION: The cooling water system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and the performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation for reactor shutdown and for loss-of-coolant accidents, including operation of applicable portions of the protection system and the transfer between normal and emergency power sources (GDC 46).

Redundancy and isolation are provided to allow periodic pressure and functional testing of the system as a whole, including the functional sequence that initiates system operation, and the transfer between the normal and diesel power sources. One of the redundant pumps in the component cooling water (CCW) system is in service during MODES 1 and 2.

During routine plant operation two (2) Service Water (SW) pumps are typically in operation; however, during the summer three service water (SW) pumps are in operation. See Section 9.2.1 for a current discussion of requirements for pump operation.

3.1.2.5 Reactor Containment

These criteria are intended to establish the design requirements for the primary containment and to identify the means for satisfying these requirements, including fracture prevention leakage testing, containment testing, inspection, and isolation.

3.1.2.5.1 General Design Criterion 50 - Containment Design Basis

CRITERION: The reactor containment structure, including access openings, penetrations, and the containment heat removal system shall be designed so that the containment structure and its internal compartments can accommodate, without exceeding the design leakage rate and, with sufficient margin, the calculated pressure and temperature conditions resulting from any loss-of-coolant accident. This margin shall reflect consideration of (1) the effects of potential energy sources which have not been included in the determination of the peak conditions, such as energy in steam generators and energy from metal-water and other chemical reactions that may result from degraded emergency core cooling functioning, (2) the limited experience and experimental data available for defining accident phenomena and containment responses, and (3) the conservatism of the calculational model and input parameters (GDC 50).

The reactor containment structure, penetrations, valves, access openings, and the containment spray system are designed with margin to accommodate the temperature and pressure conditions associated with the loss-of-coolant accident and main steam line break, without loss of function.

The design of the containment is based on a postulated main steam line break or a double-ended rupture of a reactor coolant pipe, coupled with partial loss of the redundant engineered safety features systems (minimum engineered safety features).

The containment integrity evaluation is provided in Section 6.2.1.2.

3.1.2.5.2 General Design Criterion 51 - Fracture Prevention of Containment Pressure Boundary

CRITERION: The reactor containment boundary shall be designed with sufficient margin to assure that under operating, maintenance, testing, and postulated accident conditions (1) its ferritic materials behave in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the containment boundary material during operation, maintenance, testing, and postulated accident conditions, and the uncertainties in determining (1) material properties, (2) residual, steady-state, and transient stresses, and (3) size of flaws (GDC 51).

The concrete containment is not susceptible to a low-temperature brittle fracture.

The containment liner is enclosed within the containment and thus is not exposed to the temperature extremes of the environs. The containment ambient temperature during operation is between 50°F and 125°F. The minimum service metal temperature of the containment liner is well above the NDTT + 30°F for the liner material. Containment penetrations which can be exposed to the environment are also designed to the NDTT + 30°F criterion.

3.1.2.5.3 General Design Criterion 52 - Capability for Containment Leakage Rate Testing

CRITERION: The reactor containment and other equipment which may be subjected to containment test conditions shall be designed so that periodic integrated leakage rate testing can be conducted at containment design pressure (GDC 52).

The containment system is designed and constructed and the necessary equipment is provided to permit periodic integrated leakage rate tests during plant lifetime. Most of these periodic integrated leakage rate tests of the containment system were conducted at 58% of the reactor building design pressure (35 psig). However, periodic integrated leakage rate tests will be conducted at design pressure at intervals as described in the Containment Leakage Rate Testing Program.

3.1.2.5.4 General Design Criterion 53 - Provisions for Containment Testing and Inspection

CRITERION: The reactor containment shall be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations, (2) an appropriate surveillance program, and (3) periodic testing at containment design pressure of the leaktightness of penetrations which have resilient seals and expansion bellows (GDC 53).

There are special provisions for conducting individual leakage rate tests on applicable penetrations. Penetrations will be visually inspected and pressure tested for leaktightness at periodic intervals. Provisions have been made for an inservice tendon surveillance program throughout the life of the plant intended to provide sufficient inservice historic evidence to maintain confidence that the integrity of the containment is being preserved.

3.1.2.5.5 General Design Criterion 54 - Piping Systems Penetrating Containment

CRITERION: Piping systems penetrating primary reactor containment shall be provided with leak detection, isolation, and containment capabilities having redundancy, reliability, and performance capabilities which reflect the importance to safety of isolating these piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits (GDC 54).

Piping systems penetrating containment are designed to provide the required isolation and testing capabilities. These piping systems are provided with test connections as necessary to allow periodic leak detection to be performed. The Engineered Safety Features Actuation System (ESFAS) test circuitry provides the means for testing isolation valve operability. Details of the containment isolation capability are provided in Section 6.2.4.

3.1.2.5.6 General Design Criterion 55 - Reactor Coolant Pressure Boundary Penetrating Containment

CRITERION: Each line that is part of the reactor coolant pressure boundary and that penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

1. One locked closed isolation valve inside and one locked closed isolation valve outside containment; or
2. One automatic isolation valve inside and one locked closed isolation valve outside containment; or
3. One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or
4. One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

Other appropriate requirements to minimize the probability or consequences of an accidental rupture of these lines or of lines connected to them shall be provided as necessary to assure adequate safety. Determination of the appropriateness of these requirements, such as higher quality in design, fabrication, and testing, additional provisions for inservice inspection, protection against more severe natural phenomena, and additional isolation valves and containment, shall include consideration of the population density, use characteristics, and physical characteristics of the site environs (GDC 55).

During the design phase of Ginna Station, containment isolation valves were covered by a proposed criterion that existed at that time (AIF-GDC 53): "Penetrations that require closure for the containment function shall be protected by redundant valving and associated apparatus." The design response to this criterion is in Section 3.1.1.7.17.

The criterion in effect during the design phase was met. The compliance with Criterion 55 was reviewed during the Systematic Evaluation Program (Topic VI-4) and is discussed in Section 6.2.4.4.

3.1.2.5.7 General Design Criterion 56 - Primary Containment Isolation

CRITERION: Each line that connects directly to the containment atmosphere and penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

1. One locked closed isolation valve inside and one locked closed isolation valve outside containment; or
2. One automatic isolation valve inside and one locked closed isolation valve outside containment; or
3. One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or
4. One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to the containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety (GDC 56).

The review of the Ginna Station containment isolation valve provisions relative to GDC 56 was performed during the Systematic Evaluation Program (Topic VI-4) and is discussed in Section 6.2.4.4.

3.1.2.5.8 General Design Criterion 57 - Closed System Isolation Valves

CRITERION: Each line that penetrates primary reactor containment and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere shall have at least one containment isolation valve which shall be either automatic, or locked closed, or capable of remote manual operation. This valve shall be outside containment and located as close to the containment as practical. A simple check valve may not be used as the automatic isolation valve (GDC 57).

The installation of valves was done in accordance with criteria which were applicable at the time (AIF-GDC 53). A review relative to GDC 57 was performed in the Systematic Evaluation Program (Topic VI-4). Compliance with GDC 57 is discussed in Section 6.2.4.4.

3.1.2.6 Fuel and Radioactivity Control

These criteria are intended (1) to establish station effluent release limits and to identify the means of controlling releases within these limits, (2) to define the radiation shielding, monitoring, and fission process controls necessary to effectively sense abnormal conditions and initiate required safety systems, and (3) to establish requirements for safe fuel and waste storage systems and to identify the means to satisfy these requirements.

3.1.2.6.1 General Design Criterion 60 - Control of Releases of Radioactive Materials to the Environment

CRITERION: The nuclear power unit design shall include means to control suitably the release of radioactive materials in gaseous and liquid effluents and to handle radioactive solid wastes produced during normal reactor operation, including anticipated operational occurrences. Sufficient holdup capacity shall be provided for retention of gaseous and liquid effluents containing radioactive materials, particularly where unfavorable site environmental conditions can be expected to impose unusual operational limitations upon the release of such effluents to the environment (GDC 60).

The handling, control, and release of radioactive materials during MODES 1 and 2 is in compliance with 10 CFR 50, Appendix I, and is described in the Offsite Dose Calculation Manual.

Additional information concerning the liquid and gaseous radwaste systems is provided in Sections 11.2 and 11.3, respectively.

3.1.2.6.2 General Design Criterion 61 - Fuel Storage and Handling and Radioactivity Control

CRITERION: The fuel storage and handling, radioactive waste, and other systems which may contain radioactivity shall be designed to assure adequate safety under normal and postulated accident conditions. These systems shall be designed (1) with a capability to permit appropriate periodic inspection and testing of components important to safety, (2) with suitable shielding for radiation protection, (3) with

appropriate containment, confinement, and filtering systems, (4) with a residual heat removal capability having reliability and testability that reflects the importance to safety of decay heat and other residual heat removal, and (5) to prevent significant reduction in fuel storage coolant inventory under accident conditions (GDC 61).

The spent fuel pool (SFP) and cooling system, fuel handling system, radioactive waste processing systems, and other systems that contain radioactivity are designed to ensure adequate safety under normal and postulated accident conditions and are discussed in Section 9.1, and Chapters 11 and 15.

A. Components are designed and located such that appropriate periodic inspection and testing may be performed.

B. All areas of the plant are designed with suitable shielding for radiation protection based on anticipated radiation dose rates and occupancy as discussed in Chapter 12.

C. Individual components which contain significant radioactivity are located in confined areas which are adequately ventilated through appropriate filtering systems.

D. The spent fuel pool (SFP) cooling system provides cooling to remove residual heat from the fuel stored in the spent fuel pool (SFP). The system is designed such that, in addition to permanently installed equipment, temporary connections and equipment can also be utilized.

E. The spent fuel pool (SFP) is designed such that no postulated accident could cause significant loss of coolant inventory.

3.1.2.6.3 General Design Criterion 62 - Prevention of Criticality in Fuel Storage and Handling

CRITERION: Criticality in the fuel storage and handling system shall be prevented by physical systems or processes, preferably by use of geometrically safe configurations (GDC 62).

Criticality in new and spent fuel storage areas is prevented both by physical separation of fuel assemblies and by the presence of borated water in the spent fuel storage pool. Criticality prevention is discussed in detail in Section 9.1.2.

3.1.2.6.4 General Design Criterion 63 - Monitoring Fuel and Waste Storage

CRITERION: Appropriate systems shall be provided in fuel storage and radioactive waste systems and associated handling areas (1) to detect conditions that may result in loss of residual heat removal capability and excessive radiation levels and (2) to initiate appropriate safety actions (GDC 63).

Monitoring systems are provided to alarm on excessive temperature or low water level in the spent fuel pool (SFP). Appropriate safety actions will be initiated by operator action.

Radiation monitors and alarms are provided as required to warn personnel of impending excessive levels of radiation or airborne activity. The radiation monitoring system is described in Section 12.3.

3.1.2.6.5 General Design Criterion 64 - Monitoring Radioactivity Releases

CRITERION: Means shall be provided for monitoring the reactor containment atmosphere, spaces containing components for recirculation of loss-of-coolant accident fluids, effluent discharge paths, and the plant environs for radioactivity that may be released from normal operations, including anticipated operational occurrences, and from postulated accidents (GDC 64).

The containment atmosphere is continually monitored during normal and transient station operations using the containment particulate and gas monitors. In the event of accident conditions, samples of the containment atmosphere will provide data of existing airborne radioactive concentrations within the containment. Radioactivity levels contained in the facility effluent discharge paths and in the environs are continually monitored during normal and accident conditions by the station radiation monitoring system and by the Radiation Protection Program for Ginna Station as described in Sections 11.5 and 12.5.

REFERENCES FOR SECTION 3.1

1. Deleted

2. NRC Letter P. Milano to M. Korsnick (Ginna), "Modification of the Control Room Emergency Air Treatment System and Change to Dose Calculation Methodology to Alternate Source Term," February 2, 2005.

3. NRC Letter D. M. Skay to M. Korsnick (Ginna), "Amendment Eliminating Requirements for Hydrogen Recombiners and Hydrogen Monitors Using Consolidated Line Item Improvement Process," May 5, 2005.

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

3.2.1 INTRODUCTION

As part of the Systematic Evaluation Program (SEP), Topic III-1, the original codes and standards used in the design of structures, systems, and components at Ginna Station were compared with later licensing criteria based on Regulatory Guide 1.26 (*Reference 1*) and 10 CFR 50.55a. The objective was to assess the capability of Ginna Station structures, systems, and components to perform their safety functions as judged by the later standards.

Several areas were identified where requirements had changed; however, all areas were satisfactorily resolved as documented in *References 2 through 6*. The NRC has concluded that SEP Topic III-1 regarding classification of structures, systems, and components is resolved (*Reference 6*). Section 3.2.2 summarizes the results of the review.

Table 3.2-1 lists systems and components at Ginna Station, the code required to satisfy licensing criteria effective at the time of the SEP review, the codes and standards used when the systems and components were originally produced, the seismic classification in accordance with Regulatory Guide 1.29 (*Reference 7*), and the seismic classification used in the plant design. It should be noted that the original Ginna Station seismic design included three seismic classes, but the Regulatory Guide 1.29 comparison includes only two (Seismic Category I and nonseismic). Definitions of the original seismic classes are included in Section 3.7.1.1.

The following systems and their respective components are addressed in Table 3.2-1:

- Reactor coolant system.
- Safety injection system.
- Sampling system.
- Containment spray system.
- Chemical and volume control system.
- Residual heat removal system.
- Component cooling water (CCW) system.
- Service water (SW) system.
- Main Steam System
- Feedwater system.
- Preferred Auxiliary feedwater system.
- Standby auxiliary feedwater system.
- Containment isolation system.

3.2.2 SYSTEMATIC EVALUATION PROGRAM EVALUATION

After comparing the original codes with those currently used for licensing new facilities, the following areas were identified where the requirements had changed:

1. Fracture toughness.
2. Quality group classification.
3. Code stress limits.
4. Radiography requirements.
5. Fatigue analysis of piping systems.

It was determined that changes in the areas of quality group classification, code stress limits, and fatigue analysis of piping systems have not affected the safety functions of the Ginna systems and components reviewed. In the remaining two areas (e.g., fracture toughness and radiography requirements), although no significant deviations were identified, the evaluation was incomplete due to insufficient information available at the time of the evaluation. Additional specific information was requested by the NRC in these two areas and also on the design of certain valves, pumps, and storage tanks. That information is provided in the following sections. The information was submitted to the NRC by *Reference 5*. The NRC staff reviewed the information and concluded in *Reference 6* it was adequate to fully resolve the open issues in SEP Topic III-1 regarding classification of structures, components, and systems.

3.2.2.1 Fracture Toughness

For components not exempt from current fracture toughness requirements, the following evaluations were submitted to justify that fracture toughness is sufficient to ensure component integrity.

<u>3.2.2.1.1 Pressurizer</u>

The pressurizer evaluation is based on a conservative adaptation of ASME Section NC-2311(a)(8).

In order to make the evaluation, the lowest service temperature (LST) is defined. This is the minimum temperature of the fluid retained by the component or the calculated minimum metal temperature expected during MODES 1 and 2 whenever the pressure within the component exceeds 20% of the preoperational system hydrostatic test pressure.

The hydrostatic test pressure was 3125 psia. Thus, 20% of this pressure is 625 psia. The Ginna Technical Specifications and Pressure and Temperature Limits Report (PTLR) require, for Low Temperature Overpressure Protection (LTOP) System purposes, that reactor coolant system pressure relief setpoint must be lower than 430 psig (setpoint is determined in the PTLR which accounts for instrument uncertainty) whenever reactor coolant system temperature is lower than the enable temperature specified for LTOP in the PTLR or the residual heat removal system is in operation. The lowest service temperature is thus taken as the enable temperature specified for LTOP in the PTLR.

The pressurizer head material is SA-215 WCC, which has a T_{NDT} of 30 °F. Thus, the difference between the lowest service temperature and T_{NDT} is 292 °F, which is much greater than the acceptance criteria of 90 °F. Thus, it can be concluded that the pressurizer head material is exempt from impact testing.

The pressurizer shell material is SA-302 grade B material, the same material as the reactor vessel. This material has been shown to have adequate fracture toughness as concluded in SEP Topic V-6, Reactor Vessel Integrity.

3.2.2.1.2 Accumulators

The accumulators are constructed of SA-516, grade 70 material. The T_{NDT} of this material is 0°F. The lowest service temperature of the accumulator would be the minimum expected normal containment temperature, approximately 60°F during MODE 6 (Refueling) operations. (It should be noted that by procedure the accumulators are isolated from the reactor coolant system during cooldown, when reactor coolant system pressure is about less than or equal to 1500 psig. When the accumulators are in service and connected to the reactor coolant system, containment temperature is maintained less than 125°F.) For purposes of this evaluation, the lower figure was used.

The allowable (LST - T_{NDT}) for material up to 2.50-in. thick is 30°F. The actual (LST - T_{NDT}) is 60°F. Therefore, the fracture toughness of the accumulators is considered adequate.

3.2.2.1.3 Component Cooling Water (CCW) Pumps

The component cooling water (CCW) pump casing is cast iron.

The potential for complete failure of both component cooling water (CCW) pumps due to brittle fracture is considered minimal. One component cooling water (CCW) pump provides all required services; the second pump is a standby pump only. Thus, it is not expected that both pumps would fail. In addition, in 1983 RG&E purchased a spare component cooling water (CCW) pump to be stored on site which could be manually placed in service, if needed.

Thus, based on the number of backup component cooling water (CCW) pumps available, it was not considered that impact testing was required of the component cooling water (CCW) pump material. An evaluation by RG&E later determined that due to the thickness of the piping connected to the component cooling water (CCW) pumps, impact testing was not required by the ASME Code for the pump casing material. This evaluation eliminated the need to maintain a spare pump for the purpose of resolving brittle fracture concerns. (See also section 9.2.2.4.3.)

3.2.2.1.4 Service Water Pumps

The service water (SW) pumps are vertical shaft pumps, constructed of cast iron (discharge head) and carbon steel (intake column pipe). It is not considered that brittle fracture is a significant consideration for these pumps. This type of pump has been used in similar commercial applications for many years. It is not known that there have been any problems with brittle fracture of the pump material. Two of the four service water (SW) pumps are needed to perform safe shutdown cooling functions. It is very unlikely that all four pumps would experience simultaneous brittle fracture.

Rochester Gas and Electric has also made modifications during the course of the SEP to minimize the safety requirements for operation of the service water (SW) pumps. Fire hose connections have been provided for the diesel generators and for the standby auxiliary feedwater

system (SAFW) to allow safe shutdown operation, even in the event of a loss of the service water (SW) pumps.

Thus, it is not considered that impact testing is required for the service water (SW) pump material.

3.2.2.1.5 Main Steam Piping and Valves

The main steam piping greater than 20 in. is ASTM A155-65, grade C55, Class I. Main steam piping 20 in. and smaller is ASTM A106-65, grade B.

The normal service temperature for the main steam line is 514°F to 547°F at power. Although the T_{NDT} of the main steam piping material is not available, the fact that the lowest service temperature during the great majority of the operating time of this system is greater than 500°F would indicate that a fracture mechanics evaluation is not required.

3.2.2.1.6 Feedwater Piping and Valves

The feedwater piping material is ASTM A106-64, grade C. The normal service temperature of the final feedwater piping during full power operation is approximately 432°F. Although the T_{NDT} of these materials is not available, the fact that the lowest service temperature during the great majority of the operating time of the system for final feedwater temperature is greater than 400°F would indicate that a fracture mechanics evaluation is not required. This assessment of the potential for pipe fracture was accepted by the NRC in *Reference 6*.

3.2.2.2 Radiography Requirements

Information on the radiography requirements for (1) certain Class 2 pressure vessels and (2) Class 1 and 2 welded joints was requested.

3.2.2.2.1 Class 2 Pressure Vessels

The vessels in question include the accumulators, volume control tank, reactor coolant filter, seal-water injection filter, and charging pump accumulator. All main seams of the accumulators were required to be fully radiographed per ASME Code, Section 8, Paragraph UW-51 by Westinghouse Equipment Specification 676448, dated March 15, 1967.

The charging pump accumulator (or the charging pump filter) composite record indicates that all butt welds were radiographed.

The above pressure vessels were included in the Ginna Station Inservice Inspection Program for Quality Groups A, B, and C components.

Although these pressure vessels are Class 2 components, their failure would not result in the release of significant amounts of radiation. The failure of the volume control tank was analyzed in Section 15.7.1.2 as a design-basis accident. The radiological consequences of this failure were well within the guidelines of 10 CFR 100.

It was therefore concluded that, based on the original radiography performed on some of the pressure vessels, the inclusion of these pressure vessels in the inservice inspection program

and the minor consequences associated with any potential equipment failures, no additional radiography requirements were warranted.

3.2.2.2.2 Class 1 and 2 Welded Joints

It was determined that if the confirmation of Code Case N-7 (B31.1) was applied to all Class 1 and 2 piping, the radiography requirements for Class 1 and 2 welded joints would not be an issue.

Rochester Gas and Electric has confirmed that Code Case N-7 was used specifically for certain Class 1 and Class 2 piping systems, such as the primary loop and the safety injection system. In the specifications for other Westinghouse supplied systems, the statement is made that ASA B31.1 and all applicable nuclear code cases would be used. No specific mention of Code Case N-7 is made for these systems. However, Westinghouse Equipment Specification 676262, dated April 29, 1966, provides the weld inspection schedule for Westinghouse-supplied piping systems. All piping from Class 2501 to Class 601R was 100% radiographed. Random radiography was required for 10% to 20% of the balance of the welds, with evidence of unacceptable quality corresponding to random radiography being a cause to require 100% radiographic inspection. The remaining classes of piping (601 non-radioactive, 602, 301, 302, and 151) are primarily either (a) piping systems at or near the range of atmospheric temperatures up to 212°F (to which provision 2 of Code Case N-7 does not apply) or (b) Class 3 systems.

For Gilbert Associates supplied piping systems, GAI Specification SP-5291, dated December 23, 1966, provides the following radiography requirements.

Radiography inspection is to be made of all field butt welds and all field nozzle welds 4 in. and larger, for the following systems (only those of Class 2 are discussed below):

A. Main steam system up to main steam stop valves and connected piping for main steam safety valves (MSSV) and steam admission to the auxiliary feedwater pump turbine.
B. Feedwater piping to the first check valves outside containment (3992 and 3993).
C. Steam piping to the auxiliary feedwater pump turbine.
D. Preferred auxiliary feedwater piping.
E. Steam generator blowdown piping to the containment isolation valve.
F. Service water (SW) piping, including inside containment.

Also, all shop butt welds and all 4 in. and larger nozzle welds are required to be radiographed for the above systems.

Based on the above evaluation, it is concluded that the radiography requirements imposed on the original piping and valves for Ginna Station compare favorably with current criteria.

3.2.2.2.3 Main Steam and Feedwater Piping

The main steam and feedwater piping systems in the intermediate building and portions of the turbine building are included in the augmented inservice inspection program, which required

a new baseline radiograph inspection of 100% of welds in the subject high-energy piping. This program has been reviewed and approved by the NRC, in the review of SEP Topic III-5.B, Pipe Break Outside Containment, SER dated September 4, 1981 (*Reference 8*).

3.2.2.3 Valve Design

It was requested that information be provided, on a sample basis, regarding the design of valves in order to determine if (1) Class 2 and 3 valves meet current pressure-temperature ratings and (2) Class 1 valves meet current body shape requirements.

Rochester Gas and Electric has made an extensive sampling comparison and determined that, in almost all cases, the original pressure-temperature ratings were more restrictive than those defined in ANSI B16.34-1977. The valve specifications designate that the valve body materials be A312 type 304, A358, type 304, A376 type 304 (all group 2.1 materials), A312 type 316 or A358 type 316 (group 2.2 materials), A105, A216WCB (group 1.1 materials) and A216WCC (group 1.2 materials). In only one instance evaluated, for ASA 150 lb Class, did the Ginna specifications allow a higher working pressure for the designated temperature, and the difference was only 5 lb (210 lb versus 205 lb at 300°F, and 240 lb versus 235 lb at 200°F). This is a minor difference since hydrostatic testing of the systems was originally performed at 125% of design pressure.

It is thus considered that the pressure-temperature ratings for the Ginna Class 2 and 3 valves compare favorably with current criteria.

It was also requested that valve body shapes for Class 1 valves be compared to current criteria designated in the ASME Code, NB-3544.

A drawing review of a sample of Class 1 valves was conducted to determine if there appeared to be any significant differences from the valve body shape requirements of NB-3544. From the drawings, it appeared that (1) there were no sharp fillets at the intersections of the surfaces of the pressure retaining boundary at the neck to body junction (with $r_2 \geq 0.3\ t_m$), (2) body internal contours were generally smooth in curvature, (3) flat sections were minimized, and (4) body contours at weld ends were smooth and gradual.

This sampling indicates that Class 1 valves installed at Ginna Station have body shapes which are not significantly different from present code requirements. Further, during the years since Ginna Station began operation, periodic testing, and inservice inspection, no apparent failures due to severe stress concentrations resulting from unacceptable valve body shape contours have occurred or have been observed. It is thus considered that valve body shape requirements for Class 1 valves at Ginna Station are not of concern.

3.2.2.4 Pump Design

It was requested that information be provided with respect to the codes and requirements to which the gas stripper pumps, service water (SW) pumps, and lube-oil pumps for the turbine-driven auxiliary feedwater pump (TDAFW) bearings were designed.

The gas stripper pumps are not safety-related and the turbine-driven auxiliary feedwater pump (TDAFW) and its auxiliaries perform safety functions which can be performed by other

safety-related pumps. The service water (SW) pumps were analyzed as part of the seismic review (SEP Topic III-6) and modifications resulting from that analysis were performed based on current code requirements as discussed in Section 3.9.2.2.4.1.

3.2.2.5 Storage Tank Design

It was requested that information be provided relative to the design of the refueling water storage tank (RWST), boric acid storage tanks, chemical and volume control system holdup tanks, component cooling water (CCW) surge tank, preferred auxiliary feedwater condensate storage tank (CST), and turbine-driven auxiliary feedwater pump (TDAFW) lube-oil tank. In addition to general code requirements, specific information included compressive stress requirements, and tensile allowables for biaxial stress field conditions. An evaluation was not performed for the condensate storage tank (CST), the chemical and volume control system holdup tanks, and the turbine-driven auxiliary feedwater pump (TDAFW) lube-oil tank, since they are not required to perform a safety function. Both the condensate storage tank (CST), which provides suction to the preferred auxiliary feedwater system, and the turbine-driven auxiliary feedwater pump (TDAFW), have functions which can be performed by other safety-related systems (the service water (SW) system and the standby auxiliary feedwater system, (SAFW) respectively). The failure of the chemical and volume control system holdup tanks would not release significant activity (failure would be bounded by a volume control tank rupture, which is analyzed in Section 15.7.1.2, and found acceptable).

It should further be noted that the component cooling water (CCW) surge tank has a 100-psig design pressure and it is reviewed as a pressure vessel. Since fracture toughness exemption (nominal thickness 5/8 in. or less) applies for this tank and the stress limits between current and present codes are comparable for Class 3 vessels, no additional analysis was required.

The refueling water storage tank (RWST), chemical and volume control system holdup tanks, waste holdup tank, and the boric acid storage tanks were analyzed as part of the seismic review (SEP Topic III-6) based on current code requirements. Each tank has been shown to meet required SEP seismic criteria. The refueling water storage tank (RWST) and boric acid storage tanks are discussed in detail in Sections 3.9.2.2.4.6 and 3.9.2.2.4.5, respectively.

REFERENCES FOR SECTION 3.2

1. U.S. Nuclear Regulatory Commission, Quality Group Classification and Standards for Water, Steam, and Radioactive Waste Containing Components of Nuclear Power Plants, Regulatory Guide 1.26, Revision 3, February 1, 1976.

2. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic III-1, Quality Group Classification of Components and Systems, dated December 30, 1981 (includes Franklin Research Center Technical Evaluation Report C5257-429).

3. U.S. Nuclear Regulatory Commission, Integrated Plant Safety Assessment, Systematic Evaluation Program, R. E. Ginna Nuclear Power Plant, NUREG-0821, December 1982.

4. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-1, Quality Group Classification of Components and Systems, dated June 25, 1982.

5. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-1, Quality Group Classification of Components and Systems, dated January 24, 1983.

6. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report (IPSAR) Section 4.7, Classification of Structures, Systems, and Components, dated June 28, 1983.

7. U.S. Nuclear Regulatory Commission, Seismic Design Classification, Regulatory Guide 1.29, Revision 3, September 1, 1978.

8. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic III-5B, Pipe Break Outside Containment, dated September 4, 1981.

Table 3.2-1
CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
REACTOR COOLANT SYSTEM				
Reactor vessel	ASME III Class 1	ASME III (1965) Class A[b]	Category I	Class I
Reactor vessel supports	ASME III Subsection NF	---	Category I	Class I
Steam generators, tube side[c]	ASME III Class 1	ASME III (1965) Class A	Category I	Class I
Steam generators, shell side[c]	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Pressurizer	ASME III Class 1	ASME III (1965) Class A	Category I	Class I
Reactor coolant pumps	ASME III Class 1	ASME III (1965) Class A	Category I	Class I
Reactor coolant piping, valves, and fittings	ASME III Class 1	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Pressurizer relief tank	ASME III Class 3	ASME III (1965) Class C	Nonseismic	Class II

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
SAFETY INJECTION SYSTEM				
Refueling water storage tank (RWST)	ASME III Class 2	API-650 (1964) AEC TID-7024 (8/63) MSS SP-66 (1964)	Category I	Class I
High pressure safety injection pumps	ASME III Class 2	Westinghouse equipment Spec 676370[e] (7/29/66)	Category I	Class I
Accumulators with piping and valves to reactor coolant system and from N_2 supply	ASME III Class 2	ASME III (1965) Class C; ASA B31.1 (1955), ASA B16.5 (1961)[d]; Westinghouse equipment Spec 676448[f] (3/15/67)	Category I	Class I
Accumulator check valves	ASME III Class 1	MSS SP-66 (1964)[d]	Category I	Class I
Interconnecting piping and valves required to perform safety injection function	ASME III Class 2	ASA B31.1 (1955); Code Case N-7; USAS B36.10 (1959)[d]; USAS B36.19 (1965)[d]; USAS B16.5 (1961); MSS SP-66 (1964)[d]	Category I	Class I
SAMPLING SYSTEM				
Piping and valves from reactor coolant system to 951, 953, 955, 998	ASME III Class 1	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Piping and valves from 951, 953, 955, 998, to 966 A, B, C	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
CONTAINMENT SPRAY SYSTEM				
Containment spray pumps	ASME III Class 2	Westinghouse equipment Spec 676370[e] (7/29/66)	Category I	Class I
Piping and valves to containment spray system pumps from refueling water storage tank (RWST) and spray additive tank	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Spray additive tank	ASME III Class 3	ASME III (1965) Class C	Category I	Class I
Interconnecting piping and valves from containment spray system pump discharge to containment spray system spray nozzles	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
CHEMICAL AND VOLUME CONTROL SYSTEM				
Regenerative heat exchanger	ASME III Class 1	ASME III (1965) Class A	Category I	Class I
Nonregenerative heat exchanger-tube side	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Nonregenerative heat exchanger-shell side	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Reactor coolant filter	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Volume control tank	ASME III Class 2	ASME III (1965) Class C	Category I	Class II
Charging pumps	ASME III Class 2	Westinghouse equipment Spec 676370[e] (7/29/66)	Category I	Class I
Charging pumps accumulator	ASME III Class 2	ANSI B31.7 (1968)	Category I	Class I
Excess letdown heat exchanger-tube side	ASME III Class 1	ASME III (1965) Class A	Category I	Class I
Excess letdown heat exchanger-shell side	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Seal water injection filter	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Seal water heat exchanger-tube side	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Seal water heat exchanger-shell side	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Piping (loop B) letdown via regenerative heat exchanger and letdown valves to and including letdown orifices	ASME III Class 1	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Holdup tanks	ASME III Class 3	ASME III (1965) Class C	Category I	Class I
Boric acid storage tank	ASME III Class 2	ASME III (1965) Class C	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
CHEMICAL AND VOLUME CONTROL SYSTEM cont...				
Boric acid filter	ASME III Class 3	ASME III (1965) Class C	Category I	Class I
Gas stripper package[g]	ASME III Class 3	ASME III (1965) Class C ASA B31.1 (1955)	Nonseismic	Class III
Deborating demineralizer	ASME III Class 3	ASME III Class C	Nonseismic	---
Piping (loop A) letdown line via excess letdown heat exchanger to and including HCV 123	ASME III Class 1	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Piping and valves from pump discharge to containment isolation valve	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Mixed bed demineralizer	ASME III Class 3	ASME III (1965) Class C	Nonseismic	---
Cation bed demineralizer	ASME III Class 3	ASME III (1965) Class C	Nonseismic	Class I
Piping from pump discharge via reactor coolant pump and from HCV 123 to seal water heat exchanger	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Remainder of interconnecting piping and valves with exceptions following	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Piping and valves of TCV 145 via demineralizer to valves 1106 and 1107	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Nonseismic	Class I
Base removal ion exchanger	ASME III Class 3	ASME III (1965) Class C	Nonseismic	Class I
Cation ion exchanger	ASME III Class 3	ASME III (1965) Class C	Nonseismic	Class I
Ion exchange filter	ASME III Class 3	ASME III (1965) Class C	Nonseismic	Class I
Piping and valves from boric acid storage tank via boric acid transfer pump and filter	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	**Codes and Standards RG 1.26[a]**	**Codes and Standards Used in Plant Design (Historical)**	**RG 1.29**	**Used in Plant Design**
RESIDUAL HEAT REMOVAL SYSTEM				
Residual heat removal pumps	ASME III Class 2	Westinghouse equipment Spec 676370[e] (7/29/66)	Category I	Class I
Heat exchanger-tube side	ASME III Class 2	ASME III (1965) Class C	Category I	Class I
Heat exchanger-shell side	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Interconnecting piping and valves required to perform residual heat removal function	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
COMPONENT COOLING WATER (CCW) SYSTEM				
Pumps	ASME III Class 3	Westinghouse equipment Spec 676370[e] (7/29/66)	Category I	Class I
Heat exchanger	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Surge tank	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Interconnecting piping and valves	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
SERVICE WATER (SW) SYSTEM				
Pumps	ASME III Class 3	GAI Specification RO-2204 (1966)	Category I	Class I
Piping and valves required for containment cooling inside containment and outside containment up to first isolation valves	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Remainder of piping and valves excluding those inside the turbine building	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
MAIN STEAM SYSTEM				
Atmospheric relief valves (two)	ASME III Class 2	ASME III (1977) Class 2	Category I	Class I
Safety valves (eight)	ASME III Class 2	ASA B31.1 (1955)	Category I	Class I
Piping and valves comprising main steam lines extending from the secondary side of the steam generators up to and including the outermost containment isolation valve in each main steam line and connecting piping up to and including the first valve that is normally closed or capable of automatic closure during all modes of normal reactor operation	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Piping and valves from main steam line to auxiliary feed pump turbine	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)	Category I	Class I
FEEDWATER SYSTEM				
Interconnecting piping and valves comprising feedwater lines extending from secondary side of steam generators up to and including the nonreturn valves 4003, 4004, 4000C, 4000D, 3992, and 3993	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
PREFERRED AUXILIARY FEEDWATER SYSTEM (AFW)				
Pumps-motor driven	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Pump-turbine driven	ASME III Class 3	ASME VIII (1965)	Category I	Class I
Condensate storage tank (CST)	ASME III Class 3	AWWA D100 (1965)	Category I	Class II
Piping and valves from motor driven pump discharge to valves 4000C, D, and including valves 4304 and 4310	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)	Category I	Class I
Piping and valves from turbine driven pump discharge to valves 4003, 4004, and including 4291	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Piping to suction of Preferred auxiliary feedwater system (AFW) pumps from condensate storage tanks (CST) to valves 4014, 4017, 4016, and from service water (SW) system	ASME III Class 3	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
Turbine driven pump lube oil tank, pumps, and piping	ASME III Class 3	ASA B31.1 (1955) Westinghouse equipment Spec 676428[i]	Category I	Class I
STANDBY AUXILIARY FEEDWATER SYSTEM (SAFW)				
Pumps	ASME III Class 3	ASME III (1974) Class 3	Category I	Class I
Standby auxiliary feedwater system (SAFW) piping and valves from and including valves 9706A and B to steam generators	ASME III Class 2	ASME III (1974) Class 2	Category I	Class I
Piping and valves to pump suctions from service water (SW) system to and including valves 9707A, B; 9720A, B; and 9709A, B	ASME III Class 3	ASME III (1974) Class 3	Category I	Class I
Piping and valves to pump discharge up to valves 9704 A, B and including valves 9710 A, B	ASME III Class 3	ASME III (1974) Class 3	Category I	Class I

	Quality Classification		Seismic Classification	
Structures, Systems, and Components	Codes and Standards RG 1.26[a]	Codes and Standards Used in Plant Design (Historical)	RG 1.29	Used in Plant Design
CONTAINMENT ISOLATION SYSTEM				
Interconnecting piping and valves of the reactor coolant pressure boundary that penetrate the containment up to and including the outermost containment isolation valve	ASME III Class 2	ASA B31.1 (1955) ASA B16.5 (1961)[d]	Category I	Class I
STRUCTURES				
Containment, including access hatches, air locks, liner, penetration assemblies, fuel transfer tube penetration, and crane supports	NA	---	Category I	Class I
Auxiliary building	NA	---	Category I	Class I
Control building	NA	---	Category I	Class I
Spent fuel pool	NA	---	Category I	Class I
Intermediate building	NA	---	Category I	Class I
Diesel generator building	NA	---	Category I	Class I
Standby auxiliary feedwater system (SAFW) auxiliary building addition	NA	---	Category I	Class I
Screen house (service water (SW) portion)	NA	---	Category I	Class I
Turbine building	NA	---	Nonseismic[j]	Class III

a. ASME III stands for the Boiler and Pressure Vessel Code, Section III, Division I, published by ASME, 1977 Edition, with addenda through Summer 1978.

b. PCR 2001-0042 - Reactor vessel closure head replacement - performed in accordance with Section XI Repair Replacement Program utilized ASME Section III, 1995 edition, with 1996 addenda.

c. Replacement steam generator pressure boundary and integral attachments are designed in accordance with ASME Section III, Subsection NB, Class 1 requirements, 1986, with No Addenda.

d. Information regarding code edition assumed because it was not available during SEP review.

e. Westinghouse Equipment Specification 676370 refers to ASME Code, Sections III, VIII, and XI, 1965; ASA B16.5, 1961; and Standards of the Hydraulic Institute, 1965.

f. Westinghouse Equipment Specification 676448 requires that all main seams of the accumulators are fully radiographed per ASME Code, Section 8, Paragraph UW-51.

g. Consists of preheater, stripper column with reflex condenser, and associated pumps, piping, and instrumentation. Westinghouse Equipment Specification 676428 also applies to pumps.

h. A portion of the piping in the turbine building as shown in drawing 33013-1250, sht 1 of 3 is also ASME III.

i. In this case, Westinghouse Equipment Specification 676428 applies only to the pumps.

j. The turbine building was analyzed during the SEP and it was determined that the building could meet Seismic Category I requirements without failure. Those portions of the building required to maintain its overall structural integrity are now considered Seismic Category I.

3.3 WIND AND TORNADO LOADINGS

3.3.1 INTRODUCTION

As part of the Systematic Evaluation Program (SEP), the NRC staff reviewed the design and construction of certain structures to determine their ability to resist the forces developed by straight winds and tornadoes. The SEP review identified certain limiting structural elements (*Reference 1*), which were then addressed by RG&E as part of the Ginna Structural Upgrade Program. The Structural Upgrade Program consists of a two-phase structural reanalysis program followed by installation of required modifications identified as a result of the analysis. (See also Section 3.8.) The structural reanalysis program and the resulting modifications are discussed in the following sections.

3.3.2 STRUCTURAL UPGRADE PROGRAM EVALUATION

3.3.2.1 Structural Evaluation Approach

3.3.2.1.1 Requirements

The Structural Upgrade Program for tornadoes included the resolution of four interrelated SEP Topics:

II-2.A Severe Weather Phenomena.

III-2 Wind and Tornado Loadings.

III-4.A Tornado Missiles.

III-7.B Design Codes, Design Criteria, and Load Combinations.

The Standard Review Plan (SRP) Sections 3.3.1 and 3.3.2 and Regulatory Guides 1.76 and 1.117 include guidance relative to the need for nuclear power plants to withstand the effects of natural phenomena such as wind and tornadoes. At the time of design and construction of Ginna Station, the design criteria for nuclear power plants did not include tornadoes and other phenomena, such as extreme snow and tornado missiles, to the extent currently required. Consequently, the existing design and construction of some structures important to safety may not meet current licensing criteria but are, nonetheless, capable of resisting loads to some level between the current criteria and those specified in the original FSAR.

3.3.2.1.2 Structural Evaluation Process

The purpose of the Structural Upgrade Program evaluation was to determine the level of protection (tornado wind speed characteristics) that should be used as an appropriate backfitting basis for Ginna. In order to make this judgment, RG&E used a three-step process:

A. Determine the capability of the present Ginna structures, systems, and components to withstand tornado effects.

B. Determine the costs associated with backfitting tornado protection at several wind speeds up to that specified in current criteria.

C. Define a reasonable level of tornado protection, based both on the costs associated with a range of tornado wind speed protection levels and on the range of probabilities of these tornado wind speeds.

The following process was employed for the initial Structural Upgrade Program evaluation:

AA. Define loads, load combinations, and initial acceptance criteria.

BB. Define assumptions.

CC. Evaluate the effects on the structure.

DD. Compare these effects to the original assumptions.

EE. Assess these effects as they pertain to plant shutdown.

FF. Estimate the costs associated with the repairs.

GG. Based on the cost and effects, recommend final input and acceptance criteria and the recommended degree of repair.

The evaluation was performed in two parts. First, a structural evaluation was performed to determine the capabilities of all plant structures to resist wind, snow, and tornado wind and pressures. Second, a determination was made of the minimum set of plant equipment required to bring the plant to a safe shutdown condition and the impact of postulated tornado missiles on that capability. Backfit costs were estimated in both evaluations and were then combined in a consistent fashion to provide a uniform level of protection for all phenomena.

3.3.2.1.3 Structural Evaluation Computer Program

In order to perform a structural evaluation of this complexity, a complete evaluation of the main plant structures was made. This evaluation examined the interactions of the structures in the auxiliary, intermediate, turbine, diesel generator, and control buildings and the facade structure in order to distribute the loads throughout the entire structure in a manner that best simulates the actual field conditions. A separate evaluation was performed for the screen house. The computer program GTSTRUDL was used for the structural evaluation.

GTSTRUDL is a computer-aided structural engineering software system developed, maintained, and continuously researched at the GTICES Systems Laboratory, School of Civil Engineering, Georgia Institute of Technology.

3.3.2.1.4 Input Load Criteria

Before the actual evaluation could be made, structural layout and load data were compiled. Plan and elevation drawings of only the primary members and cross-bracing were made. These drawings were reviewed in the field and checked to confirm that the member configuration and location on the drawings agreed with the field conditions. Member sizes were checked randomly to verify that the member sizes in the field conform to the drawings.

The plant drawings were reviewed to determine the service and live loads on each floor. A field verification was done for the whole plant, whereby typical floor bays were examined, the equipment on these floor bays were located, and an estimated service load calculated.

The estimated service loads also included the weights of pipes, cable trays, and conduits which are attached to the floors.

Dead loads were assumed to be the weights of the structure, fixed equipment, an allowance for permanently attached system components (e.g. pipe, duct, and cable trays), and an allowance for thermal effects and pipe reactions. Live loads were assumed to be as shown in specifications and drawings, minus whatever was allowed for permanently attached system components. Dead, live, thermal effects, and pipe reaction loads were applied as equivalent uniform loads where applicable through the slabs or decking into the main framing.

A 75-mph wind speed and 40-psf ground snow load were used as the "severe environmental loading" condition.

An "extreme snow load" of 100 psf was used as a basis for the evaluation.

The effects of the two NRC design-basis tornado missiles on equipment required for safe shutdown were also examined. The missiles, a 35-ft utility pole and a 1-in. diameter steel rod, were examined to determine the effect a missile strike would have on the equipment required to safely shut down the plant. The two missiles (pole and rod) were assumed to travel at a speed of 0.4 and 0.6 times the tornado wind speed, respectively.

A spectrum of tornado wind speeds was chosen from the "Tornado and Straight Wind Hazard Probability" report prepared by Texas Tech University (*Reference 2*). Wind speeds of 250 mph, 188 mph, and 132 mph were used. These wind speeds coincide with the Texas Tech estimates for a probability of recurrence of 1 x 10^{-7}, 1 x 10^{-6}, and 1 x 10^{-5} per year, respectively, at an upper 95% confidence level.

The wind speeds were converted into design pressures by utilizing the ANSI 58.1-1982 equation:

$$\rho = q\, G_h\, C_p$$

where:	$q =$	$0.00256\, K_z\, (IV)^2$
	$K_z =$	velocity pressure coefficient
	$I =$	importance factor
	$V =$	fastest- mile wind speed
	$G_h =$	gust response factor
	$C_\rho =$	external pressure coefficient

Differential pressures were calculated by using $q = 0.00512\, V^2$ where V represents the translational wind speed. Wind loads were applied uniformly to the plant structures.

3.3.2.1.5 General Assumptions

Once the three tornado wind speeds were converted to design pressures, the following assumptions were made prior to applying these pressures to the structures:

A. Metal siding and roof decking remain intact and attached to the main steel frame for all load conditions.

B. All external block walls remain intact for all load conditions.

C. Plant windows, louvers, and doors remain intact for all load conditions.

These assumptions maximize the loads transferred into the structures. From these assumptions, the wind and snow load combinations were then applied to the structures as uniform loads. Their influence was transferred to the main steel framing through the siding or decking.

In the evaluation, the columns were input with their orientation corresponding with the field condition. The columns were assumed to be braced against lateral buckling by floor beams or struts which are framed into the column centerlines. Columns on the building perimeter that have girts attached to their flanges were assumed not to be laterally braced by the girts against buckling on the columns subjected to axial loads. The effective lengths were usually considered to be the distance between floors in the plant for both the strong and weak axis under column buckling and lateral buckling due to beam action. Column bases were typically modeled as pinned connections (non-moment-resisting). Floor beams were assumed to be laterally braced for bending by the floor slabs and beam to column connections were generally modeled as simple pin type connections.

Girts and purlins were considered to be secondary members in this evaluation. For positive wind pressure, the outside flange of the girt is in compression. Under this condition, the siding was assumed to provide full lateral support along the compression flange. However, negative wind or differential pressures reverse the compression flange to the inside of the girt or purlin. For this type of loading, the unbraced length of the compression flange was assumed to be the distance between supports.

The typical connection at Ginna Station is a bolted connection. Beam to column connections, in general, consist of angles welded to the beam and bolted to the column. Connections in the trusses or cross-bracing consist of members bolted to gusset plates. The connection evaluation was done in accordance with the guidelines of the American Institute of Steel Construction (AISC) and using basic statics and engineering mechanics.

Column anchorages were evaluated for basic shear and/or tension loads within the guidelines of ACI 349 Appendix B.

3.3.2.1.6 Load Combinations and Acceptance Criteria

Load combinations for severe, extreme and tornado loadings were evaluated, consistent with the NRC Standard Review Plan. The following load combinations were considered in this evaluation:

(1) $D + L + S_n + W$

(2) $D + L + S'_n$

(3) $D + L + W_t$

where:	D =	Dead load
	L =	Live load
	S_n =	100-year recurrence snow = 34 psf roof load for the power block, and 27 psf for the screen house
	W =	100-year recurrence wind = 75 mph for all structures
	S′n =	Extreme snow = 100 psf
	W_t =	Tornado wind loads as defined below, and corresponding to 250-mph, 188-mph and 132-mph tornado wind speeds.
	W_t =	W_w or,
	W_t =	W_p or,
	W_t =	$W_w + 0.5\ W_p$
	W_w =	tornado wind load
	W_p =	tornado differential pressure load

These load combinations have been broken into three categories. Load combination 1 is referred to as severe, load combination 2 is referred to as extreme, and load combination 3 is referred to as tornado.

Since a probability of occurrence for all these load combinations is considered to be very low, a 1.6S (1.6 multiplied by the allowable stress limit of the steel) acceptance criteria was used for the initial analysis.

A detailed discussion of loads, load combinations, and structural code comparisons were made as part of SEP Topic III-7.B. Details of the methods and results of that analysis are provided in Section 3.8.2.1.

3.3.2.2 Structural Evaluation

The structural evaluation combined the use of GTSTRUDL with hand calculations in order to accurately analyze the structural capacities of the primary members, secondary members, connections and anchorages, and building shell. The main structural framework was analyzed using the GTSTRUDL computer program in order to determine the forces and moments in the members for each load combination. GTSTRUDL was also used to calculate the structural adequacy of the secondary members under the same loading conditions used in the primary member evaluation, only on a representative sampling basis. The end reactions found in the primary member evaluations were used to evaluate the connections and anchorages in the plant using a statistical sampling technique.

3.3.2.2.1 Primary Member Evaluation

The analysis was performed using the computer program GTSTRUDL. Two three-dimensional structural computer models of the plant were developed. One model addressed only the screen house, which is separate from the main plant, while the other model consisted of the auxiliary, turbine, diesel generator, intermediate, and control, and the facade structure.

The models were developed by establishing a global coordinate system whereby only the main steel structures were described.

The models consist of columns, beams, cross-bracing, roof trusses, and other framing components of the structure that contribute to the horizontal strength of the plant. Main interior floor framing, adjacent buildings, and secondary components had their load influence input, but were not discretely addressed. Concrete floor and roof slabs (or decking) were assumed to be plate elements in the horizontal plane and were not developed in detail.

The plant structures were then analyzed for the load cases discussed in Section 3.3.2.1.6.

A software feature of the GTSTRUDL program is a means by which the resultant loads can be changed into stresses and checked to the AISC code. This procedure is done by assigning a number to each member in the computer model and inputting their respective properties (area, section modulus, radius of gyration, etc.). In the analysis, each primary member was checked in accordance with the Eighth Edition of the AISC code. The members which passed or failed the code check were listed, as well as a listing of the load combination which resulted in the overstressed condition.

3.3.2.2.2 Secondary Member Evaluation

Secondary members are those members whose purpose is to transfer the load from the intermediate areas of the roof and walls to the primary framing. These members consist of roof purlins and girts. The analysis was performed using GTSTRUDL in a similar manner as done in the primary members evaluation; however, a representative sample of the girts and purlins was investigated instead of inputting each individual member. A sample size of 70 purlins and girts were checked with the AISC code. This representative sample addresses 95% of all the roof purlins and girts in the plant. The percentage of failures discovered in this evaluation was extrapolated to provide the number of failures expected for the 1100 actual purlins and girts.

3.3.2.2.3 Connections and Anchorages Evaluation

The results of the primary and secondary member analyses were used to check the adequacy of the beam to beam, column to beam, column to base plate, and anchor bolt to base plate connectors, or simply, connections and anchorages. Since the plant contains approximately 6000 connections and 220 anchorages, a statistical approach was chosen in the review of these elements.

A statistical sample of 60 different connections was chosen and their associated axial and/or horizontal loads were applied and analyzed. Hand calculations and computer programs were used to check the strength of the bolts, welds, and clip angles for the applied loads. The resultant stresses were checked with the allowable stresses specified in the Eighth Edition of the AISC code. For those load conditions not addressed in the code (horizontal and axial loads occurring simultaneously), engineering mechanics were used to determine the adequacy of the connections. The results of this evaluation provided a percentage of overstressed connections which could be expected at a 95% confidence level. By multiplying this percentage by

the actual number of connections in the plant, an expected number of the connections that would not satisfy the acceptance criteria was determined.

A statistical sample of 53 anchorages in the plant was also chosen and evaluated using their associated loadings. A percentage of expected overstressed anchorages was found and multiplied by the total number of anchorages in the plant to determine the expected number of overstressed anchorages.

3.3.2.2.4 Exterior Shell Evaluation

3.3.2.2.4.1 Siding

Throughout the reanalysis program it was assumed that the siding would remain intact for all wind speeds. By making such an assumption, the load distribution was transferred evenly across all the steel framework, thus maximizing the load on the framework, while removing the effects of the wind pressure directly on the internal walls and equipment. To verify this assumption, Pittsburgh Testing Laboratory performed pressure tests on three types of siding at Ginna Station. These three types of siding, as manufactured by Elwin G. Smith Corporation, are:

a. Ribwall

b. Shadowall

c. "B" panel system

The ribwall panel system is located on the middle portion of the four sides of the facade structure while the corners of the facade consist of the shadowall panels. The rest of the plant is covered by the "B" panel system. A total of six tests were performed on each panel system. The six tests consisted of three positive and three negative pressure loadings. The positive tests represented a wind load from the outside of the structure while the negative tests represented pressure from the inside of the structure or a suction from the outside. The tests checked the failure load of the panels and the fasteners. Failure was defined as a loss of function resulting from tearing of the siding or failure of any or all of the panel connectors. Once the siding pressure capacities were determined, calculations were done to determine the corresponding wind speed for various areas of the buildings.

The results of the tests are discussed in Section 3.3.2.3 and are explained in more detail in the Pittsburgh Testing Laboratory report transmitted to the NRC by *Reference 24*.

3.3.2.2.4.2 Concrete Masonry Block Walls

The auxiliary, intermediate, control, and turbine buildings contain concrete masonry block walls. The interior block walls (building partitions) were assumed to contribute only their dead weight to the structure in the evaluation. No structural stiffness was considered.

For the purposes of this analysis, the exterior block walls were assumed to remain intact, contributing only their dead load, and were assumed to transfer the tornado wind loads into the steel structure. However, no credit for shielding or structural capacity to resist tornado forces was assumed for the block walls.

3.3.2.2.4.3 Architectural Items

Architectural items include doors, windows, and louvers. These items are not required to maintain their integrity in the Structural Upgrade Program.

3.3.2.3 Results of the Structural Evaluation

This section presents a summary of the results of the analysis and discusses overstresses and failures in terms of number of members, general failure mode, and failure location for the various components of the structures. Failure does not mean collapse of a member or a mechanism but instead means the inability of such a component to meet the recommended acceptance criteria. The results are presented based on the five load combinations listed below as compared to the acceptance criteria discussed in Section 3.3.2.1.6.

A. Severe environmental ($D + L + S_n + W$)

B. Extreme snow ($D + L + S'_n$).

C. Tornado winds of 132 mph ($D + L + W_{132}$)

D. Tornado winds of 188 mph ($D + L + W_{188}$)

E. Tornado winds of 250 mph ($D + L + W_{250}$)

<u>3.3.2.3.1 Primary Members</u>

3.3.2.3.1.1 General

The evaluation of the results of the various loading conditions on primary members was based upon the number of computer members rather than actual structural members. The number of failures shown are generally higher than the actual number of member failures. This is especially true for columns where one structural member may be represented by several computer members, depending on the location of the bracing and struts. Model 1 (main plant structure) contained 3500 computer members and model 2 (screen house) contained 766 computer members (see Section 3.3.2.2.1). In the discussion below, the turbine building also includes the control building and the diesel generator rooms. Table 3.3-1 provides a summary of the primary member failures for each building as well as a description of the failures. The numbers shown are accumulative and indicate the total number of failures for all load cases considered rather than an incremental amount of failures for each specific load case.

3.3.2.3.1.2 Severe Environmental Conditions

For severe environmental conditions, 168 primary members failed the acceptance criteria. Approximately 50% of all the failures were in the turbine building. The majority of the rest were about equally spread between the intermediate/ facade building and the auxiliary building with only about 5% in the screen house. For this loading case about one quarter of the failures are beams overstressed in bending from snow loads combined with axial wind loads. The remaining failures are about equally spread between column and bracing elements.
Many of these failures, particularly for bracing, are not due to overstress but due to excessive kl/r values for compression members as allowed by codes.

3.3.2.3.1.3 Extreme Snow Load Condition

One hundred and forty-one members failed the extreme snow load condition. Ninety-eight of these also failed severe loads, resulting in an additional 43 or a total of 211 failed members. About 50% of the additional failures occurred in the turbine building and about 25% each in the auxiliary and intermediate/facade area. Most of the additional failures were roof bracing members and roof truss members.

3.3.2.3.1.4 132-mph Tornado

A total of 258 members failed the acceptance criteria for a 132-mph tornado including differential pressure effects. One hundred and seventy of these members had failed the severe and/ or extreme environmental effects. An additional 88 failed members were due to tornado wind only. Seventy percent of the additional members were in the turbine building and consisted primarily of cross-bracing elements and various chord members of the roof trusses. Minor failures (about 15%) occurred in the beams and bracing of the screen house at 132 mph. The remaining 15% were miscellaneous additional members in the auxiliary and intermediate/ facade building. Approximately 36% of the 88 members failed were the direct result of the differential pressure loadings.

Of the 299 members that failed load combinations 1 through 3, slightly more than 54% are in the turbine building, about 21% are in the auxiliary building, 18% are in the intermediate building/facade, and 7% are in the screen house.

3.3.2.3.1.5 188-mph Tornado

A total of 332 members failed the acceptance criteria for a 188-mph tornado. This number included differential pressure failures which were projected using the 132 mph results. Similar to the 132-mph tornado, 177 of these members failed the severe and/or extreme environmental effects resulting in an additional 155 failed members caused by the 188-mph tornado alone. The percentage of the 155 failed members was distributed as follows: 20% for the combined auxiliary, intermediate, and facade structure; 55% for the turbine building and 25% for the screen house. Differentiating between a 132-mph tornado and a 188-mph tornado (67 additional members fail from 132-mph to 188-mph) the increased failures in the turbine building were 60% bracing, 40% columns; and in the screen house, 75% roof trusses and 25% bracing. The 20% of member failures located in all other buildings were found distributed evenly as beams and trusses. Approximately 38% of the 155 members failed were the direct result of the differential pressure loadings.

Of the 366 total members that failed the load combinations 1 through 4, slightly less than 52% were in the turbine building, about 18% were in the auxiliary building, 18% were in the intermediate building/facade, and 12% were in the screen house.

3.3.2.3.1.6 250-mph Tornado

A total of 658 primary members failed the acceptance criteria, including differential pressure failures, for the 250-mph tornado. As in the two previous tornado wind conditions, 178 of these members failed the severe and/or extreme environmental effects. Thus, for the 250-mph tornado wind, 480 failures were due to tornado wind alone. Of these 480 failures, 325

failures occurred as a result of the 250-mph tornado loading over and above those found due to the 188-mph tornado results. Of the 325 additional failures, 22% were in the turbine building, 38% in the screen house, 16% in the facade structure, 15% in the auxiliary, and 8% in the intermediate building. The majority of failures were bracing members, 32% of the 325, with 28% of the total being columns. The screen house roof truss system contributed 25% of the total by itself and the remaining 15% were composed of beams and other truss members. Approximately 21% of the 480 members failed were the direct result of the differential pressure loadings.

Of the 691 total members that failed the load combinations 1 through 5, 38% were in the turbine building, 17% were in the auxiliary building, about 21% were in the intermediate building/facade, and 24% were in the screen house. A tabular breakdown by building and member type for failures caused by load combinations 1 through 5 is shown in Table 3.3-1.

3.3.2.3.2 Secondary Members

For the extreme snow load of 100 psf, a few (21) isolated roof purlins became overstressed.

At a 132-mph tornado loading, approximately 60% of the total girts and purlins did not meet the acceptance criteria. These members were not considered to detach themselves from the main frame but experienced high stress levels and possible permanent deformations. The problems experienced by these members are due to tornado loads that create suction effects, and loads due to the differential pressure. For these load conditions, the bending stress allowables are low because of the large unbraced length of the compression flange.

When subjected to a 188-mph tornado, 77% of all secondary members experienced overload.

At a 250-mph tornado loading, 94% of the secondary members are overloaded and they would fail by bending or by failure of their connections to the main frame.

3.3.2.3.3 Connections and Anchorages

As described in Section 3.3.2.2, the connections and anchorages were statistically sampled and then evaluated for the various load combinations.

The results for the connection analysis showed that 11% to 13% of the connections failed the acceptance criteria for the severe environmental, extreme snow, and the 132-mph tornado loading conditions. As the tornado wind speeds increased the total percentages of failed connections went to 23% for the 188-mph and to 39% for the 250-mph tornado loadings.

No anchorages failed under the extreme snow loading based on the downward loading direction. For anchorages under the severe environmental loading and the 132-mph tornado loading, 18% failed in one of the three conditions checked for anchorage capacity: anchor bolts, welds to base plates, or concrete capacity. This number increased to 50% and 75% for the increased tornado loadings of 188 mph and 250 mph, respectively.

3.3.2.3.4 Exterior Shell

3.3.2.3.4.1 Metal Siding

The results of the siding tests determined the ultimate failure loadings. These results were then correlated to locations on the various buildings at Ginna Station. It was determined that with minor modifications all the exterior siding would perform its function under a 132-mph tornado loading. As the tornado loading increased to 188 mph all of the screen house siding failed, 28% of the total siding in the auxiliary building and the intermediate building failed, 23% of the turbine building siding failed, and approximately 50% of facade siding failed. When the 250-mph tornado loading results were calculated, 100% of the siding failed.

3.3.2.3.4.2 Roof Decking

The roof decking is acceptable for the extreme snow condition except for a few isolated spans. For a 132-mph tornado the theoretical calculations show that the roof decking itself is capable of supporting loads associated with this tornado. However, the decking to purlin connection might not be able to resist the uplift loads. As the tornado wind speeds are increased to 188 mph and 250 mph the portions of roof decking predicted to fail are 41% and 100%, respectively.

3.3.2.3.4.3 Block Walls

It was assumed that exterior block walls could not meet the structural requirement of the structural upgrade program.

3.3.3 TORNADO MISSILES AND SAFE SHUTDOWN APPROACH

3.3.3.1 Background

In the NRC April 16, 1982, Safety Evaluation Report, relative to SEP Topic III-4.A (*Reference 5*), it was determined that the majority of plant structures, systems, and components required to ensure the integrity of the reactor coolant pressure boundary; the capability to shut down the reactor and maintain it in a safe shutdown condition; and the capability to prevent accidents which could result in unacceptable offsite exposures were suitably protected from postulated tornado-generated missiles.

Several items were identified, however, which required additional evaluation with respect to tornado missile protection. An evaluation of these issues, as identified in the Ginna Integrated Plant Safety Assessment Report, NUREG 0821, May 1982 (draft) and December 1982 (final), as well as a number of other items identified during the RG&E subsequent reviews, are provided in Section 3.3.3.3.

The two missiles required in the Safety Evaluation Report (*Reference 4*) to be evaluated were a steel rod, 1-in. diameter and 3-ft long, weighing 8 lb, and a wooden utility pole, 13.5-in. diameter and 35-ft long, weighing 1490 lb. The velocity of the steel rod was assumed to be 60% of the tornado wind speed; the velocity of the wooden utility pole, 40%.

3.3.3.2 Shutdown Methodology

Rochester Gas and Electric has developed methods to achieve and maintain safe shutdown conditions following the postulated tornado strike. Certain assumptions of plant status and system unavailability were made.

3.3.3.2.1 Assumptions

A. Offsite ac power is lost.

B. All equipment not protected from tornado effects is considered inoperable unless explained otherwise in Section 3.3.3.3. Also, if protection is not specifically provided, it is assumed that inadvertent operation due to ground or phase faults could occur.

C. Architectural details, such as the building shell components and secondary members, are not considered capable of withstanding tornado windspeeds; however, the failure mode of these items is such that they will not become damaging missiles.

3.3.3.2.2 Shutdown Details

One train of safeguards equipment, which will serve to provide and maintain safe MODE 3 (Hot Shutdown), will be protected. Due to the nature and methodology of the shutdown systems being protected, MODE 5 (Cold Shutdown) can also be achieved.

The safe shutdown function will be performed as follows:

A. The reactor will automatically trip as a result of the loss of the unprotected 4-kV buses or other trip signal.

B. The turbine would trip, with resultant closure of the turbine stop valves. The operator would also close the main steam isolation valves from the control room, if they did not automatically close.

C. The diesel generators would automatically start and pick up the required loads. For purposes of this shutdown method, it is assumed that diesel generator 1B will be tornado protected. This would allow operation of all safeguards equipment associated with bus 16 (train B).

   Since service water (SW) is not protected, the diesel might not have this source of cooling water. Modifications have been made to the diesel cooling system to permit an alternate water supply to be used from the yard fire loop.

D. The standby auxiliary feedwater system would provide cooling to the steam generator(s). By using one of the main steam safety valves for venting to the atmosphere, a safe MODE 3 (Hot Shutdown) condition would be established. A 160,000-gallon DI water storage tank is available west of the standby auxiliary feedwater building which is used for standby auxiliary feedwater pump (SAFW) testing and serves as an additional source of condensate grade water. Following use of the contents of that tank, additional auxiliary feedwater could be provided from the yard fire loop.

E. Charging flow for inventory makeup of primary coolant would be available via the charging system. This function is presently tornado protected.

F. In order to cool down, use of the atmospheric dump valves on the main steam header would be required. If the air or the backup nitrogen systems that control these valves could not be made operable because they are not tornado protected, these valves could be locally controlled.

G. To effect final MODE 5 (Cold Shutdown), the steam generators would be used as water-to-water heat exchangers. Using established procedures, the operators would fill up the steam generators and, in an orderly manner, achieve a MODE 5 (Cold Shutdown) condition to less than 200°F. It is contemplated that this cooldown would occur over several days.

3.3.3.3 Required Components

The structures, systems, and components required to be tornado-missile protected are those required to achieve and maintain safe shutdown conditions. Other systems considered for protection include the surface of the spent fuel pool (SFP), so that missiles and other large items would not cause unacceptable damage to the fuel assemblies; the reactor coolant pressure boundary and main steam and feedwater lines, to prevent major primary and secondary system breaks; and items whose failure could cause unacceptable inadvertent operation or failure of safety-related equipment.

The RG&E proposed resolution of these items is as follows:

3.3.3.3.1 Refueling Water Storage Tank (RWST)

An analysis of missile effects (utility pole and steel rod) and wind pressure effects due to a 188-mph tornado, was performed for the refueling water storage tank (RWST). It was determined that a minimum safety factor for any of these load combinations is 1.18. For the refueling water storage tank (RWST) perforation analysis, the perforation formula contained in EPRI report NP-769, which accounts for the energy absorption due to deformation of the relatively soft utility pole missile, was used. For the steel rod, the Ohte Formula from the Strength of Steel Plates Subjected to Missile Impact was used (*Reference 5*).

3.3.3.3.2 Electrical Buses 14, 17, and 18

Bus 14 is located on the operating floor of the auxiliary building and could be subject to damage from tornado missiles. However, safety-related bus 16, located on the intermediate level of the auxiliary building, is protected from tornado missiles, and would be available in the event of a tornado.

Buses 17 and 18 are located in the screen house. The operating floor of the screen house is not protected from the effects of tornadoes including missiles. However, RG&E has made modifications which will eliminate dependence on the service water (SW) system to achieve and maintain safe plant shutdown. Thus, no protection for buses 17 and 18 is required.

Rochester Gas and Electric has also investigated the potential for damage to buses 17 and 18 causing failure of required electrical equipment, such as a diesel generator. In order to eliminate the potential damage from fault currents, RG&E installed a new feeder breaker between diesel generator 1B and bus 17 located in diesel generator room 1B.

3.3.3.3.3 Main Steam Lines A and B, and Main Feedwater Lines A and B

An analysis of the effects of the tornado missiles on the steam lines, feedwater lines, supports, and attached piping and valves at both 132 mph and 188 mph has been completed.

3.3.3.3.3.1 Results - Steel Rod

The main steam line, main feedwater line, as well as attached piping and valves, are all thick-walled items and would not be perforated by the steel rod impact. Damage to valve operators could prevent subsequent operation; however, no loss of pressure integrity would result. Thus, secondary system integrity would be maintained. The effect of damage to piping supports was also investigated. It was determined that damage could occur causing possible loss of support. However, damage to one support member would not result in a loss of overall support to the piping system. Thus, the main steam and feedwater lines would not be expected to lose support function to the point of failure.

In order to maintain safe shutdown, decay heat removal via one safety or relief valve would be required. Although no guarantee is available that the safety or relief valves would be operable following a steel rod strike, RG&E does not believe it would be credible to postulate simultaneous failure of all 10 safety and relief valves. Thus, RG&E is confident that decay heat removal capability via one safety or relief valve would exist following a tornado.

3.3.3.3.3.2 Results - Utility Pole

Based on the results of the analysis for the 188-mph tornado, the Ginna Station main steam lines and main feedwater lines will not be perforated by the utility pole.

The results confirmed both piping systems will withstand the effects of tornado wind and missile loads combined with normal operating loads within the acceptance criteria of Service Level D of ASME NC 3600 for Class 2 piping. In performing this analysis, it was conservatively assumed that snubber restraints were ineffective in resisting the tornado wind. It was also conservatively assumed that any snubber restraint impacted by the utility pole missile would fail. It was determined that there would be some permanent, but not unacceptable, deformation of both piping systems if impacted by the utility pole missile.

3.3.3.3.3.3 Failure of Block Walls

RG&E has also committed to evaluate the possible damaging effects on the steam and feedwater lines, due to failure of block walls. The block walls are located at the entire level in the intermediate building where the steam and feedwater lines are located. Based on the tornado missile evaluation, RG&E determined that local protection for the main steam isolation valve operators and solenoid valves and the preferred auxiliary feedwater system check valves were required. Protective structures were installed to protect these components (see Section 3.8.4.5.8), and the main steam isolation valve control cables were rerouted so as not to be susceptible to damage from failed walls.

3.3.3.3.4 Surface of the Spent Fuel Pool

An analysis has been performed for RG&E by Pickard, Lowe, and Garrick, Inc., entitled "Criticality Analysis for the Spent Fuel Storage Racks." It has been calculated that, even if the utility pole caused displacement of a fuel storage box, such that several fuel storage boxes were adjacent, a K_{eff} of significantly less than 0.8894 would result, with borated water of 2000 ppm in the pool (such is the case). Rochester Gas and Electric has also performed an analysis to determine the effects of a utility pole missile on the spent fuel assemblies. As provided in the RG&E proposed amendment to the Ginna Technical Specifications submitted by letter dated January 18, 1984 (*Reference 6*), it has been determined that the worst-case utility pole strike would not result in offsite radiological consequences greater than the guideline exposures of 10 CFR Part 100. See Section 9.1.2.7 for additional information.

Rochester Gas and Electric has modified the block wall on the north side of the spent fuel pool (SFP) to prevent damage to the spent fuel due to failure of the block wall. Calculations indicate that failure of the other block wall in the vicinity of the spent fuel pool (SFP) (west side) would not adversely affect the integrity of the fuel such that offsite radiological consequences would exceed 10 CFR 100 guidelines. Detailed calculations were completed as part of the Structural Upgrade Program.

3.3.3.3.5 Diesel Generators and Their Fuel Supply

Rochester Gas and Electric determined that additional protection was required for the doors and roof of a diesel generator room. Based on that analysis, the diesel generator building was modified to withstand seismic and extreme snow loads and to protect the building from external flooding and tornado winds and missiles. The modifications included construction of a new north face missile wall and a new roof structure. The north face missile wall included pressurized, missile-resistant, and watertight equipment and personnel doors and is constructed of reinforced concrete 4 ft north of the existing north wall of the diesel generator building. The existing east and west walls were extended in reinforced concrete to meet the new north wall. The new reinforced-concrete slab roof covers the entire building including the new north face missile wall. The existing north wall and portions of the roof were left in place. The diesel generator building was modified to be capable of withstanding wind pressure, differential pressure, and missile loads associated with a 132-mph tornado and to remain stable at a windspeed of 188 mph. "Capable of withstanding" means with no significant damage and "remain stable" means that the building will remain functional.

3.3.3.3.6 Relay Room

The east wall of the relay room is light gauge metal siding. Since the room contains vital safety-related equipment, RG&E committed to provide protection of this room from tornado winds and missiles, extreme snow, and design-basis flooding.

This protection has been accomplished by building a reinforced-concrete structure on the east end of the relay room. This structure is an enclosed space adjoining the east wall of the relay room that is approximately 14 ft wide by 40 ft long, extends from grade up to the control room floor, and is enclosed by a concrete roof slab. This structure has been designed for the above loads and the operating-basis earthquake and safe shutdown earthquake.

3.3.3.3.7 Service Water System

Rochester Gas and Electric has performed an evaluation of alternative shutdown methods, which do not require use of the service water (SW) system, to achieve and maintain safe shutdown. The methods include use of fire hose connections to the diesel generator and standby auxiliary feedwater system from the yard loop or from other sources as necessary. Thus, RG&E does not intend to provide tornado protection for the service water (SW) system.

3.3.3.3.8 Standby Auxiliary Feedwater System

Although the standby auxiliary feedwater system is protected by the standby auxiliary feedwater building, the discharge piping is routed through the auxiliary building. All of the discharge piping for the C pump is located on the intermediate level of the auxiliary building, and thus protected from tornado missiles, except for a small elbow section. This small section of piping is protected by concrete walls on the south and east sides and by the reactor makeup water tank on the north and west. The C pump and valves are associated with the power supply and distribution equipment (bus 14) that are not tornado-protected. The portion of the discharge piping for the D pump that is located in the auxiliary building operating level is not tornado-protected.

Power supply and distribution equipment (bus 16) for the D pump and valves are protected. Necessary changes were made to the standby auxiliary feedwater system to provide protection against tornado missiles. System isolation was provided for a postulated break in the D pump discharge piping so that the steam generator A can be fed via the standby auxiliary feedwater cross-connect piping. A motor-operated valve (MOV-9746) was added to the discharge line of D pump downstream of valve 9701B and the cross-tie containing valves 9702C and D. This provides a means of isolating the unprotected section of the D pump discharge header in the auxiliary building so that the D pump can feed train C through the existing cross-tie. Use of the protected bus 16 power supply for the D pump and active components can be utilized in the event of damage to bus 14. See Section 3.3.3.3.2. The alternative water supply from the yard fire hydrant loop to the standby auxiliary feedwater system is protected from tornado and missile damage. The line from the fire loop runs underground and terminates in the standby auxiliary feedwater building at a fire hose connection. The alternative water supply can be used by connecting an available length of fire hose between the fire hose connection and the connection point in the standby auxiliary feedwater system (see Section 10.5.2.3).

3.3.3.3.9 Instrumentation

Rochester Gas and Electric anticipates that some primary and secondary instrumentation may require rerouting from unprotected areas in the intermediate building to the intermediate floor of the auxiliary building.

Sufficient instrumentation will be provided for the operator to monitor safe shutdown conditions.

3.3.3.3.10 Cable Tunnel

An opening exists between the cable tunnel and the operating level of the intermediate building. This opening, which is 7 ft x 7 ft, begins 6 ft above floor level, and extends to just below the ceiling. The opening is shielded from tornado missiles on the south, east, and west directions by virtue of being below grade. From the north, major equipment in the turbine building, such as the condenser, will block virtually any missile. Based on the size of the cable tunnel opening, and the shielding now in place, RG&E does not believe any additional protection is warranted.

3.3.4 DESIGN TORNADO

3.3.4.1 Introduction

Based on the analyses, RG&E attempted to determine what level of tornado protection should be considered to be appropriate for use as a design-basis tornado for the Ginna facility. The design wind speed was chosen, considering many factors, including the cost of providing protection for increasingly severe tornado wind speeds and missile effects, and the potential safety benefit derived from the increasing capacities.

The cost of modifications increases substantially as the tornado wind speed is increased from a probability level of 10^{-5} to 10^{-6} to 10^{-7}. This is not unexpected, since the forces increase as the square of the wind speed. Rochester Gas and Electric has also attempted to consider the added safety benefit which would be derived by designing protection to increasingly severe wind speeds. Some additional safety benefit would exist as specific protection measures were increased; however, because of the substantial safety protection available for the most important plant structures and systems, such as the containment, control complex, and preferred auxiliary feedwater, the incremental safety benefit, although not quantified, is expected to increase only slightly with protection for increasing wind speeds. This is especially true when considering the additional materials capacity available in the plant structures not accounted for in the analysis, the lack of credit taken for safety system separation, and inherent wind and missile damage resistance.

Based on the following justifications, expected modification costs, and the safety level provided by the modifications to be implemented, RG&E recommended that protection be provided for a tornado of 10^{-5} (132 mph) (*Reference 3*).

3.3.4.2 Safety Assessment

Rochester Gas and Electric believes that the safety afforded by protection to a wind speed associated with a probability of 10^{-5} per year is adequate. The probability level selected is considered congruent with the protection levels associated with other severe natural phenomena, such as earthquakes and flooding, and with postulated events, such as pipe breaks. This level of protection is also compatible with the draft NRC secondary safety goal of a probability of 10^{-4} per year of core melt. Rochester Gas and Electric believes that the tornado risk will be only a small fraction of the total core melt risk. It is important to note that there is conservatism even in the 10^{-5} value selected as the backfitting design basis for

tornado protection at Ginna. First, the 10^{-5} wind speed is associated with the upper 95% confidence level, rather than the median. At a median level, the selected wind speed would have a probability on the order of 10^{-6} per year. Secondly, many of the structures, systems, and components required for safe shutdown, such as the containment, control building, and preferred auxiliary feedwater system, will withstand wind speeds significantly higher than those associated with the 10^{-5} level. Finally, the method of analysis to determine current protection, and any subsequent modifications, is conservative. Tornadoes are postulated to strike the plant from all directions, and thus no credit for shadowing, or physical separation, is claimed. Tornadoes are postulated to strike with equal intensity throughout the plant, thus seemingly affecting all structures, systems, and components with equal intensity concurrently. Actually, only a fraction of the plant would see the most intense characteristics of the tornado, and residual strength is expected in the Ginna structural and equipment elements beyond that assumed in the analysis. These conservatisms are described in more detail in Section 3.3.4.3.

Rochester Gas and Electric has determined that backfitting to a tornado level associated with a 10^{-5} tornado wind speed, at the upper 95% confidence level, will provide a significant level of plant protection. Further conservatisms inherent in the selection of tornado characteristics and the analysis process provide confidence that the risk associated with a tornado strike of this magnitude would be only a small fraction of the overall risk associated with the operation of Ginna Station.

3.3.4.3 Reserve Plant Capacity

An examination of the results of the standard evaluation was made in order to establish an approximate value of the reserve capacity of the plant framing after completion of the structural upgrade.

A. Theoretical physical properties of the materials that exist in the structure and those that are used for analysis are typically lower than the actual values. For example, A36 steel has a minimum yield strength of 36 ksi but typically the actual yield values are higher.

B. The structural upgrade would be done to ensure that there are no actual failures in the primary structural framing. This means that the buildings generally would be upgraded based on elastic behavior, i.e., strains below the yield stress. In reality, steel structures are capable of absorbing a large amount of energy above the yield strain of the material. For mild steel, the ratio of strain at rupture to strain at first yield is as much as 100 times the yield strain value. This ductility feature of steel implies that gross and sudden failures will not occur at design levels although permanent deformations may result.

C. The application of loads for analysis purposes is conservative. The live loads used in the analysis are those that are defined for design considerations. In reality, the full live load on all floors will not occur simultaneously. However, for the analysis and evaluation, the full live loads were applied. These loads are vertical and contribute to the total state of stress in the beams and columns.

D. The evaluation examined the plant for tornado winds applied in four directions (north, east, south, and west). The number of overstressed members which were found as a result of the 132-mph wind speed is the total of all failures found in all four directions. The

recommended upgrade will modify all these primary members regardless of the wind direction. The actual occurrence of a tornado would affect the plant from only one direction. Therefore, the upgraded plant will have inherent conservatism because the actual number of members experiencing high loads for a single direction tornado will be less than the total number that will be upgraded.

E. The analyses that were performed assume that the building response is completely elastic. In most steel structures, local plastic deformations will occur in conjunction with the elastic response of the main frame system. For bolted steel structures such as those at Ginna Station, some degree of slipping will occur in the connections when they are loaded with these extreme loads.

   The combination of local deformations and slipping in the connections will absorb some of the total load that is applied to the structure and lessen the total stresses predicted by the elastic analysis.

F. The results of the evaluation have shown that of all the tornado wind speed components, the differential pressure had the most significant impact on the secondary members and exterior shell. At the design tornado wind speed of 132 mph, certain areas of the plant siding and secondary members experience large deflections and minor failures, primarily along the edges and corners of the roof. Rochester Gas and Electric has proposed to allow the secondary members and siding to fail, since they will have no consequence on the overall plant integrity. However, the failures of these areas of the exterior shell will tend to relieve the differential pressure by providing additional venting of the structure along with the existing vent area in all the buildings. The vent area will reduce, if not eliminate, the loads created by the differential pressure. The result will be an immediate stress relief for all the plant structures.

3.3.4.4 System Reserve Capacity

In addition to the structural reserve capacity expected to be available, due to material specifications and analytical methods, substantial conservatisms were incorporated into the safety system analysis assumptions.

In terms of tornado wind and missile protection, RG&E has assumed that, unless specifically analyzed for or denoted otherwise, failure of an unprotected system or structure would occur. Generally, no credit has been taken for the protection inherent in the equipment itself to resist tornado winds. In fact, the majority of items would not experience the peak wind characteristics of the design-basis tornado. Thus, realistically, separation of components and the equipment capability would lessen the number of failures.

For tornado missiles, RG&E has assumed that all equipment not tornado-missile protected could be damaged. Actually, for the design-basis wind speeds expected, only the lightest objects would be capable of experiencing the aerodynamic forces to actually become missiles. These lighter objects would not be expected to cause substantial damage. Also, shadowing of components would be expected to be highly effective in ameliorating missile damage.

Further, for tornado missiles, it is assumed that there is an equal probability of damage to all unprotected equipment. On a probabilistic basis, this would not be expected to occur. The probability of a tornado missile striking small objects would be expected to be significantly lower than the probability of the tornado itself, which is already considered a 10^{-5} to 10^{-6} per year event. Therefore, on a realistic basis, additional safety margins exist for tornado missile protection.

3.3.5 STRUCTURAL UPGRADE PROGRAM

3.3.5.1 Introduction

The general approach proposed by RG&E was found acceptable, as noted in the NRC SER of August 22, 1983 (*Reference 7*), with certain outstanding items yet to be resolved. Also, concurrence with the general approach, design inputs and evaluation criteria was issued as a result of the recommendations of the Advisory Committee for Reactor Safeguards (ACRS) in an April 9, 1984, letter to the Honorable Nunzio J. Palladino, Chairman of the USNRC (*Reference 8*). Certain technical issues were resolved and certain changes made in input assumptions, acceptance criteria, and analytical methodology in the following areas:

A. Changes were made to the criteria as deemed appropriate during the course of the more detailed engineering analysis conducted for the RG&E recommended design tornado.

B. The criteria and judgments that were used to assess the capability of the upgraded structure to remain stable at tornado speeds above the RG&E recommended tornado (up to approximately 200 mph).

C. Open items discussed in the Technical Evaluation Report dated August 2, 1983 (*Reference 9*).

D. Outstanding issues related to SEP Topic III-7.B.

E. ACRS concern on diesel generator operability due to differential pressure effects.

The above items are discussed in more detail in the following sections.

3.3.5.2 Criteria Changes

Additional reviews of the results of the initial evaluations were performed. The purpose of these additional reviews was to provide a more exact estimate of the type and location of the overstressed components. A two-stage approach was used for these reviews to better predict the actual components requiring modifications and the extent of overstress.

<u>3.3.5.2.1 First Stage Review</u>

The first approach was to provide a more detailed engineering review of the results of the initial analysis. Primary members were reviewed on an individual basis to determine if the computer-predicted stresses for the overstressed members were correct or if these members could be shown to be acceptable using a more detailed engineering analysis. Connections and anchorages were reviewed for the purpose of defining specifically where the overstresses occurred. The number of overstressed connections and anchorages initially reported were

based on statistical samples which were found, based on this reevaluation, to be overly conservative.

The following list summarizes the bases used in the first approach to reduce the quantities of overstressed components:

A. The screen house was deleted from the scope since this structure is not required to achieve plant shutdown.

B. The computer model was reviewed for compatibility with the actual structure since the computer model tended to idealize the actual structure (by the use of simplifying conservative assumptions).

C. The turbine building operating floor maintenance live load was reduced from 1000 psf to 100 psf since the larger load is only present during turbine/generator maintenance when the plant is already in the shutdown mode.

D. The members with excessive kl/r ratios were evaluated to determine the actual load carrying capability of the members.

E. Modifications for those members whose failure would not damage required safety equipment were deleted.

F. Individual or groups of actual anchorages were evaluated instead of using a statistical projection.

G. Individual or groups of actual connections were evaluated instead of using a statistical projection.

3.3.5.2.2 Second Stage Review

The second approach modified the original evaluation criteria. Any components found over-stressed after the first evaluation were reevaluated considering three criteria changes.

A. Live load reductions.

   The criteria for all floors, other than the turbine building operating floor, reduced the live loads to 25% of the loads shown on the construction drawings. This criteria change is consistent with live load reductions used for other extreme loading conditions and also is consistent with current industry practice.

B. Increased yield stress.

   The original evaluation criteria specified that the minimum specified yield stress (F_Y) of the steel be used. The structural steel specifications for the Ginna plant require the use of A36 steel (F_Y = 36 ksi). This criteria change will take advantage of normally higher yield stresses in the steel, and also account for the plastic versus elastic shape factors. The new criteria applied a factor of 1.2 to F_Y.

C. Reduction or elimination of tornado differential pressure.

   The original evaluation criteria specified a tornado-induced differential pressure of 0.4 psi. This differential pressure would exist only for a completely sealed structure. The previous evaluation took no credit for existing openings (doors, windows, heating, ventilating, and

air conditioning vents, etc.) which would provide venting of the buildings and thereby reduce or eliminate the effective differential pressure. The new criteria will account for the existing areas. Where possible, additional vent area will be added to either reduce or eliminate the differential pressure loads.

3.3.5.3 Stability Evaluation

In order to demonstrate that the ultimate plant capacity was actually greater than the level of the recommended design tornado, a stability evaluation was performed. This evaluation assumed that the structures were upgraded to withstand the tornado windspeed of 132 mph and the other extreme loads previously mentioned. The assessment was performed using the component maximum strength and employing the following criteria:

3.3.5.3.1 Primary Members

Primary members were evaluated for stability by assessing the members for the actual loads associated with the 188-mph tornado wind speed and using the maximum strength that those members could develop. The allowable compressive load for column members was assumed to be equal to the theoretical buckling load. For bending elements, the allowable load was based on the theoretical lateral buckling stress.

Allowable tension stress on the members was assumed to be equal to the minimum specified yield strength on their gross area or 80% of the ultimate strength on the effective net area.

All other allowable stresses not covered above were evaluated to a 1.6S x 1.2 acceptance criteria where S is as defined in AISC.

The following criteria were also used in the overall stability assessment:

A. Column Research Council plastic design formulas were used to evaluate columns.

B. A diagonal brace (in compression) in a cross-braced bay was considered to support its buckled load because the complimentary tension brace prevents excessive deflection.

C. Compression member lengths were evaluated using an effective length factor that was more representative of the actual details.

3.3.5.3.2 Connections and Anchorages

Connections and anchorages for the primary members evaluated for stability which did not meet the 1.6S x 1.2 criteria specified in the SRP, were evaluated using the following criteria.

For connections:

A. The plastic bending capacity of double clip angles was used.

B. Higher bolt shear stresses for threads out of shear plane were used.

C. A compression diagonal brace is considered to support its buckled load because the complementary tension brace prevents excessive deflection thereby reducing the load on the tension brace connection.

D. For some bracing members containing numerous bolts, the fixity of that brace was assumed to be between a fixed end condition and a pinned end condition. An effective length factor of 0.65 was used which increased the compression capacity of the member, thereby reducing the load on the complementary tension member and its connections.

For anchorages:

A. The ultimate shear and tensile strengths for anchor bolts were used.

B. The plastic bending capacity of double clip angles was used.

C. The beam pockets in the control building were considered to be capable of restraining the beam after anchor bolt failure.

3.3.5.4 NRC Technical Evaluation Report (SEP Topic III-2) Open Items

The following were responses to the issues raised in the NRC Technical Evaluation Report dated August 2, 1983 (*Reference 10*).

3.3.5.4.1 Effective Tornado Loadings

Atmospheric pressure change

"RG&E made a commitment to reexamine the calculation for atmospheric pressure changes and will apply the appropriate value in the structural loadings."

The atmospheric pressure drop used by RG&E in the evaluation for a 132-mph tornado was 0.4 psi. Franklin Research Center calculated a pressure drop of 0.46 psi using the minimum translational speed of 5 mph noted in Regulatory Guide 1.76. The translational speed corresponding to a 0.4 psi pressure drop is 12.8 mph. The regulatory guide only provides guidance that the minimum translational speed be used in regard to the ultimate heat sink calculations for the plant. Use of the minimum speed for structural design considerations is not specified by Regulatory Guide 1.76. The 12.8 mph translational speed was originally judged reasonable and it is thus considered that the 0.4 psi pressure drop is acceptable.

Windborne missiles

"RG&E has made a commitment to reexamine the effects of tornado-induced missile impacts on the primary structural members throughout the Ginna facility in its final analysis."

Rochester Gas and Electric commissioned a study, "Utility Pole Tornado Missile Trajectory Analysis," by Dr. Larry Twisdale of Applied Research Associates. In that study it was concluded that wind speeds lower than approximately 150 mph could not provide the necessary aerodynamic lift required for a utility pole to become an airborne missile. Thus, at a wind-speed of 132 mph, it was determined that there would be no adverse effect on the primary framing of Ginna structures due to a utility pole missile. At higher wind speeds approaching 200 mph, it was considered credible that a utility pole missile could become airborne for short distances. However, the probability of a utility pole missile damaging the primary Ginna structures at high wind speeds becomes increasingly small, since the probability of a high wind speed (10^{-5} at 132 to 10^{-6} at 188 mph) must be coupled with the

probability of actually hitting a primary structural element (this was estimated to be about 25% in the study, based on an area ratio to effective missile length distribution function). Thus, it is estimated that the probability of actually hitting and damaging a primary member is less than 10^{-6}, and thus is not of concern with respect to tornado protection design efforts.

3.3.5.4.2 Structural Loadings

Effective structural pressures

"RG&E has made a commitment to examine the local effects of peak pressures on primary members in the final analysis."

Rochester Gas and Electric has committed to upgrade the structure to withstand the effects of a 132-mph tornado on a stress basis. In addition, a commitment has been made to assure stability of the structure to the 188-mph wind speed. Since the average pressure associated with the 188-mph tornado is approximately the same as the peak pressure associated with the 132-mph tornado, ensuring stability (and thus, ensuring that all safety functions are met) at the average pressure for the 188-mph tornado is in effect the same as designing for the peak pressure associated with the 132-mph tornado.

3.3.5.4.3 Structural Acceptance Criteria

Roof deck

"RG&E stated that the roof decks will be reexamined for potential buckling under extreme environmental loadings. The capacities of the roof decks will be modified accordingly."

The evaluation of the roof decking done in the Structural Upgrade Program considered that the allowable stresses associated with the steel roof decking found at Ginna Station would be increased by 1.6 in accordance with the Standard Review Plan for extreme load cases. Based on information found in the American Iron and Steel Institute (AISI), "Specifications for the Design of Cold-Formed Steel Structural Members," a theoretical buckling stress for the roof decking has been estimated to be greater than the actual yield stress of the material. Stress levels found in the roof decking as a result of the extreme snow load are, in nearly all cases, less than the allowable stress of the steel decking multiplied by the 1.6 allowable overstress. For the remaining areas where the stress levels were found to be greater than the Standard Review Plan allowable stresses, the actual stress was still found to be less than the yield stress of the material. It is RG&E's conclusion that since all of the stresses associated with the extreme snow load were found to be less than the yield stress of the material (and concurrently less than the theoretical yield stress of the material), local buckling of compression areas of the decking will not occur.

3.3.5.4.4 Structural Systems

Control building

"RG&E has made a commitment to reexamine the control building east wall for the structural upgrade."

The east wall of the relay room (part of the control building) has been modified to withstand wind and tornado loadings, including missiles. The east wall of the control room was found capable of resisting these loads (*Reference 10*).

Diesel generator building

"RG&E has made a commitment to reexamine the reinforced concrete structures of the diesel generator building in the final analysis."

The diesel generator building has been modified to withstand wind, tornado, including missiles, and seismic loadings.

3.3.5.5 SEP Topic III-7.B, Loads, Load Combinations, and Design Criteria

The RG&E initial submittal, dated May 27, 1983 (*Reference 11*), defined all applicable loads and load combinations considered limiting for the concrete and steel safety-related structures at Ginna Station. In the NRC Safety Evaluation Report of August 22, 1983 (*Reference 7*), it was determined that the proper load combinations had been used in the structural reevaluation of Ginna structures.

The application of the wind and tornado loads was applied as a constant uniform load over the height of each structure, instead of stepping the wind pressure as stated in ANSI A58.1-1982. These loads were applied to the windward, leeward, sides, and roofs of all buildings, using the appropriate pressure coefficients. It was determined that the variations in the total load transferred into the structure by this assumption was small and would not affect the results of the overall analysis.

A related issue was a comparison of the steel and concrete codes used in the original Ginna design versus current codes. The following comparisons were made:

- AISC 1980 (*Reference 12*) versus AISC 1963 (*Reference 13*).
- ACI 349-80 (*Reference 14*) versus ACI 318-63 (*Reference 15*).
- ASME Section III, Division 2, 1983 (*Reference 16*) versus ACI 318-63 (*Reference 15*).

These comparisons were documented in the NRC SER of January 4, 1983 (Franklin Research Center Report TER C5257-322) (*Reference 17*). Rochester Gas and Electric responded to this report in letters dated April 22, 1983 (steel structures) (*Reference 3*) and May 27, 1983 (concrete structures) (*Reference 11*). The comparison showed that, for tornado-related loadings, all required safety-related structures either were able to meet currently required factors of safety, were shown to meet margin-to-failure criteria through detailed calculations, or were to be provided with additional reinforcement as part of the Structural Upgrade Program. For seismic loadings, it was determined that all concrete code changes were acceptable, except for the shear walls in the diesel generator buildings. These walls were to be further evaluated in conjunction with the Structural Upgrade Program (see Section 3.8.2.1).

Seismic loadings for steel structures were not specifically analyzed by RG&E. Rochester Gas and Electric considers that the main structural elements were determined to be suitable by virtue of the overall Lawrence Livermore Laboratory analysis, documented in NUREG

CR-1821 (*Reference 18*), which was approved by the NRC (*Reference 19*). The steel code changes concerning coped beams, moment connections, and steel embedments will be evaluated relative to the extreme seismic loads and load combinations, in conjunction with the overall Structural Upgrade Program.

Scuppers were installed in accordance with the RG&E May 27, 1983, submittal the NRC (*Reference 11*).

3.3.5.6 Diesel Generator Component Operability

During the ACRS presentation, questions were raised concerning operability of diesel generator components (such as the day tank) due to the tornado differential pressure of 0.4 psi.

RG&E conducted an evaluation and concluded that no operability restrictions exist due to the expected 0.4 psi differential pressure.

3.3.5.7 Conclusions

Based on a review, audit, and plant inspection, the NRC concluded that the evaluation and resolution of SEP Topics III-2, Wind and Tornado Loadings; III-4.A, Tornado Missiles; III-6, Seismic Design Considerations; and III-7.B, Load Combinations, were acceptable. The NRC also concluded that the RG&E analysis and implementation of the Structural Upgrade Program were acceptable (*Reference 20*).

The following modifications and analyses are the principal ones accomplished as part of the Structural Upgrade Program.

A. All primary structural steel framing, including their connections and anchorages, found to be overstressed when subjected to the following design loads have been modified to resist these loads: 132mph tornado windspeeds and 100 psf extreme snow load. They have also been modified as necessary to maintain integrity for 188-mph tornado windspeeds. These modifications were included in the auxiliary building, turbine building, intermediate building, control building, and facade structure.

   The acceptance criteria for the steel components that have been upgraded for the 132-mph tornado loads, the severe snow and wind loads, and the extreme snow load is 1.6 S, where S is the required section strength based on elastic design methods and allowable stresses defined in AISC 1980. This applies to primary members, primary connections, and steel portions of primary anchorages (excluding the anchor bolts).

   The acceptance criteria for anchor bolts and the concrete portion of the anchorages that have been upgraded for the 132-mph. tornado loads, the severe snow and wind loads, and the extreme snow load are in accordance with ACI 349 Appendix B.

   The acceptance criteria for loads associated with the 188-mph tornado are that there is no loss of ultimate safety function.

   The following modifications have been completed in the intermediate building restricted area side:

   1. Low roof supports.
   2. Structural members on all levels.

B. Backdraft dampers were designed and installed in the auxiliary building north wall in order to eliminate the effects of differential pressures associated with the design-basis tornado. These dampers were only required in the auxiliary building. The backdraft dampers relieve the differential pressure caused by the tornado by means of automatic louvers that remain closed during normal operations. The louvers are seismically attached to the Seismic Category I auxiliary building structure; however, the louvers themselves are nonseismic. The louvers are designed to relieve a differential pressure of 0.4 psi at a pressure drop rate of 0.1 psi per sec. The louvers will open when air pressure outside the building is 0.4 psi less than the pressure inside. The louvers consist of six 3 ft x 6 ft panels for a total surface of 108 ft^2.

C. No exterior shell or secondary member modifications were required on the basis that their failure would not damage required safety equipment (*Reference 21*).

D. The required safe shutdown equipment is protected from tornado missiles.

E. As part of the review of SEP Topic III-7.B, the shear walls in the diesel generator building were reevaluated relative to seismic forces. The diesel generator building was modified as part of the Structural Upgrade Program to withstand wind and tornado loads, including missiles, severe weather, design flooding, and seismic loads.

F. Certain modifications or protection from potential damage due to block wall failure were provided for the main steam and feedwater piping and associated valves, main steam isolation valve control cables, and the spent fuel assemblies.

G. Operability restrictions of diesel generator components due to differential pressure effects was evaluated and found to be negligible.

H. The east wall of the relay room (part of the control building) has been protected as part of the Structural Upgrade Program by a structure that will withstand wind and tornado loadings, including missiles, extreme snow loads, design-basis flooding, and operating basis earthquake and safe shutdown earthquake loads. The east wall of the control room is capable of resisting these loads (*Reference 22*).

I. As part of SEP Topic III-7.B and as noted in an RG&E letter of August 19, 1983 (*Reference 23*), certain code changes concerning coped beams, moment connections, and steel embedments in all buildings have been evaluated relative to seismic loadings.

3.3.6 INTERMEDIATE BUILDING BLOCK WALL REINFORCEMENT

In compliance with NRC Order EA-12-049, Ginna has developed beyond design basis strategies to allow the station to cope following an extended loss of AC power (ELAP) coincident with an external event (Earthquake, Tornado, or External Flood). Ginna's current design basis safe-shutdown strategy following a seismic or tornado event requires the plant to reach Mode 3. However, NRC Order EA-12-049 requires Ginna to achieve cold shutdown (Mode 5) following each of these external events.

The Intermediate Building houses several components required to complete these strategies. In the Intermediate Building Basement, cable trays containing key instrument channels pass from the protection of the cable tunnel, a Seismic Category I and Tornado-Missile Protected, subterranean structure to the containment penetrations located at the northeast of containment. However, the cabling for these loops leaves the protection of the cable tunnel within the "cold side" of the intermediate building basement before reaching the containment penetration splice boxes.

Additionally, on the main steam header, the atmospheric relief valves must be locally operated to ensure a timely cool down and therefore needs to be protected.

Due to the relatively low capacity of the unreinforced Intermediate Building block walls, wall sections, required to maintain their structural integrity, have been reinforced to withstand the applied loads from tornado missiles or tornado winds, and capable of withstanding seismic loads. In general, the Intermediate Building block walls, in areas that require reinforcement, have been covered with ¼” steel plate attached to additional structural framing. The steel plate and framing prevent tornado missiles or sections of falling block wall from impacting equipment required to support the beyond design basis mitigating strategies.

REFERENCES FOR SECTION 3.3

1. U.S. Nuclear Regulatory Commission, Integrated Plant Safety Assessment, Systematic Evaluation Program, R. E. Ginna Nuclear Power Plant, NUREG 0821, December 1982.

2. U.S. Nuclear Regulatory Commission, Final Evaluation of SEP Topic II-2, Wind and Tornado Loadings, April 21, 1982.

3. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Structural Reanalysis Program, dated April 22, 1983.

4. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic III-4.A, Tornado Missiles, R. E. Ginna Nuclear Power Plant, dated April 16, 1982.

5. Electric Power Research Institute, Strength of Steel Plates Subjected to Missile Impact, Sixth Smirt Conference, NP-769, 1981.

6. Letter from J. E. Maier, RG&E, to H. R. Denton, NRC, Subject: Application for Amendment to the Operating License, Attachment B, dated January 18, 1984.

7. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report, Sections 4.8, 4.11, and 4.17.1, dated August 22, 1983.

8. Letter from J. Ebersole, ACRS, to N. J. Palladino, NRC, Subject: Recommendations of the Advisory Committee Regarding the Ginna Structural Reanalysis Program, dated April 9, 1984.

9. U.S. Nuclear Regulatory Commission, Technical Evaluation Report, NRC Docket 50-244, August 2, 1983.

10. Letter from R. C. Mecredy, RG&E, to C. Stahle, NRC, Subject: Structural Upgrade SER, R. E. Ginna Nuclear Power Plant, dated January 25, 1989.

11. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-7.B, Design Codes, Criteria, and Load Combinations, dated May 27, 1983.

12. American Institute of Steel Construction (AISC), Manual of Steel Construction, Eighth Edition, 1980.

13. American Institute of Steel Construction (AISC), Manual of Steel Construction, Sixth Edition, 1963.

14. American Concrete Institute (ACI), Code Requirements for Nuclear Safety-Related Structures, ACI 349-80.

15. American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, AIC 318-63.

16. American Society of Mechanical Engineers, Section III, Division 2, 1983.

17. Franklin Research Center, Technical Evaluation Report, Wind and Tornado Loading, TER-C5257-400, December 2, 1981.

18. Lawrence Livermore Laboratory, Seismic Review of the R. E. Ginna Nuclear Power Plant as Part of the Systematic Evaluation Program, NUREG/CR-1821.

19. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Seismic Design, Construction, and Component Integrity, dated January 7, 1981.

20. Letter from A. Johnson, NRC, to R.C. Mecredy, RG&E, Subject: Supplemental Safety Evaluation - Systematic Evaluation Program/Structural Upgrade Program at R. E. Ginna, dated November 15, 1989.

21. Letter from C. Stahle, NRC, to R. W. Kober, RG&E, Subject: Safety Evaluation Report on the Structural Upgrade Program, dated March 24, 1987.

22. Letter from R. W. Kober, RG&E, to C. Stahle, NRC, Subject: Structural Upgrade SER, R. E. Ginna Nuclear Power Plant, dated May 26, 1987.

23. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, dated August 19, 1983.

24. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Structural Reanalysis Program, SEP Topics, II-2.A, III-2, III-4.A and III-7.B, dated May 19, 1983.

Table 3.3-1
PRIMARY MEMBER FAILURES PER LOADING COMBINATION

Loading Combination[a]

Building	Member Type	Severe	Severe + Sn′	Severe + Sn′ + Wt (132)	Severe + Sn′ + Wt (188)	Severe + Sn′ + Wt (250)
Auxiliary	Columns	20	23	25	25	39
	Beams	21	22	24	26	38
	Bracing	6	14	14	15	36
	Truss	0	0	0	0	0
	Total	47	59	63	66	113
	Columns	17	19	19	20	60
	Beams	10	12	13	18	22
	Bracing	2	7	12	12	37
	Truss	4	7	10	15	25
	Total	33	55	54	65	144
Turbine, Control, Diesel	Columns	16	17	34	44	51
	Beams	5	5	6	6	7
	Bracing	57	72	87	104	154
	Truss	2	5	34	35	52
	Total	80	99	161	189	264
Screen House	Columns	2	2	2	2	31
	Beams	4	4	4	4	10
	Bracing	2	2	6	12	20
	Truss	0	0	9	28	109
	Total	8	8	21	46	170
Totals		168	211	299	366	691

a. See Section 3.3.2.1.6 for definition of loading combinations.

3.4 WATER LEVEL (FLOOD) DESIGN

3.4.1 FLOOD PROTECTION

3.4.1.1 Flood Protection Measures for Seismic Category I Structures

3.4.1.1.1 Introduction

The general plant grade at Ginna Station is about 270 ft msl, with the exception of the area between Lake Ontario and the turbine building where the grade level is at elevation 253 ft. The plant is protected from lake flooding by a breakwater with a top elevation of 261 ft, which prevents site flooding due to high water levels in the lake and lake storms from being a significant concern. The probable maximum flood originally considered in the design of Ginna Station was caused by Lake Ontario water and resulted in a flood level of 250.78 ft, later (1973) revised to 253.28 ft. During the Systematic Evaluation Program (SEP), flood protection from Deer Creek flooding was evaluated and a design flood level based on a Deer Creek discharge of 26,000 cfs was established (Section 2.4). The NRC staff considered this an acceptable level of protection, in conjunction with the Structural Upgrade Program (Section 3.8) and emergency procedures for installation of flood protection devices (*Reference 1*).

3.4.1.1.2 Lake Ontario Flood Protection

The 261-ft msl breakwater which protects the plant from lake flooding is a stone revetment constructed in two reaches. They are an approximately 420-ft long west reach and an approximately 400-ft long east reach. The east and west reaches are separated by the 20-ft wide circulating water discharge canal. The stone revetment was initially constructed with two layers of 5-ton minimum armor stones laid upon a 1.0 vertical to a 1.5 horizontal sideslope to a minimum elevation of 257.0 ft msl. Because of the high lake levels that were predicted for Lake Ontario during the early 1970s, the crest elevation of the revetment was raised to a minimum of 261.0 ft msl by placement of cap stone along the top of the revetment.

As part of SEP Topic III-3.C, the NRC staff reviewed the design of the revetment and concluded that the original revetment design was adequate. Also, the Army Corps of Engineers was requested by the NRC to provide a technical opinion of the adequacy of the existing revetment.

The Buffalo District Corps of Engineers reviewed the design of the revetment. After visiting the site to inspect the revetment they concluded that it appeared to be structurally sound and stable with no evidence of any major structure stability program; and based on its performance to date, the anticipated durability and survivability of the revetment as constructed should exceed the life of the plant (*Reference 2*). The Corps recommended that RG&E implement a monitoring program in order to detect future movement of the armor stone. RG&E implemented an inspection program which was reported to the NRC by *Reference 3*.

3.4.1.1.3 Deer Creek Flood Protection

A Deer Creek discharge of 26,000 cfs corresponds to an elevation of 273.8 ft msl on the west and south side of the auxiliary building (west channel flow), 272.0 ft on the north and east side (east channel flow), and 256.6 ft msl on the north yard at the turbine building and screen

house. R.E. Ginna agreed to provide protection to this level (Section 2.4.7). Portable flood barriers have been installed in the auxiliary building for use in the event of flooding from Deer Creek. The flood barriers consist of a panel with a pair of inflatable gaskets on the sides and across the bottom. The panels slide into frames installed around the auxiliary building personnel access doors and the rollup vehicle access door. Air flasks located in the auxiliary building are used to inflate the gaskets. When the flood barriers are not in use they are mounted on brackets on the wall next to the doors they serve, except the rollup door barrier, which is mounted next to the 1G fan.

Emergency procedures provide for installation of the flood barriers and for connection of the alternative cooling water supply to the diesel generator (Section 9.5.5), assuming service water will be lost as a result of flooding of the screen house. The emergency procedures are to be instituted prior to the Deer Creek discharge flow reaching 10,000 cfs which corresponds to approximately 7.4 ft above the bridge level on the access road crossing Deer Creek to the station. The procedures conservatively institute the flood protection when the water rises above the handrails on the bridge.

The diesel generator building is protected from flooding from Deer Creek at a flood flow of 26,000 cfs by watertight doors in the building north wall.

3.4.1.2 Permanent Dewatering System

Ginna Station does not have a permanent dewatering system. The design-basis ground water level used in the original design of Ginna Station was 250 ft msl, which is approximately 20 ft below grade at the upper portion of the station. A ground water monitoring program was implemented from 1983 through 1987 to verify the design-basis ground water level and, as a result, the design-basis ground-water level was revised to 265.0 ft msl. It was determined that below grade safety-related structures were designed to withstand ground-water levels at grade (270.0 ft msl). See Section 2.4.10.1.

The Ginna design provides for no backfill against the containment wall. The excavation around the major portion of the vessel is graded to ensure slope stability of the in-place material under all conditions. At a limited portion of the circumference where grade level is maintained adjacent to the containment, there exists a retaining wall spaced 2 ft to 6 ft clear of the containment wall designed specifically to resist all earth pressure due to backfill. No provision is made to prevent ground water from penetrating the void created between the retaining wall or earth and the containment wall. The opening between the retaining wall and the containment wall is covered with a concrete slab to ensure that the void is not filled with debris. Where the exterior walls of the containment are exposed to ground water, the walls from the edge of the ring girder up to elevation 235 ft are waterproofed with a bitumastic membrane system reinforced with glass fibers. In addition, prior to the application of the membrane courses, the angle at the intersection of the wall and ring girder was further reinforced with glass fabric.

3.4.2 FLOODING DUE TO FAILURE OF TANKS

In the SEP Integrated Plant Safety Assessment Report (NUREG 0821), Topic IX-3, Section 4.25.3, the NRC staff expressed a concern that failure of tanks in the auxiliary building could

flood out safety-related equipment in the lower levels of the building. An RG&E evaluation determined that the total volume of all nonseismic tanks in the auxiliary building was 208,703 gal. The evaluation showed that, based on the 70,000-gal capacity of the residual heat removal pit (i.e., the lowest point in the building), and the net free surface area of the auxiliary building basement of 4813 ft^2, the failure of nonqualified tanks in a seismic event would result in a water level of 3 ft 10 in.

Loss of both residual heat removal pumps had been previously evaluated in conjunction with the fire protection review and it was determined that the plant could achieve and maintain MODE 5 (Cold Shutdown) conditions utilizing alternate methods (*Reference 4*). However, the water level resulting from a failure of all non-qualified tanks would be greater than the height of required safe shutdown equipment. As a result, RG&E qualified the three chemical and volume control system holdup tanks and the waste holdup tank to Seismic Category I. Therefore, the resulting maximum water volume which could be discharged onto the auxiliary building floor in the event of failure of the remaining nonqualified tanks is 93,803 gal. This would result in a maximum water level of only 8 in., which is below the elevation of the bottom of the safety injection pump motor of 20 in. With the qualification of these tanks, the NRC staff determined that the issue of internal flooding due to seismic qualification of tanks was adequately resolved for the Ginna plant (*Reference 5*).

The vendor supplied demineralization system in the auxiliary building (Section 11.2.2.13) was evaluated for its potential effects on plant flooding. For the purposes of the auxiliary building flooding analysis, this system resulted in a maximum water volume increase of 0.2 in., which would result in a maximum water level of 8.2 in. Since this new calculated maximum water level is below the 20-in. elevation of the bottom of the safety injection pump motor, the basis for the acceptance of the flooding analysis has not been changed.

The reactor water makeup tank and the two monitor tanks were not seismically qualified per the original plant design. These three tanks have been modified to add seismically qualified structural reinforcement which eliminates their contribution to the estimated flooding volume.

3.4.3 ROOF DRAINAGE

The low roof sections of the intermediate and auxiliary buildings; the control building, diesel generator building, and screen house roofs; and the turbine building parapets have been provided with scuppers designed to ensure that any rainwater, resulting from a design-basis storm, would not accumulate on the roofs and cause damage. The scuppers are located so that their outflow will not damage any surrounding plant structures or equipment. The flow from the scuppers will not discharge on equipment or structures required for safe shutdown.

The design-basis storm is a 24-hour rainfall totaling 19.17 in. of rain, with a 1-hour maximum of 6.11 in. The combined flow of all the scuppers on each roof is designed to handle at least this flow.

Generic Letter (GL) 89-22 (*Reference 6*) informed licensees that higher rainfall intensities over shorter time intervals and smaller areas should be considered. As part of the RG&E Individual Plant Examination for External Events (IPEEE) submittal, RG&E calculated the

roof de-watering capabilities relative to the revised probable maximum rainfall (*Reference 7*). As a result of this analysis (*Reference 8*), the Control Building roof de-watering capabilities were modified to ensure design roof loads would not be exceeded in the event the new probable maximum rainfall were to occur.

The design maximum level the rainfall is allowed by the scuppers to accumulate on the roofs is 1.6 ft. This depth of rainfall would produce a load of approximately 100 lb/ft^2, which is equal to the maximum winter precipitation for a storm with a probability of 1×10^{-4} recurrence interval (SEP Topic II-2A). A 100 lb/ft^2 load was found in the structural upgrade program to be the maximum load the roofs could support without effecting the margins of safety of the structures.

REFERENCES FOR SECTION 3.4

1. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report, Section 4.5, Plant Flooding by Deer Creek, R. E. Ginna Nuclear Power Plant, dated August 19, 1983.

2. Letter from G. P. Johnson, Corps of Engineers, to Project Officer, NRC, Subject: R. E. Ginna Nuclear Generating Plant, Town of Ontario, NY, dated December 10, 1981.

3. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Draft NUREG 0821, R. E. Ginna Nuclear Power Plant, dated August 25, 1982.

4. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Fire Protection Rule, dated April 11, 1983.

5. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report (IPSAR) Section 4.25.3, Flooding Due to Failure of Tanks, R. E. Ginna Nuclear Power Plant, dated July 8, 1983.

6. Generic Letter 89-22, Potential for Increased Roof Loads and Plant Area Flood Runoff Depth at Licensed Nuclear Power Plants Due to Recent Change in Probable Maximum Precipitation Criteria Developed by the National Weather Service, dated October 19, 1989.

7. Letter from R.C. Mecredy, RG&E, to G.S. Vissing, NRC, Subject: Individual Plant Examination for External Events (IPEEE), High Winds, External Floods and Transportation Accidents, dated December 21, 1998.

8. Design Analysis, DA-CE-98-153, Revision 1, Roof Scupper Capacity Check, dated September 22, 1999.

3.5 MISSILE PROTECTION

3.5.1 INTERNALLY GENERATED MISSILES

3.5.1.1 Introduction

3.5.1.1.1 Design Criteria

Systems containing hot pressurized fluids are carefully checked for potential sources of missiles where such missiles could be directed toward engineered safety features. Suitable engineering and quality control are applied to the design, manufacture, and installation of components to prevent the generation of missiles where such missiles could adversely affect the intended functioning of engineered safety features.

Thus, a design criterion is that components of the pressurized systems defined above are not missile sources. Prevention of missiles is accomplished by identifying all potential sources, investigating to ensure design adequacy in preventing missile generation, redesigning where the investigation discloses inadequate safety margins for missile prevention, and providing a suitable quality assurance program to avoid unanticipated deficiencies and ensure that design margins are preserved.

3.5.1.1.2 Systematic Evaluation Program

As part of the Systematic Evaluation Program (SEP Topic III-4.C), a detailed review of internally generated missile effects was conducted.

Missiles which are generated internally to the reactor facility (inside or outside containment) may cause damage to structures, systems, and components that are necessary for the safe shutdown of the reactor or for accident mitigation or may cause damage to the structures, systems, and components whose failure could result in a significant release of radioactivity. The potential sources of such missiles are valve bonnets and hardware retaining bolts, relief valve parts, instrument wells, pressure containing equipment (such as accumulators and high-pressure bottles), high speed rotating machinery, and rotating segments (i.e., impellers and fan blades). Turbine missiles are addressed in Section 3.5.1.2.

The acceptability of the design of structures, systems, and components for protection against internally generated missiles is based on meeting General Design Criterion 4. Additional guidance is contained in Regulatory Guide 1.13, Spent Fuel Storage Facility Design Basis, Revision 1, December 1975, and Regulatory Guide 1.27, Ultimate Heat Sink for Nuclear Power Plants, Revision 2, January 1976.

Systems and components needed to perform safety functions (safe shutdown or accident mitigation) are listed below and discussed in Section 3.5.1.3.

- Reactor coolant system.
- Emergency Core Cooling System (ECCS).
- Containment heat removal and atmosphere cleanup systems.
- Chemical and volume control system (some portions).

- Residual heat removal system.
- Component cooling water (CCW) system.
- Service water (SW) system.
- Diesel-generator auxiliary systems.
- Main steam system (some portions).
- Feedwater and condensate systems (some portions).
- Auxiliary feedwater systems.
- Standby auxiliary feedwater system.
- Ventilation systems for vital areas.
- Combustible gas control system.
- Refueling water storage tank (RWST).

Systems whose failure may result in release of unacceptable amounts of radioactivity are as follows:

- Spent fuel pool cooling and cleanup system.
- Sampling system.
- Waste disposal system.
- Containment purge system.
- Instrument and service air systems.

Additionally, electrical systems that are necessary to support those fluid systems needed to perform safety functions are noted in the following list.

- Diesel generators.
- Station batteries.
- 480-V switchgear and relay rooms.
- Control room.
- Cable spreading room.

Based on a safety review pursuant to SEP Topic III-4.C (*Reference 1*), the NRC staff has concluded that the design of Ginna Station for protection from internally generated missiles meets the intent of General Design Criterion 4 and the guidance from Regulatory Guides 1.13 and 1.27.

3.5.1.2 Turbine Missiles

3.5.1.2.1 Introduction

Failure of turbine disks and rotors can result in high-energy missiles that have the potential for resulting in damage to plant safety features. There are two areas of concern:

Design overspeed failures.

These are related to the material quality of the turbine disks and rotors, inservice inspection for flaws, and chemistry conditions that could lead to stress-corrosion cracking.

Destructive overspeed failures.

These are related to the reliability of the electrical overspeed protection system, the reliability of and the testing program for turbine stop valves and turbine control valves, and the inservice inspection of these valves.

The purpose of evaluating the potential for turbine missiles is to ensure that all structures, systems, and components important to safety either have adequate protection by means of structural barriers or have an acceptably low probability of damage. Criteria for evaluating missile protection are contained in General Design Criterion 4. Additional guidance is contained in Regulatory Guide 1.115, Protection Against Low Trajectory Turbine Missiles, Revision 1, July 1977; and Regulatory Guide 1.117, Tornado Design Classification, Revision 1, April 1978.

3.5.1.2.2 Turbine Inspection Program

Low-pressure turbine disk cracking in Westinghouse turbines has been experienced at several operating plants. As a result, an RG&E turbine inspection program (*References 2* and *3*) was developed to provide an acceptably high degree of assurance that turbine disks will be inspected before cracks can grow to one-half the size that could cause disk failure at speeds up to the design speed (see Section 10.2.3.4). Ginna LLC performs testing of the turbine overspeed protection system to provide assurance that the system will remain operable and thereby limit the likelihood of overspeed beyond design conditions (*Reference 7*), based on the criteria in WCAP-11525 and WCAP-11529 (*Reference 6*). These tests are described in Section 10.2.3.4.4.

The turbine supervisory instrumentation monitors turbine vibration, eccentricity, and differential thermal expansion and alarms abnormal conditions (Section 10.2.1.4).

3.5.1.2.3 Systematic Evaluation Program Topic III-4

All the systems needed for the safe shutdown of the plant are either inside or shadowed by the concrete containment building, located below the turbine pedestal, or are out of the turbine low trajectory missile strike zones. In addition, many of the systems have physically separated redundant components. On this basis, the NRC staff, in the Safety Evaluation Report (SER) for SEP Topic III-4.B considered that the probability of a low trajectory missile striking any of the safety-related systems is acceptably low.

The probability of turbine high trajectory missiles striking the safety-related systems is obtained by multiplying the conservatively estimated turbine failure and missile ejection rate, 10^{-4} per year, by the strike probability density per turbine failure, 10^{-7} per ft^2, and by the horizontal area occupied by the systems. A conservative estimate of the area occupied by these systems is 12,000 ft^2. The turbine failure rate of 10^{-4} is also conservative because of the use of a historically observed turbine failure data set. Some of the reported failures involved

old turbine designs and fabrication techniques which have been improved in currently produced turbines (a new turbine rotor was installed at the Ginna plant during the 1979 MODE 6 (Refueling) outage). The resulting probability of high trajectory missile strikes is found to be on the order of 10^{-7} per year, and the total strike probability from low and high trajectory missiles is conservatively estimated to be less than 10^{-6} per year.

Based on the above figures, in the SER for SEP Topic III-4.B, the NRC staff considered that the overall probability of turbine missiles damaging Ginna Station and leading to consequences in excess of 10 CFR 100 exposure guidelines is acceptably low (*Reference 4*).

Due to plant uprate, the maximum allowable turbine overspeed setpoint was reduced from 110% to 109.3%. The overspeed setpoint is used to ensure that the maximum turbine overspeed does not exceed 120% of turbine design speed (1800 rpm).

3.5.1.3 Effects of Internally Generated Missiles on Systems and Equipment

<u>3.5.1.3.1 Systems Needed to Perform Safety Functions</u>

3.5.1.3.1.1 Reactor Coolant System

The reactor coolant system serves as the pressure retaining boundary for the reactor coolant and is comprised of a reactor pressure vessel and two parallel heat transfer loops. Each loop contains one steam generator and one pump, connecting piping, and instrumentation. The pressurizer and associated safety and relief valves are connected to one of the reactor hot legs via the surge line. Pressurizer spray lines and associated valves are connected to the top of the pressurizer from one of the reactor coolant cold legs. The purpose of the pressurizer is to maintain primary coolant pressure and compensate for coolant volume changes as the heat load changes. All components of the primary coolant system are located within the containment building. Overpressure protection is provided to ensure the coolant system pressure does not exceed design limits.

The reactor closure head and the reactor vessel flange are joined by forty-eight 6-in. diameter studs. It is unlikely that any of the studs would become a missile since they are not subjected to direct reactor pressure and, therefore, are not exposed to sufficient pressure to create an accelerating force sufficient to cause them to become missiles.

The pressurizer safety and relief valves, which are mounted atop the pressurizer, have the potential for becoming missiles. However, the position of the pressurizer within a concrete compartment is such that any missiles generated because of a failure of these valves would not be likely to damage other components or piping of the reactor coolant system. All valves on the pressurizer spray line are located within the loop or pressurizer compartments, and thus would not be expected to damage any safety-related equipment in the event of a valve failure.

In 1995, the three missile shield blocks on top of the pressurizer compartment were reconfigured from the original design to allow air flow through the compartment. This modification was supported by an evaluation which determined that the repositioned blocks would still protect vital equipment in the containment from the effects of internally generated

missiles and released high energy fluid or steam should a piping failure occur in the pressurizer compartment.

Control rod drive assemblies are mounted on the top of the reactor vessel and are considered an extension of the reactor vessel head. A 1.25-in. thick steel missile shield is placed over the control rods during operation as protection against missile damage to safety systems caused by impacting control rod drives or reactor vessel head studs.

Instrumentation requires some penetration into the reactor coolant system. These penetrations are small and generally take the form of welded wells. Because of their size and orientation, serious damage to the reactor coolant system is highly unlikely.

The possibility that missiles may result from destructive overspeeding of one of the primary coolant pumps in the event of a pipe break in the pump suction or discharge was also reviewed. Potentially damaging impeller missile ejection from the broken pipe is minimized by a massive steel pump casing. Generation of missiles from overspeed of the motor, flywheel, and impeller of the reactor coolant pump is addressed in Section 5.4.1.

The two steam generators have manways held in position by studs on the primary and secondary sides of the shell. These small diameter studs are subject only to stored elastic energy and thus are not considered to be credible missiles.

In summary, relative to the reactor coolant system, the likelihood of missile generation and resultant damage is minimized by equipment design features, component arrangement, and compartmentalization.

3.5.1.3.1.2 Emergency Core Cooling System (ECCS)

The Emergency Core Cooling System (ECCS) serves as the means of injecting water for core protection in the event of reactor coolant system water loss. The Emergency Core Cooling System (ECCS) is comprised of the high-pressure safety injection system, the residual heat removal system (for low-pressure safety injection), and accumulator tanks. High-pressure safety injection flow and accumulator flow are directed to the reactor coolant system through the two cold-leg reactor inlet pipes. The high-head system consists of three pumps, each rated at 300 gpm. Two passive accumulator tanks containing borated water, pressurized with nitrogen to 700 psig, are provided inside the containment building. The residual heat removal system injects directly into the reactor vessel upper plenum via two nozzles on opposite sides of the vessel. The low-head residual heat removal system consists of two pumps, each rated at 1560 gpm.

The suction source of water for the high-head pumps is the refueling water storage tank (RWST). The refueling water storage tank (RWST) is not missile protected; however, the only internally generated missiles that could potentially affect the tank would originate at component cooling water (CCW) system and service water (SW) system valve locations. Both of these systems are low-pressure, cold water systems with insufficient internal energy to generate any missiles of consequence.

The high-pressure and low-pressure piping systems are separated from each other outside containment, taking suction from opposite sides of the refueling water storage tank (RWST).

One train of each of these systems is routed together in the auxiliary building. The redundant trains of these systems are routed separately. Once inside the containment, separation of the individual injection lines is provided. Each train of the residual heat removal and high-pressure safety injection piping is routed in opposite directions inside the containment. Injection headers are located outside the missile barriers. Individual injection lines connected to the injection headers pass through the missile barriers and then connect to the reactor coolant system.

The most likely sources of missiles in the Emergency Core Cooling System (ECCS) are the residual heat removal and high-pressure safety injection pumps. The high-pressure safety injection pumps are 350-hp horizontal multistage centrifugal pumps operating at 3550 rpm. The residual heat removal pumps are 200-hp horizontal single-stage centrifugal pumps operating at 1770 rpm. These pumps have a thick steel casing, making it highly improbable that a source of missiles, such as a broken impeller, would penetrate the casing to cause any damage.

The residual heat removal pumps are located in the residual heat removal pit, separated from other safety-related equipment. During MODES 1 and 2, the portions of the system upstream of the isolation valves are isolated from the high-pressure reactor coolant system, and are therefore not subjected to forces which might cause a missile to be generated. If a missile were generated as a result of pump failure during normal reactor shutdown, it would affect only the residual heat removal system. The residual heat removal system could be isolated and the reactor maintained in a stable shutdown condition, using the steam generators, until repairs could be made.

The high-pressure safety injection system is also normally cold and not at sustainable high pressure. With these conditions a leak or break would not result in significant thrust forces. Thus, it is not expected that missiles would be generated. Pressure boundary valves, which are subject to high pressure, have backseats which should prevent missile generation.

Two accumulators, located on separate sides of the containment are situated behind the steam generator missile shielding. Accumulator missile sources are not oriented towards any other safety-related equipment.

Because of the functional design features, separation, and component design provisions of the Emergency Core Cooling System (ECCS), the system will be capable of performing its intended functions considering internally generated missile sources as discussed above.

3.5.1.3.1.3 Containment Heat Removal and Atmosphere Cleanup Systems

The containment heat removal and atmosphere cleanup systems consist of two independent systems: the containment air recirculation system, and the containment spray system. The containment air recirculation system consists of four fans and heat exchangers, as well as two charcoal filter units. The containment spray system consists of two spray pumps, with associated piping, ring headers, and nozzles. The source of water for the containment spray system is the refueling water storage tank (RWST).

The four containment fan cooler units are positioned in pairs on opposite sides of the containment. Because of this separation, it is unlikely that a single missile could cause failure of more than one pair of these units. The spray system headers and nozzles are split into redundant trains. The spray nozzles are located high inside containment. Therefore, it is not likely that any missiles would reach these components. Should a number of the nozzles be damaged, containment cooling would still be provided using nozzles in the redundant train and by the fan cooler units.

The spray system pumps are located in the auxiliary building, near the high-pressure safety injection pumps. However, the orientation of the high-pressure safety injection pumps to the spray pumps is such that damage to the spray pumps is highly improbable in the unlikely event of missile generation from any of the high-pressure safety injection pumps. There are no high energy lines in this vicinity that could be a source of internally generated missiles. Further, the spray system itself is not under pressure during MODES 1 and 2. It is therefore concluded that no failure due to internally generated missiles is expected for the containment spray system.

The containment heat removal and atmosphere cleanup systems, considering their redundant features and separation, will be capable of performing their design function from the standpoint of internally generated missiles.

3.5.1.3.1.4 Chemical and Volume Control System

The chemical and volume control system controls and maintains reactor coolant system inventory and purity through the process of makeup and letdown, and provides seal injection flow to the reactor coolant pump seals. The letdown portion of the system consists of a regenerative heat exchanger and a nonregenerative heat exchanger to cool the reactor coolant letdown and three parallel orifice valves to reduce the pressure. The coolant is passed through purification and deborating demineralizers, as necessary, where corrosion and fission products are removed. The coolant is then routed to the volume control tank. Seal return flow passes from the reactor coolant pump seals, through a containment isolation valve and the seal-water heat exchanger, before returning to the volume control tank.

The seal return line is at low pressure and temperature. The charging pumps draw from the volume control tank and inject into the reactor coolant system, both through the normal makeup path and via the reactor coolant pump seals.

Borated water from the boric acid storage tanks can be added to the reactor coolant system by injection from the charging pumps. The boric acid storage tanks are protected from internally generated missiles by virtue of their location within concrete cubicles.

The most likely source of missiles in the chemical and volume control system would be generated in the letdown line and charging line on the reactor coolant system side of the regenerative heat exchanger, in portions of the chemical and volume control system connected directly to the reactor coolant system, and in the chemical and volume control system letdown piping up to the nonregenerative heat exchanger.

The only equipment that needs to be considered with respect to potential missiles from the chemical and volume control system letdown line is selected cable trays; however, potential missile sources are located remotely from safety-related cable trays.

Valve stems are the only potential missile sources associated with the charging line inside containment and the letdown line outside containment. However, the valves all have backseats and would not be expected to be a source of missiles. There are no other potential missile sources in the vicinity of these portions of the chemical and volume control system. The chemical and volume control system is adequately protected from the effects of internally generated missiles.

3.5.1.3.1.5 Residual Heat Removal System

This system is discussed as part of the low-pressure safety injection portion of the Emergency Core Cooling System (ECCS) in Section 3.5.1.3.1.2.

3.5.1.3.1.6 Component Cooling Water System

The component cooling water (CCW) system is a closed system with two motor-driven pumps rated at 150 hp and 2980 gpm, and two shell and straight tube heat exchangers. Heat transferred to the component cooling water (CCW) system is removed by the service water (SW) system and released into Lake Ontario.

The component cooling water (CCW) system removes heat from the residual heat removal heat exchangers, engineered safety features pump seals and jackets, chemical and volume control system and sampling heat exchangers, reactor coolant pump seals, bearings and motors, reactor support cooling pads, waste gas compressors, and the items in the waste and boric acid systems.

This system would be an unlikely source of missiles due to its low operating temperature and pressure. Other potential missile sources near the component cooling water (CCW) system have not been identified. However, if a missile were to cause a failure of the component cooling water (CCW) system, residual heat removal could be accomplished via the preferred auxiliary feedwater system and steam generators until repairs to the component cooling water (CCW) system could be made.

The component cooling water (CCW) system is adequately protected from internally generated missiles.

3.5.1.3.1.7 Service Water System

The service water (SW) system consists of four 5300-gpm capacity vertical motor-driven pumps located in the screen house. The original motors installed on the service water (SW) pumps were rated at 300-hp. The motors were replaced between 1995 and 1997 with 350-hp motors. The system is designed such that there are two redundant safety-related trains, each capable of supplying one set of required safety-related equipment. As a result of plant uprate two (2) Service Water (SW) pumps from either train are required to provide the necessary safe shutdown and postaccident safety functions for design bases LOCA. These pumps,

located approximately 7 ft apart, take suction from and discharge to the ultimate heat sink (Lake Ontario).

The system piping is routed underground from the screen house to the other structures. The two service water (SW) headers can be tied together via normally closed redundant manual valves. Separation of safety and nonsafety loads is provided via redundant isolation valves. The service water (SW) pumps are not considered likely sources of missiles due to their enclosure (casing) and submergence in the service water (SW) pump bay, and their low operating speed and pressure.

Also located in the screen house is one diesel-driven and one motor-driven fire pump. These pumps are not normally in operation and therefore are considered unlikely sources of internally generated missiles.

There are no potential sources of missiles in the vicinity of the service water (SW) system as the piping enters the various buildings, with the exception of that portion which enters the intermediate building. In this building, the only high-pressure system in the vicinity of the service water (SW) system is the steam generator blowdown system.

The service water (SW) system meets the requirements for protection from internally generated missiles.

3.5.1.3.1.8 Diesel-Generator Auxiliary Systems

The two diesel generators are located in separate diesel-generator rooms, located off the north side of the turbine building. These are low speed engines with no high-pressure hydraulic systems.

Due to separation of redundant portions of the system, and the segregation of the system as a whole, the system meets the design requirements with respect to internally generated missiles.

3.5.1.3.1.9 Main Steam System

The main steam system consists of two steam generators with two steam lines which connect in the intermediate building prior to entering the turbine building. Each steam line has four main steam safety valves, an atmospheric dump valve, a steam admission valve to the turbine-driven auxiliary feedwater pump (TDAFW), a main steam isolation valve, and a nonreturn valve, all located in the intermediate building, upstream of the junction of the two lines.

The main steam lines are of heavy walled construction, and are unlikely to be damaged by internally generated missiles. The main steam components are routed in a fashion so as to utilize plant structures for missile protection. Should a missile cause damage to the main steam system downstream of the isolation valve, the valve would close and the plant would shut down. If damage occurs either to the isolation valve or upstream of the valve, safe shutdown can be accomplished. A steam line break accident has been evaluated in Section 15.1.5.

The main steam system will be capable of performing its design function, considering internally generated missiles.

3.5.1.3.1.10 Feedwater and Condensate Systems

The main feedwater system consists of two motor-driven feedwater pumps which deliver water to the steam generators. Condensate from the hotwell is pumped by three 50% capacity motor-driven condensate pumps, through the hydrogen coolers, air ejectors, gland steam condenser, and then through several stages of preheating. The feedwater then passes into the containment and into the steam generators. The only area of concern for this system is that portion between the main feedwater isolation valves and the steam generators.

Due to the protection afforded by surrounding equipment, missile damage to this portion of the feedwater system is unlikely. However, if damage to this area were to occur, the preferred auxiliary feedwater system or the standby auxiliary feedwater system (SAFW) could provide the necessary feedwater flow to the second steam generator in order to effect safe shutdown.

No additional protection is needed for the feedwater and condensate systems to protect them from the effects of internally generated missiles.

3.5.1.3.1.11 Preferred Auxiliary Feedwater System

The preferred auxiliary feedwater system consists of two 100% capacity motor-driven auxiliary feedwater pumps (MDAFW), each directing flow to one steam generator, and a 200% capacity turbine-driven auxiliary feedwater pump (TDAFW), which directs flow to both steam generators. The design flow of the motor-driven pumps is 200 gpm; the turbine-driven pump is 400 gpm. The primary suction source of the pumps is from the condensate storage tanks. If necessary, the service water (SW) system will provide an unlimited water supply to these pumps.

The most likely source of missiles would be from the pumps. The turbine-driven pump is separated from the motor-driven pumps by a concrete enclosure/barrier. Separation is provided such that a postulated missile will not damage both trains associated with the motor-driven pumps. Therefore, in the unlikely event that a missile is generated, each train of the system is sufficiently separated to ensure system performance.

However, in the event that the preferred auxiliary feedwater system becomes unavailable due to a missile strike, the standby auxiliary feedwater system (SAFW) is capable of delivering the required feedwater flow to the steam generators to safely shut down the plant. No additional missile protection is needed for the preferred auxiliary feedwater system.

The preferred auxiliary feedwater system, through redundancy and separation, meets the design requirements with respect to internally generated missiles.

3.5.1.3.1.12 Standby Auxiliary Feedwater System (SAFW)

The standby auxiliary feedwater system (SAFW) consists of two 100% capacity pumps and piping which directs the flow from one pump to one steam generator. A cross-connect would allow each pump to feed either steam generator. The system would be used only in the event of a failure of the preferred auxiliary feedwater system. The standby auxiliary feedwater system (SAFW) is remotely located from the preferred auxiliary feedwater system such that

a failure in the preferred auxiliary feedwater system would not affect the ability of the standby auxiliary feedwater system (SAFW) to safely shut down the plant.

The standby auxiliary feedwater system (SAFW) needs no additional protection against the effects of internally generated missiles.

3.5.1.3.1.13 Ventilation Systems for Vital Areas

As part of the original design, safety-related pump motor coolers provide ducted air, cooled by service water (SW), to the rooms which contain the safety injection and containment spray pump motors, and to the residual heat removal pump and charging pump rooms. In 1992, service water (SW) to the room coolers for the safety injection and containment spray pump motors was blanked off (see Section 9.4.9.1).

The control room is air conditioned by its own ventilation system that is described in Section 6.4.

Ventilation for the two battery rooms is provided by an independent air conditioning system. This system takes suction from the air handling room and discharges from the battery rooms through the turbine building to the outside.

The ventilation systems are low-pressure systems, and therefore are not considered to be sources of potential missiles. There are no sources of missiles in the vicinity of the control room, battery room, or pump room ventilation systems. Though ductwork can be penetrated by missiles, the total cooling capability is not lost for any area and time is available for action to restore adequate ventilation.

The ventilation systems for vital areas will be capable of performing their design function, considering internally generated missiles.

3.5.1.3.1.14 Combustible Gas Control System

Redundant hydrogen recombiners located on opposite sides inside the containment have been provided. Since the hydrogen recombiner is not normally in operation, it is not considered to be a source of missiles. The system is not needed to shut the plant down. Should a missile strike the system, its repair could be scheduled in a timely manner so as not to interfere with plant operation. The NRC has removed from the Ginna Technical Specifications the requirements related to hydrogen recombiners and hydrogen monitors (*Reference 8*).

3.5.1.3.2 Systems Whose Failure May Result in Activity Release

3.5.1.3.2.1 Spent Fuel Pool Cooling System

The spent fuel pool (SFP) cooling system is designed to remove heat from the spent fuel pool (SFP), which is generated by stored spent fuel. The system was originally designed as a single train system, consisting of a pump, demineralizer, filter, and heat exchanger. See Section 9.1.3.1 for an update of the system configuration. Heat is removed from the system by the service water (SW) system.

The spent fuel pool (SFP) cooling system is a low-pressure system and is unlikely to generate missiles. The system arrangement is such that the spent fuel pool (SFP) itself could not be damaged. If the spent fuel pool (SFP) cooling system was damaged, the large thermal capacity of the pool would maintain temperatures below design (180°F) for many hours. As a means of alternate cooling, six (6) spent fuel pool (SFP) cooling loop options, as listed in the Technical Requirements Manual (TRM), provide 100% cooling capability (under normal operation) before any excessive heatup occurs.

The spent fuel pool (SFP) cooling system is capable of performing its function, considering internally generated missiles.

3.5.1.3.2.2 Sampling System

The sampling system provides samples for laboratory analysis to evaluate reactor coolant, feedwater steam system, and other reactor auxiliary systems during MODES 1 and 2. Samples are routed in an area away from other required safety-related equipment and into a separate room. Shielding is provided for the sampling lines. The likelihood of missiles causing damage to the sampling lines is very small. The sampling system meets the design requirements with respect to internally generated missiles.

3.5.1.3.2.3 Waste Disposal System

The entire waste disposal system is a low-pressure system, and is thus an unlikely source of missiles. The most likely sources, the gas decay tanks, are separated from other safety-related systems. The failure of a gas decay tank is a design-basis event which has been analyzed. Resultant doses are within allowable limits.

In addition, missile damage to other portions of the system will not affect the safe shutdown of the facility. This system is adequately protected from the effects of internally generated missiles.

3.5.1.3.2.4 Containment Shutdown Purge System

The containment shutdown purge system is provided to purge the containment during cold or MODE 6 (Refueling) shutdown. The system consists of ductwork, dampers, fans, and filters. The normal operating pressure of this system is low, and therefore this system is considered an unlikely source of missiles. Ductwork and components are routed away from potential missile sources. If missile damage were to occur, ample time to perform repairs would be available. The missile protection provided for the system is, therefore, acceptable.

3.5.1.3.2.5 Instrument and Service Air Systems

The instrument and service air systems consist of four air compressors (three instrument air, one service air), four aftercoolers, four air receivers as well as air dryers, prefilters, and filters. Two instrument air compressors are of the vertical type, with the use of oil-free cylinder construction. The third instrument air compressor and the service air compressor are two stage oil free rotary screw air compressors. The instrument air systems are cooled by the service water (SW) system. The service air compressor is air cooled.

The air systems are not safety-related. All equipment controlled by the air systems is either not required to operate for safe shutdown or accident mitigation, or fails in the safe position upon loss of air.

The air systems are low-pressure systems which operate between 115 psig and 125 psig. The greatest potential missile generators are the air compressors and air receivers. However, these components are located in the turbine building away from safety-related equipment.

The instrument and service air systems are not required to perform safety-related functions, and the design, with respect to internally generated missiles, will not prevent safety-related systems from performing their design functions.

3.5.1.3.3 Electrical Systems

The effects of missile generation on cabling, cable trays, instrumentation, and control panels associated with systems needed to perform safety functions were also evaluated during review of the systems discussed above.

3.5.1.3.3.1 Diesel Generators

See Section 3.5.1.3.1.8.

3.5.1.3.3.2 Station Batteries

The two station batteries are in separate rooms, both of which are located away from potential missile sources. Should a missile originate from the batteries themselves, the walls that separate the two rooms will prevent missile penetration. The separate rooms for the two station batteries provide adequate protection from internally generated missiles.

3.5.1.3.3.3 480-Volt Switchgear

Two 480-V load centers comprise the engineered safety features electrical system. The load centers are located in separate rooms, on different floors within the auxiliary building. There are no piping or pressurized sources near these rooms which could pose a potential missile source. Therefore, adequate protection from internally generated missiles has been provided.

3.5.1.3.3.4 Control Room

Piping, pressurized sources, or rotating machinery are not located within the control room. Ventilation ductwork is routed into the control room. Damaging missiles from the ventilation system are considered unlikely. There are no missile sources which could affect the proper functioning of the control room.

3.5.1.3.3.5 Cable Spreading/Relay Room

The cable spreading room (or relay room) does not contain any piping or other pressurized sources, or rotating equipment which might produce missiles. The fire protection system in this room is low pressure and thus is not capable of generating damaging missiles. There are no potential missile sources in this area that could affect safety functions.

3.5.2 EXTERNALLY GENERATED MISSILES

3.5.2.1 Tornado Missiles

Ginna Station has been assessed (SEP Topic III-4.A) to determine the ability of the plant to withstand the impact of tornado missiles. The purpose of the assessment was to verify that structures, systems, and components necessary to ensure (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe shutdown condition, and (3) the capability to prevent accidents that could result in unacceptable offsite consequences, can withstand the impact of a spectrum of tornado missiles.

Criteria for evaluating missile protection are in General Design Criterion 4. Additional guidance on tornado missiles is contained in Regulatory Guide 1.117, Tornado Design Classification, April 1978, and Regulatory Guide 1.78, Assumptions for Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release, June 1974.

As noted in Section 3.3, the design-basis tornado at Ginna Station has a maximum wind speed of 132 mph. For this wind speed, the design-basis missile is a steel rod, with 1-in. diameter, 3-ft length, 8-lb weight, and 116 ft/sec velocity striking at all elevations. A wooden utility pole is considered as a missile in some analyses, but is not a required design basis missile since a study showed that wind speed of 132 mph lacks the aerodynamic lift needed to make the pole airborne. Further discussion is found in section 3.3.5.4.1 and References 9, 10, &11.

As a result of the analysis in response to SEP Topic III-4.A, RG&E as part of the Structural Upgrade Program discussed in Section 3.3, modified the facility to provide adequate tornado protection for those systems required to perform the safety functions discussed above. The specific modifications to provide protection from tornado missiles are discussed in Section 3.3.

3.5.2.2 Site Proximity Missiles

3.5.2.2.1 Design Criteria

The potential for site proximity missiles, including aircraft, was evaluated to verify that safety-related structures, systems, and components will not be jeopardized. The acceptability of the design of the facility for protection against site proximity missiles was based on meeting the requirements of General Design Criterion 4.

3.5.2.2.2 Nearby Hazardous Activities

The potential for hazardous activities in the vicinity of Ginna Station is addressed in Section 2.2. As indicated there, little industrial activity is situated near the plant. The distances to the nearest land transportation routes (about 1700 ft to the nearest highway, and 3.5 miles to the nearest railroad) are far enough to result in low risk from potential missiles caused by transportation accidents. Similarly, the nearest large gas pipelines (about 6 miles away) do not pose a missile threat to the plant. Major Lake Ontario shipping routes (about 23 miles from the plant) are not close enough to present a credible missile hazard from lake traffic. There are no military facilities or activities near enough to the plant to create a missile hazard.

3.5.2.2.3 Aircraft Hazards

The potential for aircraft becoming missile hazards has also been evaluated. Operation of the Williamson Flying Club airport and commercial air traffic in and out of Rochester, New York, via two federal airways, 2.5 and 10 miles from the plant site, were considered. Flight activity in an Air Force restricted area in the vicinity of the plant site was also evaluated.

The Williamson Flying Club airport is a small, privately owned general aviation facility located approximately 10 miles east southeast from the plant. The airport is used for general aviation activities such as business and pleasure flying and for agricultural spraying operations. As of 1981, there were 5,000 operations per year at the facility. The small number of operations is substantially less than the criteria in Section III.3 of Section 3.5.1.5 of the Standard Review Plan (SRP), and is sufficiently small that in the SER for SEP Topic II-1.C (*Reference 5*) the NRC staff determined that these operations are not a potential hazard.

Monroe County Airport in Rochester, New York, is located about 25 miles south-west of the plant and is the nearest airport with scheduled commercial air service. Low altitude federal airways V2 and V2N (the current FAA designation is airway V483, vice V2N) pass about 10 miles south and 2.5 miles southwest of the plant, respectively. The low altitude federal airways, V2 and V483, serve about 10 flights per day. Almost all flights use V2, with V483 being used only occasionally. The probabilities for an airline crash at Ginna from these airways are 5.1×10^{-8} for airway V2 and 1.4×10^{-8} for airway V483. Because both airway probabilities are less than the 1×10^{-7} acceptance criteria, the NRC concluded in the Safety Evaluation Report for SEP Topic II-1.C, dated September 29, 1981, that the probability of a commercial air traffic crash at Ginna Station is acceptably low.

Air Force Restricted Area R-5203 is located about 8 miles north of the plant site. Whenever flight activity is conducted by the Air Force within area R-5203, radar surveillance is maintained by the 174th Fighter Wing, the 108th Tactical Control Group, or possibly the Cleveland Air Route Traffic Control Center. Pilots rely upon on-board navigational equipment to maintain their presence within the specified limits of the restricted area. Pilots can also be advised if their aircraft stray beyond their limits by the radar surveillance unit covering the area at the time. The restricted area is used daily for military flight training which includes high-speed interceptor training maneuvers, operational flight checks, and air-to-air fueling. The altitude ranges in 1981 were from 2,000 to 50,000 ft above the surface. There is also an inactive slow-speed low altitude military training route (SR-826) which passes about 6 miles west of the plant. Route SR-826 is not currently a military controlled air space. Acceptance criterion II.2 of SRP 3.5.1.6 states that, for military air space, a minimum distance of 5 miles is adequate for low level training routes, except those associated with unusual activities, such as practice bombing. Air Force Restricted Area R-5203 is about 8 miles from the site at its closest boundary, and no unusual activities such as practice bombing take place. The inactive slow-speed low altitude military training route SR-826 is about 6 miles from the plant. Therefore, this criterion is met.

REFERENCES FOR SECTION 3.5

1. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Systematic Evaluation Program Topic III-4.C, Internally Generated Missiles, dated February 17, 1982.

2. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Turbine Disc Cracking (Safety Evaluation), dated August 28, 1981.

3. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Turbine Disc Cracking, dated September 16, 1981.

4. Letter from D. L. Ziemann, NRC, to L. D. White, Jr., RG&E, Subject: SEP Topic III-4.B, Turbine Missiles, dated April 18, 1979.

5. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic II-1.C, Potential Hazards Due to Nearby Transportation, Institutional, and Military Facilities - R. E. Ginna, dated September 29, 1981.

6. Westinghouse Electric Corporation, Probabilistic Evaluation of Reduction in Turbine Valve Test Frequency, WCAP-11525 (Proprietary) and WCAP-11529 (Non-Proprietary), dated June 1987.

7. 0236-0076-CALC-001, Rev. 0, MPR Calculation - Risk Assessment of Steam Turbine Valve Test Interval Extension, dated November 23, 2016.

8. Letter from D. M. Skay, NRC to M. Korsnick, R. E. Ginna Nuclear Power Plant, LLC, "Amendment Eliminating Requirements for Hydrogen Recombiners and Hydrogen Monitors Using Consolidated Line Item Improvement Process," dated May 5, 2005.

9. Letter from R. W. Kober, RG&E, to D. M. Crutchfield, NRC, Subject: Structural Upgrade Program, dated 7/13/84 (CMIS record ID "RG005699").

10. Appendix C to Structural Reanalysis Program "Utility Pole Tornado Missile Trajectory Analysis" (CMIS record ID "RG004925.00C") dated May 19, 1983.

11. Letter from A. Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Supplemental Safety Evaluation - Systematic Evaluation Program/Structural Upgrade Program at R. E. Ginna, dated November 15, 1989.

3.6 PROTECTION AGAINST THE DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

This section describes the design features of Ginna Station that protect essential equipment from the consequences of postulated piping failures both inside and outside containment. Analyses were conducted in accordance with guidance and criteria set forth in the December 18, 1972, AEC letter (*Reference 1*) concerning high-energy pipe breaks outside containment and the Systematic Evaluation Program Review for Topics III-5.A and III-5.B related to pipe breaks inside and outside containment, respectively.

The analyses showed that, with certain modifications proposed by Ginna Station, 10 CFR 50, Appendix A, General Design Criterion 4 was met, in that all structures, systems, and components are designed to accommodate the effects of and are compatible with the environmental conditions associated with MODES 1 and 2, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These structures, systems, and components are protected against dynamic effects (including the effects of missiles, pipe whipping, and dis-charging fluids) that may result in equipment failures and from events and conditions inside and outside the nuclear power unit.

Pipe ruptures were postulated at arbitrary intermediate locations in addition to terminal ends and high stress and high usage factor locations as required at the time by Branch Technical Position (BTP) MEB 3-1 of Standard Review Plan Section 3.6.2 in NUREG 0800. Pipe whip restraints and jet impingement shields were installed as necessary to mitigate the effects of these arbitrary intermediate pipe ruptures. Generic Letter 87-11 (*Reference 2*) dated June 19, 1987, revised BTP MEB 3-1 to Revision 2 to eliminate the requirement to postulate arbitrary intermediate pipe ruptures and permitted the elimination of pipe whip restraints and jet impingement shields installed to mitigate the effects of arbitrary intermediate pipe ruptures.

3.6.1 POSTULATED PIPING FAILURES IN FLUID SYSTEMS INSIDE CONTAINMENT

3.6.1.1 Evaluation Procedure

3.6.1.1.1 Pipe Selection

A list of piping lines inside containment which normally or occasionally experience high-energy[a] service conditions are presented in Tables 3.6-1 and 3.6-2. These lines were evaluated for the effects of potential pipe breaks (*Reference 3*). The tables exclude those lines which have been recognized not to present a significant safety hazard. These exclusions are as follows:

A. Lines which are of a 1-in. diameter or less according to Regulatory Guide 1.46 and guidance from *Reference 4*.

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a. High-energy piping is defined as piping with operating temperatures 200 °F and higher or operating pressures 275 psig and greater.

B. Lines which meet Branch Technical Position ASB 3-1, Standard Review Plan 3.6.1, for protection against postulated piping failures.

C. Lines which are at reduced pressure and temperature during MODES 1 and 2.

3.6.1.1.2 Effects-Oriented Evaluation

An effects-oriented approach was utilized for evaluating the consequences of most potential high-energy line breaks. This approach postulates a high-energy pipe break inside containment anywhere along the line and analyzes the capability of the remaining systems to safely shut down the reactor. The following assumptions were made:

A. Pipe whip can occur only in the section of pipe which is attached to a sustained high-energy source. Credit is taken for all closed or automatically closed valves in the piping section which could terminate flow to the break. For example, only the segment of safety injection piping between the reactor coolant system and the check valve closest to the reactor coolant system is analyzed for whip. Safety injection piping upstream of the check valve will not whip, even though pressurized, because of the lack of a sustained energy source.

B. It is acceptable for a break, which results in a loss of coolant from one of the loops, to damage mitigation equipment for the broken loop because the unbroken loop is available for Emergency Core Cooling System (ECCS) functions.

C. Pipe of a given section modulus will not cause a loss of function in pipe of equal or larger section modulus, as a result of pipe whip or jet impingement.

Acceptance criteria for the effects-oriented evaluations are as follows:

AA. The reactor can be shut down and cooled using equipment available following the pipe break.

BB. Analysis of the event, or a more limiting event, demonstrates that the effects of the break yield doses less than 10 CFR Part 100 values.

A spectrum of loss-of-coolant accidents and a main steam line break have been analyzed and shown to have acceptable consequences (Chapter 15). Those analyses remain valid following high-energy line breaks inside containment as long as the minimum equipment assumed in the analyses remains operable.

3.6.1.1.3 Mechanistic Evaluation

Other evaluation techniques were considered to evaluate breaks that could not be shown to have acceptable consequences using the effects-oriented approach alone. For example, a mechanistic approach based upon breaks at locations of highest stress in the piping segment may result in acceptable consequences because these breaks are remote from required equipment or because the broken pipes are contained. This approach also analyzed failure mechanisms to demonstrate that the consequences were acceptable. For example, a broken pipe assumed to whip in an effects-oriented analysis may be shown to have sufficient strength to resist whipping using mechanistic methods.

3.6.1.2 Required Equipment

Systems, components, and equipment required for safe shutdown and to mitigate the consequences of postulated piping failures were reviewed to determine their capability in performing these functions when exposed to the effects of postulated high-energy piping failures.
These systems are listed below.

High-energy line breaks inside containment result in, or have the same effect as, loss-of-coolant accidents or steam or feedwater line breaks of various sizes. The engineered safety features, including the safety injection system, are required to mitigate the effects of these events.

This equipment includes the following:

- High-pressure safety injection.
- Low-pressure safety injection.
- Containment spray.
- Containment fan coolers and service water.
- Essential instrumentation.
- Auxiliary feedwater.
- Containment sump recirculation.

Other items to note concerning mitigation equipment are as follows:

A. Some breaks in the accumulator piping produce neither loss-of-coolant accident nor steam or feedwater line break effects. These accumulator line breaks require only normal systems to maintain a stable plant safe shutdown condition.

B. The low-pressure safety injection system is the portion of the residual heat removal system used to pump water to the injection nozzles in the reactor vessel.

C. All of the pumps for required systems are located outside the containment. The entire auxiliary feedwater system, except for standby auxiliary feedwater (SBAFW) injection piping, is outside containment.

D. Most lines connected to the reactor coolant system have at least one normally closed or automatically closed valve inside a loop compartment or are routed so that the compartments prevent breaks in one loop from affecting the other loop. Mitigation equipment to the unbroken loop is, in most cases, unaffected.

3.6.1.3 Safety Analysis

<u>3.6.1.3.1 Single-Failure Considerations</u>

3.6.1.3.1.1 Introduction

The only active components in engineered safety feature systems inside containment which are required to operate or change position are the containment fan coolers and the motor-operated isolation valves in the low-pressure safety injection system. Thus, these are the

only components which must be considered as potentially being affected by high-energy pipe breaks and which must also meet the single-active-failure criterion.

Single active failures of engineered safety feature pumps, valves, or power supplies outside containment have been shown previously in Emergency Core Cooling System (ECCS) analyses (Section 6.3) to have acceptable results. Passive failures of engineered safety feature equipment, including the maximum pump seal leakage or failure of a check valve, will be less limiting than the complete loss of a pump or power supply. The systems have been designed to accommodate such passive failures.

3.6.1.3.1.2 Containment Fan Coolers

Two of the containment fan coolers are located remotely from all postulated high-energy pipe breaks and will not be damaged by a break. The other two fan coolers are near only the 2-in. steam generator blowdown lines but will not be damaged as explained in Section 3.6.1.3.2.

3.6.1.3.1.3 Low-Pressure Safety Injection Isolation Valves

The two low pressure safety injection isolation valves are located on opposite sides of the reactor cavity shield wall, outside the loop compartments, and could be damaged only by a limited number of other high-energy lines. The only sustained high-energy source lines near the low-pressure safety injection lines are the accumulator lines. For all breaks in lines other than the accumulators or in the low-pressure safety injection lines themselves, neither of the isolation valves will be affected and no single failure will reduce the functioning of required equipment to less than the required minimum. An accumulator line break outside either of the loops (postulated using an effects-oriented approach) which could rupture a low-pressure safety injection line, or a low-pressure safety injection line break as the initiating event, will effectively result in a 4-in. hot-leg loss-of-coolant accident. The accumulator line break will not be more severe because a check valve inside the loop compartment prevents reactor coolant system blowdown through the accumulator line. Analysis of a 4-in. loss-of-coolant accident shows that reactor coolant system pressure remains well above the shutoff head of the low-pressure safety injection pumps and thus the transient is terminated without the use of the low-pressure safety injection system. Failure of one low-pressure safety injection isolation valve following damage to the other will have an inconsequential effect since one high-pressure safety injection pump delivers sufficient flow to mitigate the event. Additionally, for long term post-LOCA cooling following a 4-inch hot leg break, the low-pressure safety injection system is not required to prevent boron precipitation.

<u>3.6.1.3.2 High-Energy Line Break Effects</u>

3.6.1.3.2.1 Introduction

The discussion of the effects of high-energy line breaks in this section is restricted to the dynamic effects on mechanical and electrical equipment. The environmental effects on electrical equipment is discussed in Section 3.11 concerning the environmental qualification of electrical equipment per 10 CFR 50.49. The analyses of the high-energy lines presented in Tables 3.6-1 and 3.6-2 are summarized below. The results have been reported in *References 3* and *5 through 8*.

3.6.1.3.2.2 Alternate Charging

The line segment of interest is approximately 2 ft of 2-in. pipe in the loop A compartment between the reactor coolant system cold leg and the check valve. Alternate charging (identified as auxiliary charging) is not normally used so isolation valves inside and outside containment are normally closed. In addition, all three charging pumps are positive displacement pumps. For this reason, and because the design flow of a charging pump is only 60 gpm, no sustained high-energy source exists upstream of the check valve and thus, the pipe upstream of the valve will not whip. A break between the reactor coolant system and the check valve will be confined to the loop A compartment and will result in a small loss-of-coolant accident.

Mitigation equipment inside containment that may be used to mitigate loop A loss-of-coolant accidents is safety injection to loop B, low-pressure safety injection to either vessel nozzle, containment fan coolers, and containment spray. All of this equipment is remote from the break and is isolated by compartment walls. No unacceptable consequences will result from the pipe break, assuming check valve operability. If it is assumed that the check valves inside loop A were inoperable, since Ginna Station does not conduct periodic testing of these valves, the cabling for one of the two low-pressure safety injection valves could be affected by pipe whip upstream of the check valve. A single active failure of the other low-pressure safety injection valve would result in a loss of low-pressure safety injection. However, high-pressure safety injection would still be available to mitigate the small break loss-of-coolant accident. The NRC, in the Safety Evaluation Report of June 28, 1983, found this issue to be acceptably resolved (*Reference 9*).

As part of the Ginna power uprate, operation of one low pressure safety injection isolation valve is required following a cold leg small break LOCA to prevent the possibility of boron precipitation during the long-term post LOCA recirculation phase. Consequently, pipe whip of the alternate charging piping causing damage to the low head safety injection (SI) isolation valve cabling is unacceptable. Based upon a review of the piping stresses in the alternate charging piping upstream of the reactor coolant system (RCS) check valve, it has been determined that none of the piping stresses exceed the criteria specified in BTP MEB 3-1 for identifying pipe locations where pipe breaks must be postulated. Therefore, based on the relaxed criteria presented in Generic Letter 87-11, no intermediate break locations in the vicinity of the low head SI isolation cabling need to be postulated. Additionally, there are no piping terminal ends in the vicinity of the routing of the low head SI isolation valve cabling. Therefore, there are no pipe break locations in the alternate charging line that would damage the low head SI isolation valve cabling due to pipe whip. Consequently, for any break location in the alternate charging piping that needs to be postulated per MEB 3-1, one train of low head SI would be available to support long term cooling of the RCS following the limiting single active failure.

3.6.1.3.2.3 Residual Heat Removal Pump Suction

Breaks in this line are considered only between the reactor coolant system and the loop A innermost isolation valve inside containment (MOV 700) in accordance with Standard Review Plan 3.6-1. This line segment is within the loop A compartment. Piping downstream of the isolation valve will not whip because this piping is isolated from the reactor coolant system and there is no sustained high-energy source connected to the piping during power

operation and most shutdown operations. Breaks in the piping upstream of the isolation valve would result in a loss-of-coolant accident with the potential for the piping that is attached to the reactor coolant system to whip.

Also, the effect on containment integrity had not been analyzed for a break in this line which could impact the component cooling water (CCW) system piping to the reactor support coolers. The CCW system is considered a closed loop inside the containment. Therefore, in *Reference 27*, Ginna Station submitted a leak-before-break analysis (*Reference 28*) for this line. The NRC in *Reference 29* concluded that while the results of the NRC differed from the results obtained by Ginna Station, the NRC agreed with Ginna Station's conclusion that leak-before-break had been demonstrated for the analyzed portions of the residual heat removal (RHR) system. The NRC's conclusion was predicated on the leakage detection system inside containment being able to reliably detect 0.25 gallons per minute of leakage within 1 hour. The NRC further stated that Ginna Station may remove consideration of dynamic effects associated with the postulated rupture of the analyzed portions of the residual heat removal (RHR) system piping from the licensing basis.

3.6.1.3.2.4 Reactor Coolant Pump Seal-Water to Seals

The seal-water inlet lines to both reactor coolant pumps are pressurized to nominal operating pressure from the containment wall to the reactor coolant pumps. Both lines are fed by positive displacement charging pumps and are throttled outside of containment to an 8-gpm flow. Check valves near the reactor coolant pumps inside the loop compartments prevent backflow in the seal-water inlet lines. Breaks in the lines between the containment wall and the check valves will not result in pipe whip because there is no sustained high-energy source from the positive displacement charging pump because of limited flow. Breaks in the lines between the reactor coolant pumps and the check valves will be contained within the loop compartment. Mitigation of the break effects may be accomplished by the adjustment of charging and letdown flow or Emergency Core Cooling System (ECCS) actuation. All of the required piping of the mitigation systems is at least as large as the seal-water inlet lines and, particularly in the absence of any pipe whip, will not be disabled by the break. No unacceptable consequences will result from the pipe break.

3.6.1.3.2.5 Letdown Line

Letdown from the reactor coolant system is from loop B through the regenerative heat exchanger and letdown orifices inside containment. The letdown is a high-energy line over its entire length inside containment although the temperature is reduced downstream of the regenerative heat exchanger and the pressure is reduced downstream of the orifices. A break in the 2-in. letdown line will result in a small loss-of-coolant accident from loop B. Mitigation equipment inside containment which may be used to mitigate loop B loss-of-coolant accidents is safety injection to loop A, low-pressure safety injection to either vessel nozzle, containment fan coolers, and containment spray. Low-pressure safety injection, safety injection, and containment spray lines are in the vicinity of the letdown lines. The low-pressure safety injection and containment spray lines each have a section modulus much greater than the letdown line and therefore will not be affected by a broken letdown line. Letdown piping between the reactor coolant system and the outermost isolation valves inside containment has a section modulus greater than that of loop B safety injection piping in the

vicinity and therefore could cause damage to loop B safety injection piping. This portion of the safety injection system is not required to mitigate the effects of the loop B loss-of-coolant accident, however. Letdown piping downstream of the orifices and outermost isolation valves inside containment is routed near safety injection piping to loop A. These lines have a greater section modulus then the letdown piping; thus, all the required mitigating equipment will remain effective following the break.

The letdown line is located in the basement of containment and is routed in the vicinity of safety-related cable trays and conduit. An evaluation of the possible effects of a postulated failure of the letdown line was performed and it was concluded that additional protection of certain instrumentation was required. In order to ensure that safety injection is initiated and reactor coolant system pressure can be monitored, certain instrumentation cables for pressurizer pressure, pressurizer level, and reactor coolant wide-range pressure were rerouted from the basement level to the intermediate floor elevation of containment. This modification was coordinated with the 10 CFR 50, Appendix R, fire protection review.

3.6.1.3.2.6 Charging Line

The 2-in. charging line is fed from positive displacement pumps and is a high-energy line over its entire length inside containment. The normal charging path is through the regenerative heat exchanger flow control valve and check valve near cold-leg B. An alternative path is through the regenerative heat exchanger, flow control valve, and check valve near hot-leg
B. Breaks in the lines between the check valves and the containment wall would produce no pipe whip or significant impingement because of the lack of a sustained high-energy source from either end of the rupture, assuming credit is taken for check valve operability. Loss of charging flow will result in a minor loss of reactor coolant system inventory through the reactor coolant pump seals. This loss can be compensated for by alternate charging or the consequences can be mitigated by the Emergency Core Cooling System (ECCS). The Emergency Core Cooling System (ECCS) equipment would not be affected because of the lack of a sustained high-energy source supplying the break to cause pipe whip or significant impingement. The alternate charging line is remote from the break. The effects of breaks between the reactor coolant system and the check valves will be a small loss-of-coolant accident with all whipping pipes confined to the loop B compartment. The mitigation equipment inside the containment which may be used to mitigate loop B loss-of-coolant accidents is high-pressure safety injection to loop A, low-pressure safety injection to either vessel nozzle, containment spray, and containment fan coolers. All of this equipment is remote from the break, outside the loop B compartment walls. No unacceptable consequences will result from the break.

In order to take credit for the operability of the charging line check valves, the NRC required that Ginna Station conduct a check valve operability testing program. In lieu of such a commitment, Ginna Station chose to provide sufficient analysis or compensating measures such that no credit for the check valves was necessary. As noted in the NRC Safety Evaluation Report for Integrated Plant Safety Assessment Report (IPSAR), Section 4.13 (*Reference 9*), the effects of a failure of the charging line check valves result in consequences identical to those of a letdown line break. Modifications (instrument rerouting) for the postulated letdown line break will thus also ameliorate the effects of the postulated charging line breaks with failure of the check valves to operate.

3.6.1.3.2.7 Steam Generator Blowdown Lines

The 2-in. steam generator blowdown lines exit from the steam generators at elevation 255 ft, above the lower support structure for the steam generators. The lines exit from the loop compartments and are routed above the intermediate floor to the containment penetrations. A break in either of the blowdown lines will result in a small feedwater line break accident. Auxiliary feedwater is entirely outside containment. Other engineered safety feature equipment which may be needed to mitigate the accident, except service water (SW) and two of the fan coolers, is on the basement elevation and is separated from the blowdown lines by large reactor coolant system component steel support structures and a concrete floor. The fan coolers are on the same elevation as the blowdown lines. In *Reference 26*, a piping stress analysis was performed, which verified that the highest pipe stresses in the blowdown lines are in locations away from the fan coolers and service water (SW) lines. Therefore, by using the relaxed criteria presented in Generic Letter 87-11 and its attached BTP MEB 3-1 (Revision 2), consideration of blowdown piping breaks in the vicinity of these components is not required.

The steam generator blowdown lines were also evaluated for effects on safety-related cable trays and conduit. The B blowdown line passes near this safety-related cable tray and conduit. Calculations were performed to evaluate the stresses in the B line as part of the piping Seismic Upgrade Program. The stresses in the line are lower than 0.8 ($1.2S_b + S_A$); thus, breaks need only be postulated at the terminal ends and the two intermediate highest stress locations. Neither breaks at the terminal ends nor at the intermediate high-stress locations, which are located inside the loop compartments, will damage required safety-related instrumentation. No unacceptable consequences will result from a steam generator blowdown line break.

In the early 1990s, the majority of the 2-inch steam generator (SG) blowdown piping inside containment for SG "A" was replaced with 3-inch piping. As part of the plant modification that replaced the original 2-inch piping with 3-inch piping, a stress analysis for the 3-inch piping was performed (*Reference 36*). This stress analysis included an evaluation of pipe stress per the requirements of BTP MEB 3-1 to determine the piping locations for postulating pipe ruptures. The results of the analysis determined that there were no intermediate pipe locations that had stress levels that exceeded the value specified in BTP MEB 3-1 for identifying required break locations. Therefore, based on the relaxed criteria presented in Generic Letter 87-11, no intermediate break locations were postulated in the SG "A" 3-inch blowdown piping. Only breaks at terminal ends were postulated and none of these break locations were in the vicinity of the fan coolers or SW piping inside containment.

3.6.1.3.2.8 Main Steam and Feedwater Lines

The main steam and feedwater lines are above the operating floor and separated by at least one concrete floor from all engineered safety feature equipment and piping inside containment, except the containment spray headers and spray rings. The containment spray headers, rising along the containment walls, are remote enough from the main steam and feedwater lines so as not to be struck by broken lines. The spray rings are attached to the containment dome and are high enough above the main steam and feedwater lines that they

will not be struck. The two steam generators are on opposite sides of the containment, far enough apart so that a broken pipe on one steam generator will not affect the other. The feedwater lines have a smaller section modulus than the main steam lines. A rupture of a feedwater line will not cause the main steam line to rupture.

The main steam and feedwater lines are generally separated vertically by 20 ft or more. Ruptures in a main steam line which could produce pipe whip will cause motion in the plane of the pipe that will not carry it into the feedwater line. Nevertheless, if a main steam line rupture is also postulated to cause a feedwater line break, it is not expected that the consequences would be unacceptable or even more severe than those resulting from just a main steam line rupture. The total mass and energy release will be smaller than for a main steam break alone because there will be no auxiliary feedwater flow to the broken loop and because the average enthalpy of the escaping fluid will be less, due to reduced heat transfer during the transient. Secondary fluid will escape both as steam and as liquid feedwater and will remove less heat than the main steam line break alone.

Because an effects-oriented evaluation of the main steam and feedwater lines could not rule out the potential for a ruptured line striking the containment wall, a mechanistic evaluation of the main steam line was performed. The analysis methods used made evaluation of the main steam line a conservative envelope for both main steam and feedwater line rupture effects upon the containment wall. The thrust force applied by the escaping fluid to the pipe was calculated by multiplying the initial pressure by the pipe cross-sectional area. The steam line force calculation thus enveloped the feedwater force calculation. The evaluation of pipe whip effect on containment wall integrity was performed for both main steam lines A and B. The piping stress analysis results from the Ginna Station seismic upgrade program were used in the evaluation. The piping break locations were postulated at the following locations:

a. Terminal ends of piping run.
b. Sections where $S_{01} + S_E J > 0.8\ (1.2S_h + S_A)$, where the occasional loads are due to normal and upset (operating-basis earthquake) conditions.
c. A minimum of two intermediate locations of maximum stress.

Since none of the combinations $S_{01} + S_E$ exceeded the stress limit, circumferential breaks were assumed at the two intermediate points of maximum stress.

The instantaneous thrust force generated by the flashing steam-water mixture was calculated according to the methods described in "Structural Analysis and Design of Nuclear Plant Facilities," J. D. Stevenson et al., ASCE, 1980 (*Reference 10*).

This thrust force results in piping moments that may exceed the ultimate plastic moment at a local cross section. A plastic hinge may be formed and the kinetic moment of the thrust force may accelerate the pipe toward the containment wall.

The dynamic characteristics of the pipe required to evaluate its penetration in the containment wall for those locations where the wall is struck are:

1. The striking velocity of the pipe v_o.

2. The effective pipe diameter

$$d = \sqrt{4A_C / \pi} \quad \text{(Equation 3.6-1)}$$

where: A_C is the contact area.

3. The pipe weight W.
4. The pipe shape factor N.

These variables were evaluated for the postulated break cases. In the evaluation, the effect of the existing pipe supports and the crane structure were neglected to maximize the impact upon the wall. This is conservative since these restraints tend to decelerate the pipe motion and, therefore, decrease the striking velocity v_o.

The analyses evaluated the structural integrity of the wall considering overall wall response and evaluated the total pipe penetration depth in the wall.

The containment liner plate was not considered in the evaluation of the containment shell integrity. Characteristics of the wall were based upon prestressed concrete detailed drawings for the R. E. Ginna Nuclear Power Plant. The modified National Defense Research Committee (NDRC) formula was used for penetration depth calculations. In addition, the evaluation considered the response of the reinforced-concrete wall system to resist penetration from a deformable missile. The characteristics of the missile were used to develop an applied force time-history and an analysis for the overall response to the force was carried out as for an impulsive load. The analytical methods used are outlined in *Reference 10*.

The analysis results for penetration depth (X in inches) using the NDRC formula were as follows:

a. For break location in main steam line A: X = 13.96 in.

b. For break location in main steam line B: X = 3.48 in.

The analysis for missile penetration into the wall considering overall wall response resulted in Xm/Xc = 1.352. This is considerably less than the allowable ductility ratio for impulse loads for flexure in structures. The rectangular impulse load considered

aa. Collapse load of slab = 29,649 K.

bb. Plastic hinge moment = 2360 in K/in.

cc. Duration of impulse load = 0.00098 sec.

The conclusion of the analysis was that, even neglecting the 3/8-in. steel liner plate, structural integrity of the containment shell is ensured.

Main steam or feedwater line ruptures could result in a pipe whip which strikes the containment crane support structure. The crane is supported by eight vertical columns and horizontal bracing. A complete loss of one support column will not cause the crane to fall.

As part of the mechanistic evaluation discussed above, it was determined that the two highest stress locations between the terminal ends of the B main steam line which were postulated to break are not located along the pipe where it passes between the crane supports. Breaks at the terminal ends also will not impact the crane supports; therefore, it was concluded that the dynamic effects of a main steam line break will not cause crane failure.

3.6.1.3.2.9 Residual Heat Removal Pump Discharge Line

Breaks in this line are considered only between the reactor coolant system and the loop B innermost isolation valve (MOV 721) inside containment in accordance with Standard Review Plan 3.6.1. This line segment is within the loop B compartment. Piping upstream of the isolation valve will not whip because no sustained high energy source is connected to the piping during power operation and most shutdown operations. Breaks in the piping downstream of the isolation valve would result in a loss-of-coolant accident with the potential for piping that is attached to the reactor coolant system to whip. Therefore, a leak-before-break analysis was submitted to the NRC for review and approval (*Reference 28*). This was approved by NRC SER dated February 25, 1999 (*Reference 29*), as noted in Section 3.6.1.3.2.3.

3.6.1.3.2.10 Standby Auxiliary Feedwater Lines

Breaks are considered in this line between the steam generators and the check valves inside containment. These 3-in. line segments are attached to the feedwater lines near the steam generators and are above the operating floor. A break in these lines will result in a small feedwater line break. Auxiliary feedwater flow to the unbroken steam generator feedwater line will not be affected. All of the engineered safety feature equipment is remote from these lines. No unacceptable consequences will result from a break in these lines.

3.6.1.3.2.11 Accumulator Lines and Branch Lines

The accumulator branch lines greater than 1-in. diameter are two 2-in. level instrument taps, one 2-in. line to the reactor coolant drain tank, and one 2-in. high-pressure safety injection discharge line connected to each accumulator line injecting to the reactor coolant system. During operation, when the accumulators are pressurized, the lines to the reactor coolant drain tank are isolated approximately 5 ft from the accumulator tanks. The instrument tap lines run vertically along the side of the accumulator tanks.

The safety injection lines discharge into the accumulator lines near the shield wall outside each compartment with a check valve in the line 10 ft or less from the point of intersection with the accumulator line. Breach of the accumulator line due to a safety injection line pipe break will not result in a loss-of-coolant accident because of the check valves inside the compartment. The safety injection lines, if broken, could impact or impinge upon the 4-in. low-pressure safety injection lines; however, the section modulus of the low-pressure safety injection lines, shown on Table 3.6-3, is larger than that of the safety injection lines. The low-pressure safety injection lines will not be damaged to the extent that loss of function occurs and check valves in the lines will maintain the reactor coolant system pressure boundary. Operation and shutdown of the plant can be accomplished using the normal charging and letdown paths, which are remote from these break locations, or by charging and letdown

through the reactor coolant pump seals. Seal injection to the reactor coolant pump A passes through the area containing the safety injection branch line to loop A; however, the seal injection line has a larger section modulus than the safety injection line and will not incur damage that will cause loss of function as a result of the break.

Breaks in the 10-in. accumulator lines inside the loop compartments between the reactor coolant system and the accumulator line check valves would result in a loss-of-coolant accident; the effects of pipe whip or impingement will be confined to a single loop compartment. All of the mitigation equipment, including safety injection to the unbroken loop, low-pressure safety injection to the vessel nozzles, containment fan coolers, and containment spray, is outside the compartments and remote from the breaks.

Breaks in the A accumulator line between the accumulator tank skirt and the loop compartment walls will not, by themselves, result in a loss of primary coolant. The check valve located inside the loop B compartment will prevent loss of primary coolant. Only accumulator fluid will be lost as a result of the break. Interaction with other equipment is acceptable, provided the interaction does not cause loss of primary inventory or interfere with maintaining the plant in a safe shutdown condition; therefore, equipment required only for mitigation of loss-of-coolant accidents or large secondary system breaks need not remain functional for the accumulator break. The following equipment was eliminated from consideration:

- Containment spray line.
- High-pressure safety injection line.
- Low-pressure safety injection valve control circuits.
- Fan coolers.

However, the following equipment required further evaluation and is discussed below.

- Low-pressure safety injection line.
- Residual heat removal outlet line.
- Instrumentation circuits.

The A accumulator line stresses have been determined in the Seismic Piping Upgrade program (Section 3.9.2.1.8). Stresses in the line are low and, generally, are only 10% to 25% of allowable. Thus, breaks need only be defined at the terminal ends and at the two highest stress intermediate locations. The two terminal break locations are at the reactor coolant loop and inside the accumulator skirt. As discussed earlier, breaks inside the loop compartment will not affect the required mitigation equipment. The terminal end break inside the accumulator skirt will not damage any equipment or circuits required for safe shutdown.

In order to comprehensively address potential dynamic effect concerns from the accumulators both lines "A" and "B," Ginna elected to perform a detailed fracture mechanics analysis for these lines and submit it to the NRC for review and approval (*Reference 31* and *32*).

The ASME Code Class 1 portion of the accumulator A piping extends from the RCS loop cold leg nozzle to check valve 867A and motor-operated valve 721 (nodes 856 through 960).

This portion of the accumulator A piping also serves as part of the RHR system. All the nodal locations on the affected portion, with the exception of Node 856, were considered in the LBB evaluation of the RHR system piping, which the staff approved in a safety evaluation dated February 25, 1999. Therefore, the scope of the LBB evaluation for the accumulator A line in the application involves a short pipe segment that includes only Node 856, which is located at one end of check valve 867A.

The scope of the LBB evaluation for the accumulator B line includes only the elbow between the cold leg nozzle of the RCS loop (node 60) to check valve 867B (node 80), excluding the valve itself. There is no flow in the accumulator lines during normal operation, including no possibility of cold in-leakage past the insolation valves, and complex system transients are not involved.

The accumulator piping for both A and B is constructed from ASME Code, Section II, material classification SA-376, Type 316, stainless steel. The welds are fabricated with stainless steel electrodes (ASME IX filler material E316) using the SMAW process. The welds in both accumulator lines do not contain Alloy 82/182 material, which is susceptible to stress-corrosion cracking. The piping is schedule 160 with a nominal diameter of 10 inches. The operating pressure is 2235 psig and the operating temperature is 550 °F.

Leak Before Break (LBB) Methodology

Draft SRP 3.6.3 and NUREG-1061, Volume 3, specify that the LBB approached should not be applied to high energy piping that has experienced stress-corrosion cracking, water-hammer, or, low-and high-cycle fatigue. Ginna established that no active degradation mechanisms (flow accelerated corrosion and stress-corrosion cracking) were expected in the accumulator piping segments. Ginna referenced an NRC report, NUREG-0927, in which it was stated that the probability of water hammer occurrence in the affected portions of the accumulator system is very low. In addition, Ginna has implemented Electric Power Research Institute (EPRI) guidelines TR-106438, "Water Hammer Handbook for Nuclear Plant Engineers and Operators"' May 1996, to prevent and mitigate water hammer events at Ginna. As for fatigue, Ginna demonstrated by fatigue crack growth analysis as discussed below that fatigue will not be a significant problem for the accumulator lines.

NUREG-1061, Volume 3, recommends that actual plant-specific material properties be used in the LBB evaluations. Ginna determined that actual archival materials for the accumulator piping is not available, therefore the least favorable material properties from the EPRI Ductile Fracture Handbook, 1989, was used as basis for flaw acceptance criteria in the ASME Code, Section XI.

The material properties of interest for fracture mechanics and leakage calculations are the modulus of elasticity, the yield stress, the ultimate stress, the Ramberg-Osgood parameters for describing the stress strain curve, the fracture toughness, and the power law coefficient for describing the material J-Resistance curve. In the analysis, the least favorable of the base metal and weld metal properties were used to obtain conservative results.

Considering the highest stress locations coincident with the worst material properties, Ginna identified the following limiting locations for the LBB analysis, for the accumulator A pipe,

the critical location is the weld between the pipe and check valve 867A (node 856). For the accumulator B pipe, the two critical locations are the weld joint between the accumulator B pipe and the cold leg nozzle (node 60), and the weld joint between the accumulator pipe elbow and check valve 867B (node 80). These nodes were considered for the analysis because they are located at welds in a tee and elbow and consequently reflect high stresses due to the stress intensification effects. In addition, the SMAW weld properties at these nodes will provide the most conservative critical flaw and leakage flaw sizes because of its low toughness and susceptibility to thermal aging.

At the three criteria locations, Ginna calculated the leakage flaw sizes using loading associated with normal operating conditions, including axial forces and moments due to pressure, dead weight, and thermal expansion. Ginna calculated the length of a through-wall circumferential flaw which would generate a leakage rate of 2.5 gpm, which is 10 times the leakage detection capability of 0.025 gpm at Ginna.

Ginna calculated the critical flaw sizes at the critical locations using elastic-plastic fracture mechanics with the J-integral method. The pipe loading applied to the cracks were axial forces and moments of normal operating and faulted conditions. Including safe shutdown earthquake and seismic anchor motion loads. Ginna used the "absolute sum" method to add the individual axial forces and moments into the combined axial forces and moments.

Ginna performed a fatigue crack growth analysis to determine the sensitivity of the accumulator lines to the postulated small cracks when subjected to the various transients. Cracks of various depth and aspect ratios were assumed. The fatigue crack growth analysis showed that in 60 years the fatigue crack growth is insignificant. The analysis also showed that the crack will grow though-wall before extending in length significantly. This indicates that leakage will occur before safety margins are exceeded.

The NRC staff confirmed that the proposed pipe segments in accumulator lines A and B can be shown to exhibit LBB behavior consistent with the guidance in Draft SRP 3.6.3 and NUREG-1061, Volume 3. The NRC noted that Ginna has shown that: (1) a margin of 10 exists between the calculated leak rate from the leakage flaw and the leak detection capability of 0.25 gpm; (2) a margin of 2 or more exists between the critical flaw and the flaw having a leak rate of 2.5 gpm; (3) fatigue crack growth in 60 years has been shown to be insignificant.; (4) loadings are applied to postulated cracks and pipes consistent with SRP 3.6.3; and (5) there are no active degradation mechanisms associated with accumulator lines A and B.

The effect of a break in the accumulator level measurement taps on nearby instrument circuits has been evaluated. It had been determined that the A accumulator level tap is in the vicinity of safety-related cable trays and conduit. A break in this 2-in. line was evaluated, and, as a result, safe shutdown instrumentation was rerouted away from the dynamic effects of the postulated break.

3.6.1.3.2.12 Auxiliary Spray Line

The 2-in. auxiliary spray line is not normally used for pressure control and the isolation valve is normally closed. There is a check valve inside the pressurizer compartment. Breaks in the line between the reactor coolant system and the check valve will result in a small loss-of-

coolant accident. The effects of the break will be limited to the pressurizer compartment. None of the engineered safety feature equipment, including safety injection to loop B, will be affected by the break.

Breaks in the line upstream of the check valve will have minimal effect. Reactor coolant system blowdown is prevented by the check valve and the positive displacement charging pumps will not provide a sustained high-energy source to cause pipe whip or significant impingement. The loss of charging flow and mitigation of the accident effects as a result of the break are discussed in the charging line item above. Further, a failure in the check valve to operate will have results which are bounded by the effects of the postulated letdown line break.

3.6.1.3.2.13 Reactor Coolant System

Asymmetric blowdown loads resulting from double-ended pipe breaks in the main coolant loop piping are not considered as a design basis for Ginna Station. *Reference 11* provided the NRC safety evaluation of information submitted by Westinghouse for a group of plants that included Ginna Station to resolve Unresolved Safety Issue A-2, asymmetric loss-of-coolant accident loads. The evaluation concluded that the asymmetric loss-of-coolant accident loads need not be considered as a design basis provided certain conditions were met. By *Reference 12* Ginna Station submitted information regarding the capability of installed leakage detection systems to detect a 1-gpm leak within 4 hours. By *Reference 13* the NRC concluded that the leakage detection systems at Ginna Station met the criteria specified in *Reference 11*. See Section 5.4.11.1.2.

3.6.1.3.2.14 Pressurizer Surge Line

A fracture mechanics analysis was performed of this line, so that the dynamic effects associated with pipe rupture would be outside the design basis of the plant. The submittals were provided in *References 31* and *32*, and approved in *Reference 33*.

The 10-inch pressurizer surge line connects reactor coolant system (RCS) hot-leg B to the bottom of the pressurizer. The line is run along the loop B compartment wall and an exterior vertical wall of the refueling canal before turning upward to connect to the bottom of the pressurizer. Rupture of the line may require operation of the nearby low-pressure safety injection, high-pressure safety injection, and containment spray to mitigate the loss-of-coolant accident. These lines, although nearby, are mostly routed on the underside of the refueling canal which is above the basement floor. The surge line and mitigating equipment pipes are on walls which are normal to each other at an exterior corner over most of the pipe run.

The scope of the LBB evaluation for the pressurizer surge line coves the entire line from the primary loop nozzle junction to the pressurizer shell nozzle. The surge line was fabricated from wrought austenitic stainless steel, American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code) material classification SA-376, Type 316, and does not include any cast materials.

The piping welds were fabricated from stainless steel using gas tungsten arc welding (GTAW) and/or shielded metal arc welding (SMAW). None of the welds contain Alloy 82/182 material; therefore, primary water stress-corrosion cracking (PWSCC) is not a concern for

the welds. There is a stainless steel safe-end piece between the nozzle and reducer at the pressurizer, therefore, primary water stress-corrosion cracking (PWSCC) is not a concern for the welds. There is a stainless steel safe-end piece between the nozzle and reducer at the pressurizer nozzle location. The outside diameter of the pipe is 10.75 inches and the minimum wall thickness is 0.896 inch.

Leak Before Break (LBB) Methodology

Draft SRP 3.6.3 and NUREG-1061, Volume 3. Specify that the LBB approach should not be applied to high energy piping that has experienced stress-corrosion crackling, water hammer or low-and-high-cycle fatigue. Ginna has established that no active degradation mechanisms (e.g., flow accelerated corrosion, stress-corrosion cracking, fatigue) were expected in the subject piping segments. Also, it has been established that no unanalyzable loading events (water hammer) would be expected to occur in the surge piping segments, and that there has been no service-induced crackling or wall thinning in the surge lines of Westinghouse PWRs.

As part of the Ginna inservice inspection program, welds in the pressurizer surge line are periodically inspected. During the 2003 refueling outage, based on the liquid penetrant test, there were indication on the outer diameter surface of the weld that connects the safe-end to the pressurizer surge nozzle. A boat sample was removed from a section containing a number of indications and examined to determine the root cause of the indications. The root cause was attributed to hot cracking, which developed during original construction. Similar indications were observed during the 2005 refueling outage inspection of the same weld. The indications were ground out until the linear indications disappeared. It appears that the indications were not service-induced and not caused by any active degradation mechanism.

The mechanical properties of the surge line at room temperature were obtained from the manufacturer's certified materials test reports. The minimum and average tensile properties were calculated by using the ratio of the ASME Code, Section II properties at various operating temperature. The representative minimum yield strength and minimum ultimate strength at operating temperature were used for the flaw stability evaluations and the representative average yield strength was used for the leak rate predictions.

Based on consideration of the highest stress locations coincident with the worst material properties, Ginna identified three bounding locations at nodes 1020, 1120, and 1280 for the LBB analysis. Node 1020 has the highest stress and is located at the weld joint between the surge line and the hot leg. Node 11220 has the second highest stress and is located at the weld at the end of the bend of the pipe. Node 1280 has the third highest stress and is located at the weld joint between the surge line and pressurizer nozzle.

At the three limiting locations, Ginna calculated leakage flaw sizes using loading associated with normal operating conditions, which included axial forces and moments due to pressure, dead weight, and thermal expansion. Ginna also calculated the length of a through-wall circumferential flaw at the three weld locations that would generate a leakage rate of 2.5 gpm. This evaluation was based on the crack morphology parameters (surface roughness and number of turns) associated with fatigue cracks. Comparing a leak rate of 2.5 gpm to a detection capability of 0.25 gpm within 1 hour, a margin of 10 was achieved, which satisfies the LBB criterion in Draft SRP 3.6.3.

In recent industry experience, improved fuel integrity and reduced RCS radioactivity levels have caused the gaseous channel of the containment atmosphere radiation monitor to become less effective for RCS leakage detection. The detection of RCS leakage could take longer than is required in the plant technical specifications. In light of the experience, the NRC staff asked Ginna (Reference 34) whether the current leakage detection capability of 0.25 gpm at Ginna can still be maintained and satisfy Regulatory Guide (RG) 1.45, "Reactor Coolant Pressure Boundary Leakage Detection Systems," May 1973. Ginna cited its previous LBB application submittal and associated staff review for the residual heat removal (RHR) system piping in 1998 (*References 28* and *29*). In a letter dated August 8, 1998 (*References 35*) Ginna stated that the particulate monitors were demonstrated to be capable of detecting very small leak rates, even with robust fuel. Ginna did not credit the gaseous monitors to meet the 0.25 gpm within 1-hour detection capability, although the monitors are a useful backup. The second credited leak detection system is inventory balance. In Reference 29 the NRC staff approved the LBB application for the RHR system piping. In the NRC staff's safety evaluation, the staff found that the 0.25 gpm detection capability is acceptable because Ginna has a relatively small containment volume, effective recirculation of air in the containment, and the second generation of R-11 detector. For the current LBB application on the surge line, the staff also found that the leakage detection capability of 0.25 gpm is acceptable based on the previous information submitted.

Ginna calculated the critical flaw sizes for the 3 bounding locations based on limit load analysis, which follows the net section collapse criterion in NUREG-1061, Volume 3. The loading from the faulted conditions, which include normal operation conditions in conjunction with safe shutdown earthquake and seismic anchor motion loads was used. The severe transients such as thermal stratification and forced cooldown (also known as pressurizer reflood) were included. In the critical flaw calculation, the "absolute sum" method was used to add the individual axial forces and moments into the combined axial forces and moments. When analyzing the stainless steel weld using a limit load approach, an additional factor (Z-factor) was incorporated to account for the generally lower toughness and low load carrying capacity of the SMAW welds. Ginna applied the Z-factor to increase the applied loads and thus reduce the critical flaw size, which would be conservative. The ratios between the critical flaw size and the leakage flaw size for the bounding locations maintained a factor of 2, which satisfied the guidance in Draft SRP 3.6.3.

Ginna also performed a fatigue crack growth analysis to determine the sensitivity of the surge line to the postulated small cracks when subjected to the various transients. Five cracks were assumed at the reducer between the surge line pipe and pressurizer nozzle. The initial flaws were assumed to be 10% of the wall thickness with an aspect ratio (crack length to depth) of 6 to 1. The result showed that the maximum final crack size after 60 years was insignificant. The flaw growth through the pipe wall is not expected to occur and it was concluded that fatigue crack growth is not a concern.

It has thus been confirmed that the surge line can be shown to exhibit LBB behavior consistent with the guidance in Draft SRP 3.6.3 and NUREG-1061, Volume 3. Ginna has shown that: (1) a margin of 10 exists between the calculated leak rate from the leakage flaw size and the detection capability of the leakage detection system; (2) a margin of 2 or more exists between the critical flaw size and the leakage flaw size having a leak rate of 2.5 gpm;

(3) loadings are applied to postulated cracks and pipes consistent with SRP 3.6.3; (4) fatigue crack growth in 60 years has been shown to be insignificant; and (5) there are no active degradation mechanisms associated with the surge line.

3.6.1.3.2.15 Pressurizer Spray Lines

The pressurizer spray nozzle is fed with 3-in. lines from each loop through isolation valves inside the pressurizer compartment. The line from loop B is routed entirely within the loop B compartment and the pressurizer compartment. The line from loop A passes outside the loop A compartment near the accumulator and high-pressure safety injection lines to loop A and near a low-pressure safety injection and containment spray line before entering the pressurizer compartment. A rupture in either of the pressurizer spray lines will result in a small loss-of-coolant accident. All of these lines, with the exception of the 2-in. portion of the high-pressure safety injection line, have a section modulus greater than that of the pressurizer spray line and will not incur damage that will cause loss of function. For a break in either of the spray lines, the mitigating equipment inside containment which may be required is safety injection to the unbroken loop, containment fan coolers, and containment spray.

Safety injection to the unbroken loop and the containment fan coolers are remote from all break locations in both loop A and loop B spray lines. The containment spray lines are remote from breaks in the loop B line but could be affected by the loop A line. The containment spray line has a larger section modulus than the pressurizer spray line and will not incur damage that will cause loss of function.

Reach rods for the containment sump valves to the low-pressure safety injection pump suction lines are also in the area which may be affected by the loop A spray line. These valves are open and are inactive in the accident sequence. If breaks occur where damage can be done to the sump valve reach rods, flow may be restricted. A mechanistic evaluation of the pressurizer spray line from loop A, which passes near the reach rods, was performed and showed that breaks need not be postulated near the reach rods.

3.6.1.3.2.16 Pressurizer Safety and Relief Lines

The high-energy portions of the pressurizer safety and relief piping are the lines from the top of the pressurizer to the safety and relief valves. These lines are all less than 10 ft in length and are contained entirely within the pressurizer compartment. Ruptures in any of the lines will result in a small hot-leg loss-of-coolant accident. All of the engineered safety feature equipment required to mitigate the effects of the break is outside the compartment and will not be affected. No unacceptable consequences will result from the pipe break.

3.6.2 POSTULATED PIPING FAILURES IN FLUID SYSTEMS OUTSIDE CONTAINMENT

3.6.2.1 Introduction and Summary

3.6.2.1.1 Initial Evaluation

In December 1972, the NRC staff sent letters to all power reactor licensees requesting an analysis of the effects of postulated failures of high-energy lines outside of containment (*Reference 1*).

In response to that letter, Ginna Station submitted an evaluation of the effects of postulated high-energy line breaks outside of containment on November 1, 1973 (*Reference 14*). As a result of that evaluation and subsequent follow-up evaluations, Ginna Station committed to perform station modifications and to implement an augmented inservice inspection program to mitigate the effects of postulated pipe breaks (*Reference 15*). The augmented inservice inspection program was approved by the NRC in Amendment 7 to the Ginna operating license (DPR-18) by letter dated May 14, 1975 (*Reference 16*). The station modifications were as follows:

1. An augmented inservice inspection program was initiated to further reduce the probability of a main feedwater or steam line rupture.
2. A standby auxiliary feedwater system was (SAFW) added to further improve steam generator feedwater reliability and specifically to substitute for the preferred auxiliary feedwater in the low probability that preferred auxiliary feedwater pumps are damaged due to nearby high-energy pipe breaks within the intermediate building.
3. Check valves were added to existing preferred auxiliary feedwater lines near the connections to the main feedwater lines to minimize the preferred auxiliary feedwater piping that is pressurized during MODES 1 and 2.
4. Two parallel remotely operated valves were added to a crossover line between the motor-driven pump discharges to provide additional auxiliary feedwater makeup capability.
5. A large metal plate jet shield was installed underneath the main steam header in the intermediate building to protect the service water (SW) piping from a postulated crack in the main steam line. Jet impingement shields were added to protect vital equipment including containment isolation valves, motor generators, transfer switches, cable trays, terminal boxes and wiring, pressure transmitters, and reactor trip breakers. Also, jet shields were added to protect main steam bypass valves and piping, and at other locations.
6. Instrument cabling was relocated to areas that will not be affected by postulated high-energy pipe breaks.
7. The heating and ventilation system was modified to withstand postulated high-energy pipe breaks without further endangering the capability to safely shut down the plant.
8. In the east end of the cable tunnel, the cable tray which connects the intermediate building and the control building air handling room was sealed with a barrier and fire resistant materials. The cable tray which connects the control building air handling room and the relay room was also sealed with fire resistant materials.

9. Openings around pipes and cable trays that pass through the areas required for safe shut-down of the plant were sealed to prevent steam leakage into these areas in the unlikely event of steam or feedwater line breaks in the turbine building.
10. Steam generator blowdown lines were rerouted through the subbasement to minimize the potentially detrimental effects of breaks in these lines within the intermediate building.
11. Sufficient floor grating was installed at manholes to guard against flooding of safety-related equipment in the intermediate building resulting from an assumed feedwater line break.
12. Steam line pressure and feedwater flow transmitters were relocated away from the locations that could be affected by postulated high-energy line breaks.
13. Pressure-shielding steel diaphragm walls were installed at the control building-turbine building wall and at the diesel building-turbine building wall to ensure continued operability of safety-related equipment following a postulated high-energy pipe break in the turbine building.

3.6.2.1.2 Systematic Evaluation Program Reevaluation

In addition, certain modifications were made as a result of the Systematic Evaluation Program reevaluation of the effects of pipe breaks outside containment. These are summarized below:

1. Hose connections from the fire water system have been installed to provide an alternate source of cooling water for the diesel generators that is independent of the service water (SW) system. This responded to the possible damage to the power supplies to all service water (SW) pumps from high-or moderate-energy line breaks in the screen house.
2. The doorway between the mechanical equipment room and the battery rooms was replaced with a watertight wall. A water relief valve was provided between the mechanical equipment room and the turbine building. The evaluation had shown that a moderate-energy line crack in the service water (SW) piping located in the mechanical equipment room could result in the flooding of both battery rooms.
3. Pipe whip and jet impingement protection is being provided for the 6-in. heating steam line riser located on the intermediate floor of the auxiliary building to protect safety-related electrical equipment in the vicinity of the riser.
4. Heating steam lines have been removed from the relay room and air handling room in order to maintain a mild environment for the purpose of environmental qualification of electrical equipment in the rooms.
5. A spare charging pump breaker and feeder breaker for bus 16, stored in an area not subject to a heating or process steam line break, and spare power cable which can be routed from bus 16 to the charging pump were provided in order to restore power to the charging pump in the event that either the breakers or power feeds fail as a result of a postulated break in the steam heating line in the auxiliary building. The later environmental qualification of the charging pump and power supply breakers at bus 16 eliminated the need for the spare breakers. The spare power cable is still required, and is stored in an area outside the auxiliary building. See Section 3.6.2.5.1.8 for details of these modifications.

3.6.2.2 Evaluation Procedure

3.6.2.2.1 Initial Evaluation

The initial evaluation in response to the NRC December 1972 letter (*Reference 1*) was accomplished as discussed in the following paragraphs.

Piping lines were divided into three categories: high energy, moderate energy, and low energy. High-energy lines were those that exceeded 200°F and 275 psig, moderate-energy lines were those that exceeded 200°F or 275 psig, and low-energy lines were those that did not exceed 200°F or 275 psig. Only those lines that were in the same building or in the proximity of safety-related equipment required for safe shutdown were considered in the review. The lines reviewed are listed as follows:

Lines Considered	Location	Energy Level
Main steam	Intermediate and turbine buildings	High
Feedwater	Intermediate and turbine buildings	High
Preferred auxiliary feedwater	Intermediate building	Low
Steam supply to preferred auxiliary feedwater pump turbine	Intermediate building	Low
Steam generator blowdown	Intermediate and turbine buildings	High
Charging line	Auxiliary building	Moderate
Plant steam	Auxiliary, intermediate, and turbine buildings	Moderate

The lines considered high energy were evaluated for the effects of longitudinal and circumferential (full-diameter) breaks. The lines considered moderate energy were evaluated for the effects of crack breaks. The lines considered low energy were not postulated to break. The effects of full-diameter breaks considered were whip, jet impingement, pressurization, environmental, and flooding. The effects of crack breaks considered were jet impingement and environmental.

Main steam and feedwater line postulated breaks were reviewed to determine the equipment required to bring the plant to a safe shutdown. Should a major main steam line break occur, reactor trip, preferred auxiliary feedwater system operation, and isolation of main steam and main feedwater would be initiated. Following a major feedwater line break, reactor trip and preferred auxiliary feedwater system operation would be initiated. For these postulated breaks, cooling would be accomplished by feedwater addition through the preferred auxiliary feedwater system. The equipment required to bring the plant to a safe shutdown following main steam or feedwater pipe ruptures was listed.

3.6.2.2.2 Systematic Evaluation Program Reevaluation

The reevaluations of the effects of pipe breaks outside containment, in response to SEP Topic III-5.B, involved the comparison of Ginna Station with the then current NRC criteria for pipe breaks outside containment as set forth in Standard Review Plans 3.6.1 and 3.6.2 and Branch Technical Positions ASB 3-1 and MEB 3-1. Current criteria define a high-energy fluid system as one where the maximum operating temperature is greater than or equal to 200°F or the maximum operating pressure is greater than or equal to 275 psig. In the initial (1972) review, a high-energy system was one in which both temperature and pressure exceed 200°F and 275 psig, respectively. All other piping is considered moderate-energy piping in accordance with current criteria. An effects-oriented approach to determine the acceptability of plant response to pipe breaks, i.e., each structure, system, component, and power supply which must function to mitigate the effects of the pipe break and to safely shut down the plant was examined to determine its susceptibility to the effects of the postulated break. Break effects considered were compartment pressurization, pipe whip, jet impingement, spray, flooding, and environmental conditions of temperature, pressure, and humidity.

The SEP reevaluation of pipe breaks outside containment considered the zones within the plant which contain systems required for safe shutdown and/or systems required to mitigate the effects of postulated pipe breaks. These zones were the screen house, diesel-generator rooms, intermediate building (elevation 293, 278, and 253 ft), turbine building (elevation 289, 271, and 253 ft), control room, relay room, battery rooms, mechanical equipment room, and auxiliary building (elevation 271, 253, and 235 ft).

The safe shutdown systems which were examined from the standpoint of protection from pipe break effects were identified in the NRC staff's SEP Safe Shutdown Review for Ginna. These systems included the following:

- Reactor Trip System (RTS).
- Auxiliary feedwater system.
- Main steam safety, isolation, and atmospheric dump valves.
- Service water (SW) system.
- Chemical and volume control system.
- Component cooling water (CCW) system.
- Residual heat removal system.
- Instrumentation for shutdown and cooldown.
- Emergency power (ac and dc) and control power for the above systems and components.

The evaluations were conducted as described in Sections 3.6.2.3 and 3.6.2.4 to determine the possible break locations and effects associated with the postulated failure of the piping.

3.6.2.3 Analysis Criteria

3.6.2.3.1 December 18, 1972, AEC Letter Evaluation Criteria

For those lines outside containment a mechanistic analysis to determine break locations was performed in response to the AEC letter of December 18, 1972 (*Reference 1*), requesting general information related to the consideration of the effects of piping system breaks outside containment. The criteria used in that evaluation is presented below.

The mechanistic evaluation was as follows. Design-basis breaks in straight or curved pipes of a 4-in. diameter or greater were assumed to be either longitudinal or circumferential, with the break area equal to the flow area of the pipe. Design-basis breaks at branch points were assumed to be circumferential in the branch and longitudinal in the run with the break area equal to the flow area of the branch. The criteria used to select design-basis break locations were as follows:

A. Postulated breaks at terminal points (anchored, rigid attachment to equipment, or anchor extensions).

B. Postulated breaks at branch points.

C. Postulated intermediate breaks between terminal points whenever the primary stress (pressure, weight, operating-basis earthquake) plus secondary stress (thermal) exceeds 80% of ($S_h + S_A$), or where secondary stress alone exceeds 80% of S_A.

D. As a minimum, two intermediate breaks between terminal points were selected at locations of highest stress.

Crack breaks were postulated at adverse locations in moderate and high-energy piping and were assumed to be one-half the pipe diameter in length and one-half the pipe wall thickness in width.

3.6.2.3.2 Systematic Evaluation Program Criteria

In response to the NRC SEP review, an effects-oriented approach was used to reevaluate the analyses and its conformance with current criteria. The criteria utilized in this approach was selected from that used in the NRC Standard Review Plans 3.6.1 and 3.6.2 and associated Branch Technical Positions ASB 3-1 and MEB 3-1 (Revision 1). Excerpts from that criteria are as follows:

3.6.2.3.2.1 High-Energy Fluid Systems Piping

1. Breaks and cracks need not be postulated in those portions of piping from containment wall to and including the inboard or outboard isolation valves provided they meet the requirements of the ASME Code, Section III, Subarticle NE-1120 and the additional design requirements specified in MEB 3-1.
2. Breaks in Class 1 piping (ASME Code, Section III) should be
   a. At terminal ends.

b. At intermediate locations where the maximum stress range as calculated by Equation 10 and either Equations 12 or 13 of paragraph NB-3653, ASME III, exceeds 2.4 S_m.

c. At intermediate locations where the cumulative usage factor exceeds 0.1.

d. If two intermediate locations cannot be determined by b. and c. above, two highest stress locations based on Equation 10 should be selected. If the piping run has only one change or no change of direction, only one intermediate location should be postulated. As a result of piping reanalysis, the highest stress locations may be shifted; however, the initially determined intermediate break locations need not be changed unless one of the following conditions exist. (Note: This requirement was changed by Generic Letter 87-11, which eliminated arbitrary pipe break locations.)

1. Maximum stress ranges or cumulative usage factors exceed the threshold levels in b. or c. above.
2. A change is required in pipe parameters such as major differences in pipe size, wall thickness, and routing.
3. Breaks at the new highest stress locations are significantly apart from the original locations and result in consequences to safety-related systems requiring additional safety protection.

In such conditions, the newly determined highest stress locations should be the intermediate break locations.

3. With the exceptions of those portions of piping identified in item 1 above, breaks in Class 2 and 3 piping (ASME Code, Section III) should be postulated at the following locations in those portions of each piping and branch run.

a. At terminal ends.

b. At intermediate locations selected by one of the following criteria:

1. At each pipe fitting (e.g., elbow, tee, cross, flange, and nonstandard fitting), welded attachment, and valve. Where the piping contains no fittings, welded attachments, or valves, at one location at each extreme of the piping run adjacent to the protective structure.
2. At each location where the stresses exceed 0.8 (1.2 S_h + S_A) but at not less than two separated locations chosen on the basis of highest stress. Where the piping consists of a straight run without fittings, welded attachments, or valves, and all stresses are below 0.8 (1.2 S_h + S_A), a minimum of one location chosen on the basis of highest stress. As a result of piping reanalysis, the highest stress locations may be shifted from original calculations. (Note: This requirement was changed by Generic Letter 87-11, which eliminated arbitrary pipe break locations.)

4. Breaks in non-nuclear class piping should be postulated at the following locations in each piping or branch run. (Note: This requirement was changed by Generic Letter 87-11, which eliminated arbitrary pipe break locations.)

a. At terminal ends of the run if located adjacent to the protective structure.

b. At each intermediate pipe fitting, welded attachment, and valve.

5. If a structure separates a high-energy line from an essential component, that separating structure should be designed to withstand the consequences of the pipe break in the high-energy line which produces the greatest effect at the structure irrespective of the fact that the above criteria might not require such a break location to be postulated.
6. Leakage cracks should be postulated in ASME Code, Section III, Class 1 piping where the stress range by Equation 10 of Paragraph NB-3653 exceeds 1.2 S_m, and in Class 2 and 3 or nonsafety class piping where the stress by the sum of Equations 9 and 10 of Paragraph NC/ND 3652 exceeds 0.4 (1.2 S_h + S_A). Non-safety class piping which has not been evaluated to obtain similar stress information shall have cracks postulated at locations that result in the most severe environmental consequence. (Note: This requirement was changed by Generic Letter 87-11, which eliminated arbitrary pipe break locations.)

3.6.2.3.2.2 Moderate-Energy Fluid System Piping

1. Fluid Systems Separated from Essential Systems and Components.

   A review of the piping layout and plant arrangement drawings should clearly show that the effects of through-wall leakage cracks at any location in piping designed to seismic and nonseismic standards are isolated or physically remote from essential systems and components.
2. Fluid System Piping in Containment Penetration Areas.

   Leakage cracks need not be postulated in those portions of piping from containment wall to and including the inboard or outboard isolation valves, provided they meet the requirements of the ASME Code, Section III, Subarticle NE-1120, and are designed such that the maximum stress range does not exceed 0.4 (1.2 S_h + S_A) for ASME Code, Section III, Class 2 piping.
3. Fluid Systems in Areas Other Than Containment Penetration.
   a. Through-wall leakage cracks should be postulated in fluid system piping located adjacent to structures, systems, or components important to safety, except where exempted by Section 3.6.2.3.2.1, item 1 above and item 4 below or where the maximum stress range in these portions of Class 1 piping (ASME Code, Section III) is less than 1.2 S_m, and Class 2 or 3 or nonsafety class piping is less than 0.4 (1.2 S_h + S_A). The cracks should be postulated to occur individually at locations that result in the maximum effects from fluid spraying and flooding, with the consequent hazards or environmental conditions developed.
   b. Through-wall leakage cracks should be postulated in fluid system piping designed to nonseismic standards as necessary to satisfy that the functional capability of essential systems and components will be maintained after the piping failure, assuming a concurrent single active failure.
4. Moderate-Energy Fluid Systems in Proximity to High-Energy Fluid Systems.

Cracks need not be postulated in moderate-energy fluid system piping located in an area in which a break in high-energy fluid system piping is postulated, provided such cracks would not result in more limiting environmental conditions than the high-energy piping break.

5. Fluid Systems Qualifying as High-Energy or Moderate-Energy Systems.

   Through-wall leakage cracks instead of breaks may be postulated in the piping of those fluid systems that qualify as high-energy fluid systems for only short operational periods but qualify as moderate-energy fluid systems for the major operational period.

3.6.2.3.2.3 Type of Breaks and Leakage Cracks in Fluid System Piping

1. Circumferential Pipe Breaks.

   The following circumferential breaks should be postulated individually in high-energy fluid system piping at the locations specified above.

   a. Circumferential breaks should be postulated in fluid system piping and branch runs exceeding a nominal pipe size of 1 in., except where the maximum stress range exceeds the limits specified in Section 3.6.2.3.2.1, items 2 and 3, but the circumferential stress range is at least 1.5 times the axial stress range. Instrument lines, 1-in. and less nominal pipe or tubing size should meet the provisions of Regulatory Guide 1.11.

   b. Where break locations are selected without the benefit of stress calculations, breaks should be postulated at the piping welds to each fitting, valve, or welded attachment. Alternatively, a single break location at the section of maximum stress range may be selected as determined by detailed stress analyses (e.g., finite element analyses) or tests on a pipe fitting.

   c. Circumferential breaks should be assumed to result in pipe severance and separation amounting to at least a one-diameter lateral displacement of the ruptured piping sections unless physically limited by piping restraints, structural members, or piping stiffness as may be demonstrated by inelastic limit analysis (e.g., a plastic hinge in the piping is not developed under loading).

   d. The dynamic force of the jet discharge at the break location should be based on the effective cross-sectional flow area of the pipe and on a calculated fluid pressure as modified by an analytically or experimentally determined thrust coefficient. Limited pipe displacement at the break location, line restrictions, flow limiters, positive pump-controlled flow, and the absence of energy reservoirs may be taken into account, as applicable, in the reduction of jet discharge.

   e. Pipe whipping should be assumed to occur in the plane defined by the piping geometry and configuration, and to initiate pipe movement in the direction of the jet reaction.

2. Longitudinal Pipe Breaks.

   The following longitudinal breaks should be postulated in high-energy fluid system piping at the locations of the circumferential breaks specified in item 1 above.

   a. Longitudinal breaks in fluid system piping and branch runs should be postulated in nominal pipe sizes 4-in. and larger, except where the maximum stress range exceeds

the limits specified in Section 3.6.2.3.2.1, items 1 and 2, but the axial stress range is at least 1.5 times the circumferential stress range.

b. Longitudinal breaks need not be postulated at

1. Terminal ends.
2. At intermediate locations where the criterion for a minimum number of break locations must be satisfied.
3. Longitudinal breaks should be assumed to result in an axial split without pipe severance. Splits should be oriented (but not concurrently) at two diametrically opposed points on the piping circumference such that the jet reactions cause out-of-plane blending of the piping configuration. Alternatively, a single split may be assumed at the section of highest tensile stress as determined by detailed stress analysis (e.g., finite element analysis).
4. The dynamic force of the fluid jet discharge should be based on a circular or elliptical (2 D x 1/2 D) break area equal to the effective cross-sectional flow area of the pipe at the break location and on a calculated fluid pressure modified by an analytically or experimentally determined thrust coefficient as determined for a circumferential break at the same location. Line restrictions, flow limiters, positive pump-controlled flow, and the absence of energy reservoirs may be taken into account, as applicable, in the reduction of jet discharge.
5. Piping movements should be assumed to occur in the direction of the jet reaction unless limited by structural members, piping restraints, or piping stiffness as demonstrated by inelastic limit analysis.

3. Through-Wall Leakage Cracks.

The following through-wall leakage cracks should be postulated in moderate-energy fluid system piping at the locations specified in Section 3.6.2.3.2.2 above. (Note: This requirement was changed by Generic Letter 87-11.)

a. Cracks should be postulated in moderate-energy fluid system piping and branch runs exceeding a nominal pipe size of 1 in. These cracks should be postulated individually at locations that result in the most severe environmental consequences.

b. Fluid flow from a crack should be based on a circular opening of area equal to that of a rectangle one-half pipe diameter in length and one-half pipe wall thickness in width.

c. The flow from the crack should be assumed to result in an environment that wets all unprotected components within the compartment, with consequent flooding in the compartment and communicating compartments. Flooding effects should be determined on the basis of a conservatively estimated time period required to effect corrective actions.

3.6.2.3.2.4 Assumptions

In analyzing the effects of postulated piping failures, the following assumptions should be made with regard to the operability of systems and components:

1. Offsite power should be assumed to be unavailable if a trip of the turbine-generator system or Reactor Trip System (RTS) is a direct consequence of a postulated piping failure.
2. A single active component failure should be assumed in systems used to mitigate consequences of the postulated piping failure and to shut down the reactor, except as noted in item 3 below. The single active component failure is assumed to occur in addition to the postulated piping failure and any direct consequences of the piping failure, such as unit trip and loss of offsite power.
3. Where the postulated piping failure is assumed to occur in one, two, or more redundant trains of a dual-purpose moderate-energy essential system (i.e., one required to operate during normal plant conditions as well as to shut down the reactor and mitigate the consequences of the piping failure), single failures of components in the other train or trains of that system need not be assumed, provided the following: The system is designed to Seismic Category I standards, is powered from both offsite and onsite sources, and is constructed, operated, and inspected to quality assurance, testing, and inservice inspection standards appropriate for nuclear safety systems. Examples of systems that may, in some plant designs, qualify as dual-purpose essential systems are service water (SW) systems, component cooling systems, and residual heat removal systems.
4. All available systems, including those actuated by operator actions, may be employed to mitigate the consequences of a postulated piping failure. In judging the availability of systems, account should be taken of the postulated failure and its direct consequences such as unit trip and loss of offsite power, and of the assumed single active component failure and its direct consequences. The feasibility of carrying out operator actions should be judged on the basis of ample time and adequate access to equipment being available for the proposed actions.

3.6.2.3.2.5 Effects of Piping Failure

1. The effects of a postulated piping failure, including environmental conditions resulting from the escape of contained fluids, should not preclude habitability of the control room or access to surrounding areas important to the safe control of reactor operations needed to cope with the consequences of the piping failure.
2. The functional capability of essential systems and components should be maintained after a failure of piping not designed to Seismic Category I standards, assuming a concurrent single active failure.

3.6.2.4 Analysis in Response to December 18, 1972, AEC Letter

3.6.2.4.1 Rupture Load Analysis

In response to the December 18, 1972, AEC letter, the rupture loads for the main steam and feedwater piping systems outside containment were generated for each postulated pipe break location considering circumferential and longitudinal pipe ruptures. Analytical considerations included the numerical solution of the continuity, momentum, and energy, and state equations for every volume associated with the break using the computer coded solution PRTHRUST. Furthermore, the effects of both wave and blowdown thrust components were considered and credit was taken for flow limiters, pipe friction, and restrictions in the line.

In the development of the thrust-time curves, it was also postulated that the break occurred instantaneously and that the fluid condition inside the pipe was taken as the maximum pressurization conditions. The rupture loads were calculated up to a maximum time of 0.5 sec after the rupture. Thrust forces on branch lines were assumed as 1.26 PA for steam and 2.0 PA for fluid where P is the initial pipe stagnation pressure and A is the pipe flow area when no blowdown calculation is used.

3.6.2.4.2 Main Steam System Load Analysis

The transients were developed for the thrust forces during the first 0.5 sec after the longitudinal and circumferential breaks in the main steam piping system. These results were generated assuming that the stop and Main Steam non-return check valve in the steam lines remain inactive during the first 0.5 sec. The results indicated that the maximum thrust force is developed within the first 0.0005 sec after the break. This peak is the result of rapid acceleration of the steam at the break location before the limiting condition imposed by hydrodynamic and thermodynamic aspects of the flow field are achieved. As the depressurization wave moves upstream the flow rate decreases and consequently the forces decrease rapidly reaching a state where it becomes relatively constant.

These data were used as forcing functions in investigating the dynamic response of the ruptured pipe.

3.6.2.4.3 Feedwater System Load Analysis

The thrust forces on the feedwater piping due to circumferential and longitudinal breaks were calculated for the first 0.5 sec after the pipe rupture. Certain breaks were evaluated based on two fluid conditions inside the feedwater piping, i.e., MODE 3 (Hot Shutdown) and full load, because of the uncertainty as to which condition represents the most severe case. The most severe loading profile was used in further developments of the investigation.

3.6.2.4.4 Jet Impingement Load Analysis

Circumferential and longitudinal breaks in the main steam and feedwater piping result in the formation of jets which might impinge on safety-related structures, systems, or components. The configuration of the jet arising from longitudinal and crack breaks is such that the jet axis is perpendicular to the axis of the pipe and the orientation is about any point along the circumference of the pipe. For jets generated by circumferential breaks, the jet axis is parallel to the pipe axis and the orientation is always in the direction of the axis of the ruptured pipe.

For jet impingement effects on known targets, the following factors were considered:

- Break type, geometry, and orientation of jet axis.
- Jet expansion of 25 degrees.
- Target geometry and distance from jet.
- Fluid conditions.

3.6.2.4.5 Pipe Whip Analysis for Main Steam and Feedwater Piping

3.6.2.4.5.1 Analytical Methods

The piping dynamic response analyses were performed using the PIPERUP computer program. This program performs nonlinear elastic-plastic pipe whip analyses of three-dimensional piping systems subjected to dynamic time-history for any functions. The piping is modeled as an assemblage of straight and curved-beam finite elements. The analysis is conducted by integrating the system equations of motion with time.

Each pipe element is initially represented in the program as a combination of three subelements, whose sum stiffness equals the elastic stiffness of the pipe. During the analysis, if computed loads at a point are detected to exceed the yield capacity of the pipe, one of the three subelements is hinged; thus, the stiffness of the remaining two subelements corresponds to the strain hardening modulus of the material. The analysis is then continued: if the computed loads are later detected to exceed the ultimate capacity of the pipe, the second subelement is hinged, leaving a single subelement with a very small stiffness. Prediction of a plastic collapse mechanism, or pipe whip, is based on detection of excessive deflections.

The material properties used in the analysis were taken from ASME Code, Section III, for the piping materials at operating temperatures. Lower bound material property values were used to predict piping response.

The PIPERUP program has the capability to represent a flexible support with an initial gap between the pipe and its support. This feature was used to model conditions of pipe impact on structural components such as walls, floors, pipe sleeves, and columns. Evaluation of structural failure of such components was based on reaction loads computed by the program. In cases where pipe whip would impact other safety-related equipment, such as cabling and instrumentation, failure of that equipment was automatically assumed to occur.

For circumferential (guillotine) breaks, response of piping on both sides of the break was considered. For longitudinal breaks, loading at any critical orientation about the circumference of the pipe was considered.

The thermal hydraulic blowdown thrust loads input to the piping dynamic response analyses were obtained using the PRTHRUST program, as described in Section 3.6.2.4.6.

3.6.2.4.5.2 Results of Analysis

The intermediate building structure was shown to be generally incapable of resisting the pipe whip effects of most postulated main steam and feedwater pipe breaks within the building. Also, analyses of the main steam and feedwater anchor assemblies showed that these elements would be overstressed due to the breaks, and reactions from the anchor loading were shown to be excessive for the basic structural steel framing of the intermediate building.

Although the control building is somewhat remote from high-energy piping, it was determined to be possibly damaged by a main steam line pipe whip because the facade columns would not be effective in restraining the pipe whip.

Because of these potential effects of the postulated main steam line and feedwater line breaks, an augmented inservice inspection program was proposed by Ginna Station and implemented to protect against potential damage. This program consisted of radiographic examination of all welds at the design-basis break locations in the main steam and feedwater lines and at other locations where a failure would result in unacceptable consequences. Presently, volumetric techniques are employed. The augmented inservice inspection program is described within the High Energy Program section of the Inservice Inspection (ISI) Program document. This High Energy Program is designed to preclude design bases or consequential main steam or feedwater pipe breaks.

Certain consequential main steam and feedwater line breaks in the turbine building were also calculated to possibly produce pipe whip damage to the intermediate building. Thus, these break locations were included in the augmented inservice inspection program. Modifications to systems, components, and structures to preclude damage to safety-related equipment required for safe shutdown are discussed in Section 3.6.2.1.

<u>3.6.2.4.6 Blowdown Analysis</u>

3.6.2.4.6.1 Main Steam Blowdown Analysis

The thermal-hydraulic analysis of the main steam blowdown was performed utilizing the PRTHRUST computer code. A model of the main steam analysis was constructed to represent the major pieces of equipment in the main steam system with their interconnecting piping and adjoining systems (condensate and feedwater). The model includes inventories of steam and water, piping flows, and heat sources applied on a control volume basis. Main steam line volumes were selected to account for segments between the elbows on either side of the postulated break point.

The main steam system blowdown analysis was conducted for both the short-term effects (i.e., pipe thrust) and the full duration transient (compartment differential pressures and building environment). The short-term transient blowdown (0.5 sec) is unaffected by the initiation of trip devices to mitigate the consequences of the accident due to their reaction time. The analysis was performed for break locations where double-ended (circumferential) guillotine ruptures were postulated, as well as for longitudinal breaks equal to the pipe cross-sectional flow area. In all cases, a break flow discharge coefficient of 1.0 was used for maximum blowdown flow rates.

The long-term blowdown is a continuation of the short-term analysis considering the effect of trip device activation. The long-term transient was carried out assuming the worst-case single active component failure. The results of this analysis were used to determine structural loadings.

3.6.2.4.6.2 Feedwater Blowdown Analysis

The PRTHRUST digital computer code was used in analyzing the feedwater blowdown transients. The system was represented by an assemblage of control volumes connected by flow paths or junctions. The effects of valves, pumps, heat exchangers, and check valves are included in the code. In addition, the program allows the operation of active devices to be

triggered by time or by physical signal such as pressure. The feedwater lines were divided so that volume size and junction location would provide optimum system representation for the particular case being analyzed.

It was assumed that for the duration of these analyses, the feedwater pumps would continue to operate and that flow would be a function of head (until automatic trips were initiated). It was further assumed that for the duration of these analyses, an unlimited supply of water at constant pressure was available at the feedwater pump suction. Both main feedwater pumps were combined and modeled as a single pump. The results of this analysis were used to determine structural loadings.

3.6.2.4.7 Compartment Pressurization Analysis

3.6.2.4.7.1 Main Steam Line Ruptures

The pressure-temperature transients resulting from a rupture of a main steam line in the intermediate and turbine buildings were investigated. These transients were calculated by using the main steam blowdown model to provide mass and energy flow into control volumes representing the intermediate building and turbine building with associated vent areas. The pressure transients were used in the structural evaluation described in Section 3.6.2.5.1.

3.6.2.4.7.2 Building Pressurization for a Branch Line Rupture

Small branch connections not included in the inservice inspection program had to be considered from a building pressurization standpoint. The worst-case branch rupture would be a 6-in. line (0.181 ft^2) leading from the main steam header. The steady-state steam flow that would issue from the postulated break (277 lbm/sec) is considered to transfer all of its latent heat of condensation to the surrounding air within the intermediate building. The resultant increase of air pressure would drive relief flow out of the building vent areas on an incremental steady-state basis. While this method of analysis is extremely conservative (because relative humidity is not taken into account), it provides an upper bound for intermediate building pressure. With an intermediate building vent area of 155 ft^2, this method of analysis gives
0.08 psi maximum intermediate building pressure, which is below the allowable limit. The plant uprate to 1775 MWt slightly increases the main steam system operating pressure at full power. This increase in pressure results in a slight increase in the steam blowdown rate and corresponding intermediate building pressurization. However, the increase in intermediate building pressure due to uprate is small and is still well below the allowable limits.

Building pressurization within the turbine building due to a branch line rupture is negligibly small; therefore, no damage is predicted for the adjacent control building or intermediate building from a branch rupture or crack break within the turbine building. See Section 3.6.2.5.1.4 for results of the structural analysis for pressurization of the turbine building.

3.6.2.4.8 Flooding Analysis

3.6.2.4.8.1 Intermediate Building Flooding

An intermediate building flooding analysis due to a postulated feed system rupture was performed. With slight modifications (new drainage provided) there is no danger of damage

to nuclear safety-related equipment due to flooding caused by a feedwater line rupture. The NRC was also concerned about possible flooding in the intermediate building subbasement due to a postulated high-or moderate-energy line failure. Ginna LLC considered this not to be of concern because present routine walk-through inspections of the intermediate building would detect a pipe leak long before there was any danger of flooding safety-related equipment. If the postulated leak occurred at a level above the subbasement, leakage into the subbasement via the floor drains would be obvious during the routine once-per-shift walk-throughs. Even a large secondary-side break would result in only a 2-ft depth of water in the subbasement. If the leak were in the service water (SW) piping located in the subbasement of the intermediate building, there would be a significant time interval between the initiation of the crack and the flooding of safety-related equipment. The intermediate building subbasement has a volume of approximately 50,000 ft^3. With a service water (SW) leak rate of about 640 gpm, it would take over 9.7 hours to begin flooding the basement level. It was considered that a sizable leak rate such as this would be detected visibly or audibly by personnel during the walkthroughs, or by personnel monitoring the control board (the 640-gpm leak would be a significant fraction (10%) of the service water (SW) pump flow).

There are two sump pumps in the subbasement. Sump high water level alarms sound in the water treatment room. Even if the basement elevation was flooded, safe shutdown would not be prevented. Based on this and the other information provided above, the NRC staff concluded that there are adequate means to warn of flooding conditions in the subbasement and, therefore, no modifications are required.

3.6.2.4.8.2 Screen House and Turbine Building Flooding

Protection is provided to protect safety-related equipment in the screen house and the turbine building from flooding because of leaks in the circulating water system. The protection consists of float switches in the circulating water pump pit in the screen house and in the condenser pit in the turbine hall with redundant two-out-of-three logic for tripping the circulating water pumps. Permanently installed, Seismic Category I dikes are in the screen house, and elevated doorways are between the turbine building and the control building to contain the water that may escape from the circulating water system. The design of these protective features is described in Section 10.6.2.9.

3.6.2.5 Systematic Evaluation Program Analysis

3.6.2.5.1 Zone Reevaluation Performed as Part of the Systematic Evaluation Program Review

The Systematic Evaluation Program included a review of the facility with respect to current Standard Review Plan criteria as well as a reevaluation of the original criteria and resolutions.

3.6.2.5.1.1 Screen House

Service water (SW) system or fire system moderate energy line cracks and heating steam line breaks could result in the loss of the service water (SW) system by damaging 480-V electrical buses 17 and 18 or their associated electrical motor control centers and cabling. Loss of the service water (SW) system would result in a plant trip because of the loss of several

components cooled by the service water (SW) system such as the reactor feed pump lube-oil systems, circulating water pumps, and the component cooling water (CCW) system. In accordance with current criteria, a pipe break that results in a reactor or turbine trip results, in turn, in a loss of offsite power. To supply ac power following a loss of offsite power, redundant emergency diesel generators are available; however, the diesel generators are supplied with cooling water by the service water (SW) system. Therefore, the postulated pipe break could cause the total loss of ac power at the plant, and reactor core decay heat removal would be dependent on the turbine-driven auxiliary feedwater pump.

To conduct a plant cooldown following a fire that causes a loss of the service water (SW) system with no offsite power available, Ginna Station has developed a procedure, which requires the installation of fire hoses from the yard hydrant system to provide the diesel generators with cooling water and to provide additional water to the preferred auxiliary and standby auxiliary feedwater pumps for steam generator makeup water. While the fire hoses are being installed, the turbine-driven auxiliary feedwater pump (TDAFW) can be used to add water from the condensate storage tank (CST) to the steam generators for decay heat removal.
After a diesel generator is operable, additional preferred auxiliary feedwater pumps and the reactor coolant system charging pumps can be operated as required.

The procedure can be used for the pipe break case even if the turbine-driven auxiliary feed pump is assumed to fail. Without feedwater addition, the steam generators can remove decay heat for approximately 35 minutes before they are boiled dry. This time could be used to make up the temporary diesel-generator cooling connections to start a diesel generator and a motor-driven auxiliary feed pump.

The NRC concluded that any further modification of the screen house to provide additional protection from pipe break effects for service water (SW) system components or for buses 17 and 18 is not required (*Reference 17*) (SEP Topic III-5.B).

3.6.2.5.1.2 Intermediate Building

Flooding from pipe breaks in the intermediate building would flow via open stairways and hatch gratings to the subbasement of the intermediate building. Sufficient drainage area is available so that no appreciable buildup of water would occur on any floor of the intermediate building except for the subbasement. No equipment necessary for safe shutdown or flood mitigation is located on this level, but if the flooding condition went unchecked, the intermediate building 253-ft elevation could be affected. Equipment on this elevation includes the preferred auxiliary feedwater pumps and the reactor trip breakers. If this equipment were flooded, a reactor trip would occur and the preferred auxiliary feedwater system would be inoperable. The standby auxiliary feedwater system, which is not located in the intermediate building, would still be operable even if a loss of offsite power occurred.

Postulated ruptures of the main steam or feedwater lines in the intermediate building would cause pressurization within the building. The intermediate building is a steel frame structure with walls constructed of concrete blocks and floor slabs of 5-in.-thick reinforced concrete. The peak pressure following the pipe break was determined by using the PRTHRUST computer program. Considering the existing vent area of approximately 140 ft^2, the pressure

inside the intermediate building reaches a maximum of 15.6 psig at 1.5 sec after the 36-in. main steam header breaks.

In analyzing the reinforced-concrete slabs subject to pressurization, yieldline theory was employed in calculating their maximum load carrying capacity. This theory takes into consideration the inelastic behavior of the reinforced-concrete elements.

The limit load capacities of the steel beams and girders were determined by plastic analysis.

The concrete block walls were analyzed as plates with an ultimate net compressive strength of masonry (f'm) of 528 psi, per ASTM C90 and a tensile strength of

$$(1.5 x 6 x \sqrt{f'm}) \qquad \text{(Equation 3.6-2)}$$

equal to 207 psi. The lateral uniform pressure required to fail the wall in bending with tension controlling was determined for the block walls. The critical shear was also checked.

The roof of the intermediate building is constructed of galvanized steel decking. Local buckling governs the pressure capacity of these panels.

All structural components in the intermediate building, with the following exceptions, are capable of withstanding the internal pressures in the building caused by the postulated breaks. The exceptions are the concrete block walls and the beams and decking of the high roof of the intermediate building. The pressure capacities of these components are 1.0 psi and 0.85 psi, respectively, as compared to the predicted pressure differentials of 15.6 psi and 15.47 psi.

Because of the severe consequences of postulated main steam and main feed line breaks in the intermediate building and because plant modifications to prevent these consequences were not practical, a two-part program to reduce the vulnerability of the plant to a high-energy line break in the intermediate building was undertaken. The first part of the program was the augmented radiographic inspection program to provide added assurance that postulated large main steam and main feedwater line breaks would not occur. The second part of the Ginna Station program was to move essential equipment from the intermediate building into locations unaffected by a high-energy line break in the intermediate building, shield equipment from the effects of the high-energy line breaks, or provide additional equipment. The intent of this program is to preclude the large (greater than the equivalent of 6-in. diameter) breaks and acceptably mitigate the small breaks. A summary of plant modifications installed and equipment relocated is provided in Section 3.6.2.1.

3.6.2.5.1.3 Turbine Building Main Steam and Main Feedwater Line Breaks

Postulated main steam and main feedwater system high-energy line breaks in the turbine building could result in the 24-in. main steam lines whipping into the intermediate building at an elevation which could result in damage to the B main steam line safety valves, the atmospheric dump valves, and the turbine-driven auxiliary feedwater pump (TDAFW) steam supply line. Also, breaks in the main steam line or main feedwater lines could result in pressurization of the turbine building itself. The pressurization of the turbine building could

adversely affect those areas adjacent to the turbine building in which safe shutdown or pipe break mitigating equipment is located.

In order to reduce the probability of postulated main steam and main feedwater line breaks in the turbine building a two-part program similar to that described for the intermediate building was undertaken.

The NRC-approved augmented inspection program was applied in the turbine building to main steam lines larger than a 12-in. diameter and several locations on the 20-in.-diameter main feedwater header. The inspection program limits the breaks which must be considered to be a 12-in. main steam or 20-in. main feedwater line break, which are the largest potential double-ended breaks in locations which are not inspected. Of these, the 20-in. main feedwater line is more limiting. To protect the areas adjacent to the turbine building from the effects of high-energy line breaks, pressure diaphragm walls between the turbine building and the control room, relay room, battery rooms, mechanical equipment room, and diesel-generator rooms were installed. The design differential pressure for these walls is 0.7 psi for the control room and 1.14 psi for the other spaces.

The pressure resulting from a 20-in. main feedwater or 12-in. main steam line break in the turbine building is sufficient to cause failure of the turbine building/intermediate building concrete block walls (design pressure 0.13 psid). If these walls failed, the following systems and components could be damaged by falling cinder blocks or adverse environmental conditions: one containment purge exhaust fan on the intermediate building 298-ft elevation, the preferred auxiliary feedwater system steam supply valves on the intermediate building 278-ft elevation, and the preferred auxiliary feedwater system turbine-driven pump, reactor trip breakers, and reactor rod control motor-generator sets on the intermediate building 253-ft elevation.

The purge exhaust fan is not required to function to mitigate a high-energy line break outside containment. The rod control motor generators and reactor trip breakers fail safe if damaged and would not prevent a reactor trip (core shutdown). The preferred auxiliary feedwater system function is required for a safe shutdown; however, the standby auxiliary feedwater system has been installed to accomplish this function if a high-energy line break disables the preferred auxiliary feedwater system. The turbine-driven auxiliary feedwater system pump is not specifically required to operate following a postulated high-energy line break since, even if offsite power were assumed to be lost, the redundant emergency diesel generators would be available to power the two standby auxiliary feedwater system pumps or the remaining two preferred auxiliary feedwater system pumps, all of which are driven by electric motors. Only one of these four motor-driven pumps is required for a plant shutdown and cooldown.

3.6.2.5.1.4 Structural Analysis of the Turbine Building for Pressurization

The turbine building is a steel frame structure with walls constructed of girts and galvanized sheet steel. Floor slabs are made of reinforced concrete.

The two largest high-energy lines within the turbine building are the 36-in. and the 24-in. main steam lines. However, these pipes are covered by the augmented inservice inspection

program which precludes all breaks except the crack break. This program reduces the maximum break area for these pipes to under 0.10 ft^2.

The two largest high-energy lines subject to a double-ended rupture are the 20-in. feedwater line and the 12-in. main steam line, both on the mezzanine level of the turbine building. These two lines were analyzed in detail to determine their mass and energy release following a postulated pipe rupture (*Reference 18*).

The worst-case break of the 20-in. feedwater line is a double-ended rupture (1.755 ft^2) while the plant is operating at full power conditions. To maximize mass and energy release, the break location chosen was in the 20-in. line just downstream of the No. 5 feedwater heaters. This location maximizes the available energy and inventory for the short-term release from the feedwater system. Determination of this mass and energy release was made using the FLASH (*References 19* and *20*) computer code series assuming a 1-msec break opening time and Moody flow with a 1.0 multiplier.

Since the turbine building pressurization is a short-term phenomenon (less than 1.0 sec), only short-term mass and energy release from the feedwater break is required. Therefore, no provision was made for feedwater pump trip or Main Feedwater Regulating Valve (MFRV) closure. To further ensure maximizing the mass and energy release, a single failure of one downstream check valve, 3992 or 3993, was assumed and no credit was taken for the flow limiter just upstream of these check valves.

A double-ended rupture (0.71 ft^2) of the 12-in. main steam dump to the condenser while the plant is in the MODE 3 (Hot Shutdown) condition was analyzed as the worst main steam line break. The break location chosen was just downstream of the 36-in. header. Mass and energy release from this break was also determined using the FLASH computer code series assuming a 1-msec break opening time and Moody flow with a 1.0 multiplier.

Since only short-term mass and energy release data is required, no provision for safety valve closures was made. To maximize the available inventory from the condenser side of the break, a steam dump isolation valve was assumed to fail.

The turbine building response to a pipe rupture within the building itself was analyzed using a three-node model and the COMPARE (*Reference 21*) computer code. Both the feedwater and the main steam breaks occur in the western half of the mezzanine level of the turbine building.

Node one of the model represented both the mezzanine and the basement levels of the turbine building, since flow area between the two levels is large. The operating level of the turbine building was represented by node two. The outside environment corresponded to node three.

Results of the analysis showed that the 20-in. feedwater breaks cause the most severe pressure transients within the turbine building. Calculated pressure differentials were determined to be 0.46 psid for the operating level and 0.85 psid for the mezzanine/basement level. The plant uprate to 1775 MWt increases the calculated turbine building peak differential pressures of the operating floor and the mezzanine/basement floors to 0.49psi and 0.91psi, respectively. Pressure differentials used for structural design of the turbine building steel diaphragm walls

are 0.70 psid for the operating level and 1.14 psid for the mezzanine/basement levels (see item 13 in Section 3.6.2.1). The steel diaphragm walls are at nearly opposite ends of the turbine building from the high-energy piping and therefore not subject to damage from pipe whip or jet impingement that could accompany the high-energy pipe break. The NRC concluded as a result of their safety evaluation of the structural adequacy of the turbine building steel diaphragm walls and of the results of the analysis reported in *Reference 16* that the structural criteria and design methods for the steel diaphragm walls are adequate to ensure safe shutdown of the reactor following a high-energy pipe break in the turbine building (*Reference 22*).

In addition to installation of the steel diaphragm walls at the control building-turbine building wall and the diesel generator building-turbine building wall, the turbine building structure was reinforced to withstand the pressurization resulting from the 20-in. feedwater and 12-in. main steam dump line breaks.

3.6.2.5.1.5 Battery Room/Mechanical Equipment Room Flooding

A service water (SW) system or fire main system postulated failure in the mechanical equipment room was considered capable of flooding both battery rooms and result in a loss of all emergency dc power. No sump level or flood alarms are installed in this space or in the battery rooms, which were originally connected to the mechanical equipment room via normally closed nonwatertight doors. The non-watertight door between the air handling room and the B battery room has been replaced by a wall to preclude flooding the battery rooms and a water relief valve has been installed between the mechanical equipment room and the turbine building.

3.6.2.5.1.6 Auxiliary Feedwater Line Breaks on the 253-Ft Elevation of the Intermediate Building

The preferred auxiliary feedwater system discharge lines from the pumps in the intermediate building (253-ft elevation) to the B main feedwater header run along the north wall of the intermediate building at approximately the 270-ft elevation. A break in this line, which is a high-energy line, could result in pipe whip or jet impingement on cable trays and containment electrical penetrations in that area. (The steam lines for the turbine-driven auxiliary feedwater system pump are also in this area but are not considered high-energy lines since they are not pressurized during normal plant conditions.) However, since the standby auxiliary feed-water system is routed completely separate from the preferred auxiliary feedwater system, safe shutdown could be accomplished following postulated auxiliary feedwater line breaks.

3.6.2.5.1.7 Relay Room and Air Handling Room

Crack breaks in the plant heating steam lines could cause high temperatures and high humidity in these rooms. The effects of these crack breaks were found to be acceptable because of the existence of temperature monitors for the detection of the failure. However, RG&E decided that it would be necessary to maintain the room as a mild environment for the purpose of the environmental qualification of electrical equipment as required by 10 CFR 50.49. Therefore, the heating steam lines were cut and capped or welded shut outside the

control building thus removing the source of high energy from the rooms. The steam heaters in the air handling room were replaced with electric resistance heaters.

3.6.2.5.1.8 Auxiliary Building

Postulated breaks in steam heating or process steam lines in the auxiliary building in the vicinity of safety-related equipment, such as an electrical bus, motor control center, or cable trays and conduit, could affect the operability of required safe shutdown equipment due to dynamic effects (jet impingement and pipe whip). Also, the general steam environment, although not expected to be severe throughout the entire auxiliary building, could possibly affect additional equipment required for safe plant shutdown.

In order to maintain a safe plant shutdown, the turbine-driven auxiliary feedwater system, which would not be affected by a high-energy line break in the auxiliary building, would be available to maintain preferred auxiliary feedwater flow to the steam generators, and thus maintain a safe shutdown condition. The condensate storage tanks (CST) have sufficient capacity to maintain auxiliary feedwater flow for at least 2 hours. The other sources of auxiliary feedwater described in Section 3.6.2.5.1.1 would also be available, since they are located away from the auxiliary building. Thus, auxiliary feedwater and cooling water would be available indefinitely.

In addition to auxiliary feedwater addition, a source of charging flow would be required to maintain inventory. For this purpose, the charging pumps would be used. The charging pumps, motors and “A” variable frequency drive (VFD) are located in the basement of the auxiliary building, in a separate concrete room, and thus are protected from the direct effects of a steam line break. Fire protection modifications for the charging room seal off major openings in the doors, windows, and ventilation penetrations. These barriers are designed for the postulated environmental effects of a steam line break (150°F, 0.1 psig). Thus, no significant steam environment would affect the charging pumps. Furthermore, the charging pump components required to withstand the adverse environment, per 10 CFR 50.49, have margin for operation even following exposure to these effects. The “B” and “C” charging pump motor VFDs are housed in a separate concrete room, adjacent to the charging pump room. The VFDs can be exposed to temperatures up to 158°F (Reference 37) and would be available for use following the isolation of the steam line break and restoration to ambient conditions. Any valves required to inject flow could be manipulated manually. The only equipment that might be affected by direct effects of the steam line breaks could be the charging pump and power supply breakers at bus 16 (intermediate floor of the auxiliary building), and power and control cabling in the basement of the auxiliary building. In order to resolve these issues, Ginna Station provided pipe whip and jet impingement protection for the steam line risers, located on the intermediate floor of the auxiliary building. The charging pump and power supply breakers at bus 16 were environmentally qualified for operation in the auxiliary building environment. The possibility of damage to the charging pump B power feed from bus 16 exists from direct impingement in the event of rupture of one of the 2-1/2-in. steam heating lines in the auxiliary building basement. Consequently, Ginna LLC maintains a spare cable which could be routed from bus 16 to the charging pump motor VFDs. The spare cable is stored in an area outside the auxiliary building. The control wiring and the DC power source for the breakers are not required because each breaker can be closed manually.

The breaker for charging pump A is located at bus 14 on the operating floor of the auxiliary building. Bus 14 is not in the vicinity of a process steam line and is unlikely to be affected by the dynamic effects of postulated steam pipe breaks. Furthermore, fire protection modifications rerouted the power cable and control wiring from bus 14 to the charging pump A in Hemyc wrapped conduit to provide a one hour rated fire barrier. This change also provides protection against the dynamic effects of postulated breaks in the steam heating or process steam lines in the auxiliary building.

It is estimated that the auxiliary building could be restored to ambient conditions and the spare power cable for the charging pump could be installed in less than 8 hours. While a source of charging is expected to be available (i.e., charging pump 1A-bus 14), the spare cable, routed from bus 16 to the charging pump motor VFDs, provides defense in depth. The NRC found the proposed method of achieving safe shutdown and the proposed actions to counter the effects of the postulated pipe breaks in the auxiliary building to be acceptable (*Reference 17*). The qualification of the charging pump and power supply breakers at bus 16 eliminated the need for the spare breakers referred to in *Reference 17*.

3.6.2.5.2 Main Steam Safety and Relief Valves

3.6.2.5.2.1 Pipe Failures in the Intermediate Building

Postulated main feedwater line breaks in the intermediate building could result in jet impingement on the main steam safety and relief valves. The jet from a crack in the B main feedwater line (upstream of the check valve) could impinge on the A main steam safety valves and atmospheric relief valves such that the valves inadvertently open. The opening of these valves would be roughly equivalent to a 1-ft^2 steam line break size, which is within the spectrum of steam line breaks analyzed in UFSAR Section 15.1.5. The A main feedwater line would be isolated to limit the blowdown and the standby auxiliary feedwater system would be actuated to provide feedwater to the B steam generator. The check valve would prevent the flow from being diverted out the cracked portion of the feedwater line. All necessary equipment to mitigate the event and reach safe shutdown is outside the intermediate building and thus would be unaffected by either the feedwater line failure or the steam blowdown. The cooldown could be controlled by operation of the steam safety or relief valves on the (unaffected) B main steam line.

Another consideration was a postulated crack in the A main feedwater line in the Intermediate Building. It is possible but not likely for the resulting jet to impinge on the safety valves and relief valves for both the A and B steam lines. The A steam line is closer to the A feedwater line than is the B steam line and thus could provide some shielding of the jet from the feedwater line crack. The nearest steam relief component associated with the B steam line is approximately 60 feet from the A feedwater line. At this distance the jet pressure from the feedwater line based on a 10-degree half angle of expansion for the jet is approximately 0.14 psi. This jet pressure is not sufficient to cause a break in the 6" inlet pipes to the B steam line safety and relief valves. Therefore, the potential damage to the main steam line from a crack in the A main feedwater line would be limited to the A steam line safety and relief valve components. If as a bounding assumption, all of the A steam line safety and relief valves are assumed to experience a complete severance of their 6" inlet pipes, the resulting main steam

header break area would be approximately 1 ft^2. Since this break area is within the spectrum of steam line breaks analyzed in the UFSAR Section 15.1.5, the consequences of this break are bounded by the results discussed in UFSAR Section 15.1.5. Since steam breaks in the B steam line header are not expected for this scenario, long term cooling of the RCS can be performed by using the B steam generator. As with a postulated crack in the B main feedwater line all the necessary equipment to mitigate the event and reach safe shutdown is located outside of the Intermediate Building and thus would be unaffected by either the feedwater line break or the steam blowdown.

3.6.2.5.2.2 Pipe Failures in the Turbine Building

Rupture of a main steam or main feedwater line in the turbine building could lead to building pressurization in excess of the capacity of the block wall between the turbine and intermediate buildings. Failure of the wall could result in blocks falling on nearby equipment and piping in the intermediate building. The blocks could potentially cause a loss of integrity of the main steam safety and relief valves. Damage to the main steam safety and relief valves would not prevent safe shutdown, as long as the main steam isolation valves remained operable, and auxiliary feedwater flow could be maintained to the steam generators. In such an event, the total break area would be approximately 2 ft^2, which is substantially smaller than the design-basis steam line break area of 4.37 ft^2. Thus, reactor coolant system pressure, temperature, and reactivity responses would be enveloped. Auxiliary feedwater would be provided by the standby auxiliary feedwater system (operator action time of 14.5 minutes is assumed). Other emergency functions, such as safety injection system actuation, would be unaffected by damage to the intermediate building. Auxiliary feedwater injection, with relief through the openings in the steam lines, would continue until the residual heat removal system could be placed into operation, at which time normal cooldown to MODE 5 (Cold Shutdown) could commence.

In order to ensure safe shutdown capability in the event of the block wall failure in the intermediate building, RG&E has performed the necessary analyses and modifications in conjunction with the Ginna Station Structural Upgrade Program to:

a. Ensure that the main steam lines and feedwater lines would not lose their structural integrity.

b. Protect the main steam isolation valves and accessories, as needed, to ensure operation.

c. Protect the normal motor-driven and turbine-driven auxiliary feedwater connections to the main feedwater lines, up to and including the check valves. This will ensure that standby auxiliary feedwater, which connects to the feedwater lines inside containment, would be routed to the steam generators.

As part of the Steam Generator Replacement Project, a flow venturi was installed in the main steam outlet nozzle for each steam generator. The flow venturis were sized with a flow area of 1.4 ft^2. This decreases the maximum credible design basis steam line break from 4.37 ft^2 assumed during SEP to 1.4 ft^2. This break size is smaller than the 2.0 ft^2 break conservatively assumed during the SEP review to assess the impact of the failure of the Intermediate Building masonry block walls following a steam line break in the Turbine Building.

Based on a review of the physical arrangement of equipment and piping in the Intermediate Building and an engineering assessment of the consequences of a block wall failure, the failure of the masonry block walls would not develop sufficient force to cause complete severance of the 6" schedule 80 carbon steel piping between the 30" main steam lines and the main steam safety valves and the atmospheric relief valves. Consequently, the 2 ft^2 steam line break scenario assumed during the SEP review is overly conservative. A less conservative but still bounding scenario of the consequences of the block wall failure is the complete severance of all main steam piping connections 3" and smaller in both 30" main steam headers. This scenario would result in a 12" Turbine Building steam line break (1.1ft^2) as the initiating event; and, a subsequent 0.2 ft^2 Intermediate Building steam line break in each of the 30" main steam headers. The total break area would be 1.1 ft^2. The consequences of this steam line break scenario have been analyzed consistent with the assumptions used for steam line breaks as discussed in UFSAR Section 15.1.5; and, the RCS response for this break scenario are bounded by the UFSAR Section 15.1.5 design basis steam line break.

3.6.2.5.2.3 Decay Heat Removal Following Blowdown from Both Steam Generators

As discussed above, postulated breaks in the turbine or intermediate buildings could, in the worst case, result in opening steam safety and relief valves on both main steam lines. The rate of emptying of the steam generator would depend on how many valves open, plant initial conditions, and availability of the preferred auxiliary feedwater system.

It is possible that the steam generators could be emptied in this event. In order to depressurize and cool the primary system sufficiently to permit operation of the residual heat removal system, decay heat removal through the steam generators must be reestablished.

The effect of adding auxiliary feedwater to a hot, dry steam generator has been considered. Rochester Gas and Electric presented results that showed that with 40 cycles of such feedwater addition, the usage factor on the tubes is still very low (*Reference 23*). This analysis provides assurance that the primary-secondary boundary will be maintained. The replacement steam generators (RSGs) were also evaluated for a limited number of cycles of cold main feedwater or auxiliary feedwater into a hot, dry steam generator (Reference 30). This evaluation demonstrated that the stresses in the vessel as a result of this transient remained lower than ASME Code Service Level D allowables. This evaluation did not address the tubing since the replacement steam generator (RSG) lattice grids preclude denting and subsequent locking at the supports and therefore do not impose a large axial restraint force on the tubes.

Should the preferred auxiliary feedwater system be unavailable due to the break effects (steam environment in the intermediate building), the standby auxiliary feedwater system would be manually actuated. Should the steam generator become ineffective as a heat sink, the capability exists to establish feed and bleed through the reactor coolant system for decay heat removal. The Westinghouse Owner’s Group Emergency Response Guidelines, approved by the NRC in *Reference 24*, provide for such a contingency. As part of the Three Mile Island Action Plan, NUREG 0737, Task I.C.1, the Ginna Station emergency procedures were modified in accordance with these guidelines.

3.6.2.5.2.4 Conclusions

The NRC has concluded that RG&E has demonstrated that given a postulated pipe failure in the intermediate or turbine building that damages main steam relief and/or safety valves, the consequences can be mitigated and a safe shutdown condition can be attained and that jet impingement shielding or protection from the effects of block wall failure for these components is not required (*Reference 25*) (SEP Topic III-5.B).

REFERENCES FOR SECTION 3.6

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Table 3.6-1
LINES PENETRATING CONTAINMENT WHICH NORMALLY OR OCCASIONALLY EXPERIENCE HIGH-ENERGY SERVICE CONDITIONS

			Normal Maximum Operating Conditions		
Penetration Number	**Line Size (in.)**	**Designation**	**Pressure (psi)**	**Temperature (° F)**	**Remarks**
120	1	Accumulator N_2	700	a	Vented during normal operation.
102	2	Charging (alternate)[b]	2250	a	No jet or whip upstream of check valve 383A; consider only line between reactor coolant pressure boundary and valve 383A.
140	10	Residual heat removal, out [b]	360	350	Consider only reactor coolant pressure boundary to valve 700; see SRP 3.6-1.
108	3	Reactor coolant pump seal water, out	< 100	200	Normally operated < 200° F, alarmed at 190° F.
106	2	Reactor coolant pump seal water, in [b]	2250	a	
110	2	Reactor coolant pump seal water, in [b]	2250	a	
110	3/4	Accumulator test	1500	a	Normally depressurized during test.
112	2	Letdown [b]	600	380	Higher pressure and temperature upstream of orifices and regenerative heat exchanger.
100	2	Charging [b]	2250	a	Higher temperature downstream of regenerative heat exchanger.
206	3/8	Sample, pressurizer liquid	2250	650	Eliminate because of size.
206	3/8	Sample, steam generator	1000	550	Eliminate because of size

			Normal Maximum Operating Conditions		
Penetration Number	Line Size (in.)	Designation	Pressure (psi)	Temperature (° F)	Remarks
205	3/8	Sample, reactor coolant hot leg system	2250	650	Eliminate because of size
207	3/8	Sample, pressurizer steam	2250	650	Eliminate because of size.
207	3/8	Sample, steam generator	1000	550	Eliminate because of size.
301	2	Unit heater steam	150	340	Decommissioned and welded shut in 1995.
303	1	Unit heater steam	150	340	Decommissioned and welded shut in 1995.
322	2	Steam generator blowdown [b]	1000	550	
321	2	Steam generator blowdown [b]	1000	550	
401	30	Main steam[b]	1000	550	
402	30	Main steam [b]	1000	550	
403	14	Feedwater [b]	1000	435	
404	14	Feedwater [b]	1000	435	
111	10	Residual heat removal, in [b]	360	350	Consider only reactor coolant pressure boundary to valve 721; see SRP 3.6-1.
119 and 120	3	Standby auxiliary feed [b]	1000	435	Consider only main feedwater line to check valves 9705A and B.

a. Indicates normal maximum temperature is less than 200° F.

b. Indicates those lines to be considered for potential high-energy line breaks.

Table 3.6-2
LINES INSIDE CONTAINMENT BUT NOT PENETRATING CONTAINMENT WHICH NORMALLY OR OCCASIONALLY EXPERIENCE HIGH-ENERGY SERVICE CONDITIONS

			Normal Maximum Conditions		
Line Designation	Size (in.)	System	Pressure (psi)	Temperature (°F)	Remarks
Primary system	---	Reactor coolant[a]	2250	600	Consider safety injection branch lines between reactor coolant pressure boundary and the first check valves.
Accumulator and branch lines[c]	---	Safety injection	700	b	Consider 2-in. branch lines to reactor coolant drain tank only up to valves 844A and B.
Auxiliary spray[c]	2	Chemical volume and control	2250	350	
Pressurizer surge[c]	10	Reactor coolant	2250	650	
Pressurizer spray	3	Reactor coolant	2250	650	
Pressurizer deadweight	1/8	Reactor coolant	2250	650	Eliminate because of size.
Tester tube flange leakoff	3/8-3/4	Reactor coolant	2250	600	Eliminate because of size.
Excess letdown	3/4	Reactor coolant	2250	600	Eliminate because of size.
Reactor overpressure protection N_2 lines	1	Reactor overpressurization	800	b	Eliminate because of size.
Pressurizer safety	4	Reactor coolant	2250	600	Consider only lines from pressurizer to valves 433 and 434.
Pressurizer relief	3	Reactor coolant	2250	600	Consider only lines from pressurizer to valves 430 and 431C.

a. Reactor coolant system piping breaks are being evaluated under NRC Task Action Plan A-2.

b. Indicates normal maximum temperature is less than 200 °F.

c. Indicates those lines to be considered for potential high-energy line breaks.

Table 3.6-3
CONTAINMENT PIPE DATA

Pipe Line	Size (in.)	Schedule	Section Modulus	Affected Portion of System
Safety injection	4	80	4.27	Penetration to T feeding hot and cold legs.
Safety injection	2	80	0.73	T to motor-operated valves 878A, B, C, and D.
Safety injection	2	160	0.98	Motor-operated valves 878A, B, C, and D to reactor coolant system or accumulator lines.
Low-pressure safety injection	10	40	29.90	Penetration to nozzle branch lines.
Low-pressure safety injection	6	40	8.50	Branch lines to motor-operated valves 852A and B.
Low-pressure safety injection	6	160	20.03	Motor-operated valves 852A and B to reducer.
Low-pressure safety injection	4	160	5.90	Reducer to nozzle.
Containment spray	6	40	8.50	All piping except spray rings.
Containment spray	4	40	3.21	Spray rings.
Containment spray	3	40	1.72	Spray rings.

Pipe Line	Size (in.)	Schedule	Section Modulus	Affected Portion of System
Seal water	8	40	16.81	All piping except connections to coolers.
Letdown	2	160	0.98	Reactor coolant system to valves 200A and B and 202.
Letdown	2	80	0.73	Downstream from valves 200A and B and 202.
Steam generator blowdown	2	80	0.73	All.
Main steam	30	80	>700	All.
Feedwater	14	100	118	All.
Reactor coolant	3	160	2.88	All.

3.7 SEISMIC DESIGN

3.7.1 SEISMIC INPUT

3.7.1.1 Introduction

3.7.1.1.1 Original Seismic Classification

Structures, systems, equipment, and components related to plant safety are required to withstand the design-basis earthquake. These structures, systems, and components are placed in the applicable seismic category depending on their function. The original classifications of all components, systems, and structures of Ginna Station for the purpose of seismic design were Class I, Class II, or Class III as recommended in:

1. TID 7024, Nuclear Reactors and Earthquakes, August 1963.
2. G. W. Housner, "Design of Nuclear Power Reactors Against Earthquakes," Proceedings of the Second World Conference on Earthquake Engineering, Volume I, Japan, 1960, pages 133, 134, and 137.

Class I

Those structures and components including instruments and controls whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity. Also, those structures and components vital to safe shutdown and isolation of the reactor.

Class II

Those structures and components which are important to reactor operation but not essential to safe shutdown and isolation of the reactor and whose failure could not result in the release of substantial amounts of radioactivity.

Class III

Those structures and components which are not related to reactor operation or containment.

All components, systems, and structures classified as Class I were designed in accordance with the following criteria:

A. Primary steady-state stresses, when combined with the seismic stress resulting from the response to a ground acceleration of 0.08g acting in the vertical and horizontal planes simultaneously, are maintained within the allowable working stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards, e.g., ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings.
B. Primary steady-state stresses when combined with the seismic stress resulting from the response to a ground acceleration of 0.20g acting in the vertical and horizontal planes

simultaneously, are limited so that the function of the component, system, or structure shall not be impaired as to prevent a safe and orderly shutdown of the plant.

All Class II components were designed on the basis of a static analysis for a ground acceleration of 0.08g acting in the vertical and horizontal directions simultaneously. For Ginna Station, there were no Class II structures.

The structural design of all Class III structures met the requirements of the applicable building code which was the State Building Construction Code of the State of New York, 1961. This code did not reference the Uniform Building Code.

3.7.1.1.2 Seismic Reevaluation

3.7.1.1.2.1 Scope of Reevaluation

The NRC conducted a seismic reevaluation of Ginna Station commencing in 1979 as part of the Systematic Evaluation Program (SEP). The reevaluation was conducted by the Lawrence Livermore National Laboratory for the NRC. The scope of the reevaluation was limited to identifying safety issues and to providing an integrated, balanced approach to backfit considerations in accordance with 10 CFR 50.109, which specifies that backfitting will be required only if substantial additional protection can be demonstrated for the public health and safety. The seismic reevaluation centered on the following:

a. An assessment of the integrity of the reactor coolant pressure boundary; i.e., major components that contain coolant for the core and piping or any component not isolable (usually by a double valve) from the core.
b. A general evaluation of the capability of essential structures, systems, and components to shut down the reactor safely and maintain it in a safe shutdown condition, including removal of residual heat, during and after a postulated safe shutdown earthquake. The assessment of this subgroup of equipment can be used to infer the capability of such other safety-related systems as the Emergency Core Cooling System (ECCS).

3.7.1.1.2.2 Reevaluation Criteria

Rochester Gas and Electric Corporation (RG&E) supplied a list of mechanical and electrical equipment necessary to ensure the integrity of the reactor coolant pressure boundary and to safely shut down the reactor and maintain it in a safe shutdown condition during and after a postulated seismic event. Rochester Gas and Electric Corporation also listed the criteria that it considered appropriate for evaluating the seismic classification of Ginna Station structures, systems, and components (*Reference 1*). The criteria reflected plant-specific requirements, not the more general light-water reactor standards currently in effect. They were as follows:

A. Seismic classification will be restricted to those structures, systems, and components required for safe shutdown, and to maintain reactor coolant pressure boundary integrity, and to prevent other design-basis accidents which could potentially result in offsite exposures comparable to the guideline exposures of 10 CFR 100. These latter systems and components include, for example, the steam, feed-water, and blowdown piping up to the first isolation valve, and the spent fuel pool (SFP), including fuel racks. Also included are

all structures, systems, and components not required to function, but whose failure could irreversibly prevent the functioning of required safe shutdown equipment or cause a design-basis accident. Seismic design of these items will ensure a very low probability of failure in the event of a safe shutdown earthquake.

System boundaries, for purposes of seismic reevaluation will be considered to terminate at the first normally closed, auto-close, or remote-manual valve in connected piping.

B. Safe shutdown is defined as the capability to control residual heat removal under all plant conditions resulting from a seismic event (with the consequential loss of function of non-seismic equipment) and a loss of offsite power. Safe shutdown may be the maintenance of an extended MODE 3 (Hot Shutdown) condition, or a gradual cooldown to MODE 5 (Cold Shutdown) conditions. For Ginna Station, safe shutdown assumes gradual cooldown and depressurization in the event of a safe shut-down earthquake.

The safe shutdown earthquake was the only earthquake level considered in the reevaluation because it represents the limiting seismic loading to which the plant must respond safely. Because a plant designed to shut down safely following a safe shutdown earthquake will be safe for a lesser earthquake, investigation of the effects of the operating-basis earthquake was deemed unnecessary.

In 1979, RG&E commenced a seismic piping upgrade program for Ginna Station to upgrade the seismic design of certain piping systems to current industry standards for Seismic Category I.

3.7.1.2 Design Response Spectra

The Ginna Station was originally designed for an operating-basis earthquake characterized by a peak horizontal ground acceleration of 0.08g and for a safe shutdown earthquake with a peak horizontal ground motion of 0.2g. Peak horizontal and vertical accelerations were assumed to be the same. The response spectra used were those developed by Housner (*Reference 2*) and are shown in Figures 3.7-1 and 3.7-2. The site seismology is described in Section 2.5.2.

For the SEP reevaluation a safe shutdown earthquake with a peak horizontal ground motion of 0.2g was used. Two-thirds of that value was used for the vertical component. The response spectra used was that given in Regulatory Guide 1.60. It is noted that the site-specific ground response spectra (Figure 3.7-3), recommended by the NRC (*Reference 3*) for SEP evaluation of the seismic design adequacy of Ginna Station, indicates a peak horizontal ground motion acceleration of 0.17g, less than the 0.2g value used.

3.7.1.3 Design Time-History

In the design of Ginna Station the seismic accelerations were computed as outlined in TID 7024 (*Reference 4*) and the Portland Cement publication (*Reference 5*). Response spectra developed by Housner (*Reference 2*) were used as described in Section 3.7.1.2.

During the SEP reevaluation, a time-history method was used to generate in-structure response spectra for the interior structures. Only horizontal excitations were included in the analysis. The input base excitation was a synthetic time-history acceleration record for which

the corresponding response spectra were compatible with the 0.2g Regulatory Guide 1.60 spectra. Response spectra associated with two orthogonal horizontal base excitations were generated independently at equipment locations and then combined by the square root of the sum of the squares method. Peaks of the spectra were broadened ±15% in accordance with current practice.

3.7.1.4 Critical Damping Values

Table 3.7-1 lists the damping values used for the original Ginna Station seismic design together with those from Regulatory Guide 1.61 for the safe shutdown earthquake and those values recommended in NUREG/CR-0098 (*Reference 6*) for structures at or below the yield point. The damping values used in the original design of Ginna Station are lower than current design levels. One reason is that the design damping values were used for the operating-basis earthquake, and the design loads were increased for the safe shutdown earthquake evaluation in direct proportion to the ratio of the two values of A_{max} (0.08g and 0.2g). Because higher response and, consequently, increased damping are expected for the safe shutdown earthquake, a significant degree of conservatism was typically introduced over current practice.

A comparison of the response spectrum developed by Housner for 2% damping with the 7% spectrum from Regulatory Guide 1.60 indicates the relative magnitudes of the response of bolted steel structures and equipment designed to Ginna versus current criteria. Similarly, the 0.5% spectrum for the original design and the 3% spectrum from Regulatory Guide 1.60 may be used to compare expected levels of response for base-level-mounted large piping for the two criteria. Figure 3.7-4 shows these comparisons. Similarly, expected levels of response for base-level-mounted large piping for the two criteria can be made by comparing the 0.5% Housner spectrum and the 3% Regulatory Guide 1.60 spectrum.

The NUREG/CR-0098 damping values are those recommended for the SEP reevaluation. The reason for permitting higher damping values is discussed in *Reference 6*. Although there are limited data on which to base damping values, it is known that the Regulatory Guide 1.61 values are conservative to ensure that adequate dynamic response values are obtained for design purposes. The lower values in the NUREG/CR-0098 column of values in Table 3.7-1 in most cases are close to the Regulatory Guide 1.61 values. The upper values in the NUREG/CR-0098 column are best-estimate values believed to be average or slightly above average values; these values are recommended for use in design or evaluation for stresses at or near yield, and when moderately conservative estimates are made of the other parameters entering into the design or evaluation.

3.7.1.5 Supporting Media for Seismic Category I Structures

All Ginna Station Seismic Category I buildings except the control building and diesel generator building are founded on solid bedrock. The foundations of the control and diesel generator buildings were excavated to the surface of bedrock. Lean concrete or compacted backfill was placed on the rock surface to a depth whereby the elevation of the top of the fill material was coincident with the elevation of the bottom of the concrete foundation of that particular building. Thus, all Seismic Category I buildings have rigid foundations. The turbine building foundation is a concrete mat supported by compacted fill material. See Section 3.8.5.

3.7.2 SEISMIC SYSTEM ANALYSIS

3.7.2.1 Seismic Analysis Methods

3.7.2.1.1 Original Seismic Analysis

The following method of analysis was applied to the original seismic Class I structures and components, including instrumentation in the original Ginna Station design:

A. The natural periods of vibration of the structure or component were determined.

B. The response acceleration of the component to the seismic motion was taken from the response spectrum curve at the appropriate period.

C. Stresses and deflections resulting from the combined influence of normal loads and the seismic load due to the 0.08g earthquake were calculated and checked against the limits imposed by the design standard.

D. Stresses and deflections resulting from the combined influence of normal loads and the seismic loads due to the 0.2g earthquake were calculated and checked to verify that deflections did not cause loss of function and that stresses did not produce rupture.

The maximum response acceleration of a structure or equipment item was read from the response spectrum for selected values of damping and a fundamental natural frequency. The frequency was either

- Calculated from a mathematical model,
- Measured from a plastic model (the case of the reactor coolant system),
- Estimated by experience, or
- Selected to be conservative (the peak of the spectrum was used).

From the mass of the structure or equipment and the maximum response acceleration, the equivalent static force was obtained. The equivalent static force, which represents the total dynamic effect, was then distributed along the system according to a selected shape (an inverted triangle for the containment) or according to the mass distribution. The static response to this equivalent static force was taken to be the seismic response of the system. Responses to horizontal and vertical ground accelerations were calculated separately, then combined by direct addition in most cases.

The containment and the residual heat removal system pipe line from the reactor coolant system loop to containment were analyzed by both the equivalent static and the response spectrum methods.

The seismic Class I piping systems were analyzed by a lumped mass approach. The number of masses lumped between any two supports was based upon the spacing interval and increases with the length of the spacing interval. Every mass was given an acceleration equal to the maximum response from the response curve with 0.5% of critical damping, i.e., 0.8g for 0.2g ground acceleration. Each piping system with its supports was modeled as a three-dimensional frame and the loads given by the mass times the acceleration were applied at each lumped mass along three directions, two horizontal and one vertical, separately. The

moments and torque for each of the three loading directions were then obtained by stiffness analysis. The stresses were calculated at critical points in the piping and its supports for each loading direction. The stresses in the piping were found by using the USAS B31.1 formula

$$S = \frac{\left(M_X^2 + M_Y^2 + M_Z^2\right)^{1/2}}{Z}$$

(Equation 3.7-1)

where: S = stress

M_X, M_Y, M_Z = moments about the two horizontal directions and the vertical direction

Z = section modulus

At each point the stresses obtained for the two horizontal loadings were conservatively combined by the square root of the sum of the squares. This value was then conservatively combined with the stress obtained for the vertical loading by direct addition.

3.7.2.1.2 Seismic Reevaluation

The seismic analysis methods changed greatly from the time of the design of Ginna Station to the SEP reevaluation. The original seismic analysis was primarily by the equivalent static method based on an estimated fundamental frequency of the structure. Response spectra were used primarily to predict the peak acceleration of the fundamental mode. The check of the static design analysis of the containment building was the only analysis that involved a multi-mode system.

Current analytical techniques and computer models at the time of the SEP reevaluation had increased considerably the sophistication and level of detail that could be treated. A complete dynamic analysis of complicated structural systems such as the interconnected building complex could be done conveniently and inexpensively.

For the SEP reevaluation, seismic analysis of the building complex was performed by the finite element method using the computer program SAP4.7 A three-dimensional mathematical model of the building complex was developed. The frequencies and mode shapes of the structural system were obtained from the computer analysis. After the frequencies and mode shapes were obtained, the structural responses were computed by the response spectrum method. The seismic input was defined by the horizontal spectral curve of the safe shutdown earthquake specified in Regulatory Guide 1.60 for 10% structural damping and 0.2g peak ground acceleration.

Two structural models were analyzed, one with half the bracing area (half-area model), one with the full bracing area (full-area model). For each model, two analyses were performed, one with the input excitation in the north-south direction, the other in the east-west direction.

The current licensing requirements would typically require load combinations different from those considered when Ginna Station was designed. The seismic reevaluation concentrated

on the original design combinations with primary attention devoted to the seismic margins. Other current assumptions and criteria are discussed in the following sections in comparison with those used in the design and analysis of Ginna Station.

3.7.2.2 Natural Frequencies and Response Loads

The frequencies and the ten largest modal participation factors of the full-area and half-area models are listed in Table 3.7-2. The modes with low frequencies were those dominated by steel parts of the structural system (i.e., the framing system) and the high-frequency modes were dominated by the concrete structures (i.e., the control building and the basement structures of the auxiliary building). Since several high frequency modes had significant modal participation factors, they were included in the dynamic analysis especially in computing the in-structure response spectra.

3.7.2.3 Procedure Used for Mathematical Modeling

A three-dimensional mathematical model for the building complex was prepared for the computer program SAP4 (*Reference 7*). All steel frames were modeled by beam elements. The model's rigid diaphragms for all roofs and floors were represented by the rigid restraint. The two-story concrete substructure of the auxiliary building and the control building were modeled by equivalent beams. The four shear walls of the diesel-generator building were represented by four elastic springs attached to the north frame of the turbine building at the diesel-generator building roof. The masses of the service building roof were lumped to the turbine and intermediate buildings. All other masses were lumped to the centers of gravity of floors or roofs.

3.7.2.4 Soil-Structure Interaction

Soil-structure interaction was not considered in the design of Ginna Station. Sophisticated methods of treating soil-structure interaction exist; however, for structures that are founded on competent rock, as is Ginna Station, the effects of soil-structure interaction are considered relatively small. There is little radiation damping, and consideration of rock foundation compliance results in only slight increases in the periods of response of a structure when compared with the fixed-base case. It was expected that any variation in load that results from neglecting soil-structure interaction would be well within the accuracy of the calculations.

This would be especially true for the containment structure, in which the walls are attached to the foundation rock by rock anchors. Therefore, soil-structure interaction was not taken into account in the seismic reevaluation.

3.7.2.5 Development of Floor Response Spectra

A direct method was applied to generate seismic input spectra for equipment at various locations in the structure (*References 8* and *9*). The method treated the earthquake input motions and the response motions as random processes. The response spectrum at any location in the structure was derived from the frequency response function of an oscillator, the frequency response function of the structure at that location, and the input ground response spectrum. This method avoided the troublesome task in the time-history approach of selecting the proper corresponding time-history input for the specified spectrum. The in-structure spectra generated from the half-area and full-area models were enveloped to give

the final spectra (Figure 3.7-5). If peaks were still obvious at structural frequencies, spectrum-widening techniques in accordance with current practice were then applied to ensure ±15% broadening to account for modeling and material uncertainties.

3.7.2.6 Combination of Earthquake Directional Components

The original design of Ginna Station structures involved the combination of a vertical and horizontal load, usually on an absolute basis. Current recommended practice is to combine the responses for the three principal simultaneous earthquake directions by the square root of the sum of the squares as described in Regulatory Guide 1.92. There is only a small difference between the two combination methods for circular plant structures like the containment building, which is the only structure for which a dynamic analysis was originally performed.

3.7.2.7 Combination of Modal Responses

For the SEP evaluation a detailed dynamic analysis using the response spectrum method was performed. In each analysis (east-west and north-south direction), 44 response modes were used and the individual modal responses were combined by the square root of the sum of the squares method.

3.7.2.8 Interaction of Nonseismic Structures with Seismic Category I Structures

A complex of interconnected buildings surrounds the containment building. Though contiguous, these buildings are independent of the containment building. The auxiliary, intermediate, control, and diesel-generator buildings are Seismic Category I structures, and the turbine and service buildings are nonseismic structures (see Figure 3.7-6). In the original analysis, each Class I structure was treated independently. For the SEP reevaluation the interconnected nature of the buildings was considered an important feature, especially in view of the lack of detailed original seismic design information. Therefore, both Class I and Class III buildings were included in the reanalysis model. Gilbert Associates, Inc., developed separate models for the auxiliary and control buildings in 1979. The basic assumptions and model properties for these two buildings were adopted and incorporated into the reanalysis.

The auxiliary, intermediate, turbine, control, diesel-generator, and service buildings form an interconnected U-shaped building complex that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which makes the building complex a complicated three-dimensional structural system. The compositions and interrelationships of the buildings in the complex are described in Section 3.8.4.

The principal lateral force-resisting systems of the interconnected building complex are the braced frames. Several such systems tie all buildings together to act as one three-dimensional structural system. It was, therefore, necessary to model these buildings in a single three-dimensional model to properly simulate interaction effects. The results of the reevaluation are discussed in Section 3.8.4.

3.7.2.9 Use of Constant Vertical Static Factors

Vertical responses in the SEP evaluation were obtained by taking 13% (0.2g × 2/3) of the dead load responses.

3.7.3 SEISMIC SUBSYSTEM ANALYSIS

3.7.3.1 Seismic Analysis Methods

3.7.3.1.1 Original Design

3.7.3.1.1.1 Piping and Tanks

Most of the original piping systems were analyzed by static methods, primarily the equivalent static method. Seismic input for these analyses were based on the Housner ground response spectra (Figures 3.7-1 and 3.7-2). Peak spectral accelerations were taken from the curves for those components for which the natural frequency was estimated. If natural frequencies were unknown, the maxima of the curves were used.

Exceptions to the static analysis approach included the analysis of

a. The residual heat removal system line from the reactor coolant system loop A to the containment penetration.

b. The main steam line from steam generator B to the containment penetration.

c. The reactor coolant system.

Two response spectrum analyses of the residual heat removal system line were performed. One analysis used the response spectra in Figures 3.7-1 and 3.7-2 as input; the other used a response spectrum that was a modification of the 0.5% damping spectrum in these figures to account for building effects at the steam-line elevation.

Both static and dynamic analysis were performed on the main steam line of loop B inside the containment. The modified response spectrum used for the residual heat removal system line analysis was also used for this dynamic analysis.

The reactor coolant system was qualified by tests using a plastic model. Input was a sinusoidal wave for the vertical direction and each of the two horizontal directions, independently. The plastic model output (mode shapes and frequencies) was then used as input, along with the Housner spectrum, to a three-dimensional mathematical model of the primary coolant loop.

The piping lines of the safety injection system were analyzed by selecting the peak of the 0.5% critical damping response spectra corresponding to the 0.2g maximum potential earthquake. Concentrated forces at selected locations on the pipe line were applied with the force equal to the product of the concentrated lumped mass and the maximum acceleration from the response spectra. Combination of bending stress, S_b, and torsional stress, S_t, is made according to the USAS B31.1 formula,

$$S = (S_b^{\,2} + 4\,S^{t2})^{1/2}.$$

The analysis of tanks was performed in the manner set forth in TID 7024, taking into account the possible dynamic effects resulting from the sloshing of the water. The techniques are set forth in Chapters 5 and 6 of TID 7024. Shell stresses and support stresses were limited to those permitted in the pressure vessel codes and the structural steel standards of AISC. Selected tanks were subsequently reanalyzed as part of the SEP (see Section 3.9.2.2.4).

Seismic Class I components were qualified on an individual and often generic basis. Qualification of the major equipment items (*Reference 1*), such as the steam generator, control rod drive mechanism, reactor internals, reactor vessel, and pressurizer, are summarized in the following.

3.7.3.1.1.2 Steam Generator

The original series 44 steam generators were evaluated to a set of generic loads including seismic. The seismic loads were based on envelope horizontal response spectra for 1% equipment damping shown on Figure 3.7-7 for the operating-basis and safe shutdown earthquakes. The generic curves were based on an envelope of floor response spectra of eleven plants with Westinghouse nuclear steam supply systems. The dynamic analyses are by the response spectrum method with the steam generator idealized by lumped masses inter-connected by three-dimensional beam elements (Figure 3.7-8). In 1996, the steam generators were replaced. See Section 5.4.2.3 for updated information.

3.7.3.1.1.3 Control Rod Drive Mechanisms

There are 29 equivalent model L-106 control rod drive mechanisms attached to the reactor vessel head adapters for the plant which were installed by PCR 2001-0042 during the 2003 refueling outage. The original model L-106 seismic analysis of the mechanisms consisted of two phases. The first phase involved comparing the results of a computer analysis of the mechanism for internal pressure and thermal loads with the ASME Section III stress allowances. The results of the comparison were used to derive the allowable seismic bending moment for the mechanism. In the second part of the analysis, a lumped mass beam model of the control rod drive mechanism system was used to calculate the bending moments at various locations on the assembly resulting from seismic loading. These calculated bending moments were compared with the allowable seismic bending moment for the mechanism.

The seismic design basis used for the original drive mechanisms was 0.8g in both the horizontal and vertical directions. For these loads the bending moments throughout the mechanism were below the allowable bending moments. The results of the second phase of the analysis were used in the design and analysis of the control rod drive mechanism seismic support mechanism.

Equivalent L-106 control rod drives provided by PCR 2001-0042 are evaluated for seismic conditions in *Reference 11*.

3.7.3.1.1.4 Reactor Internals

The Ginna reactor internals assembly is a standard 12-ft, two-loop assembly (Figure 3.9-9) that was qualified on a generic basis. The qualification analysis used a linear response spectra analysis with a lumped mass beam finite element model as shown on Figure 3.7-9. The input

for the analysis was the response spectra for the Kansai plant as shown on Figure 3.7-10. Two cases were considered in the analysis: first, where the modal contributions were combined by the square root of the sum of the squares and; second, where the modal contributions were summed by the absolute method. Two models were evaluated, one with horizontal and rotational stiffness representing the soil as shown on Figure 3.7-9 and the second model where a fixed base was assumed. In both cases, 5% damping was used for the concrete and 1% for the internals. In the vertical direction, a single degree of freedom model, uncoupled from the horizontal direction, was used. Stresses were obtained by adding horizontal and vertical responses absolutely.

3.7.3.1.1.5 Reactor Vessel

Seismic analysis of the reactor vessel was performed by applying a steady-state acceleration to the piping and calculating the resulting nozzle reactions. Stress calculations for the following three cases were performed:

a. Design seismic plus thermal loads.

b. No loss of function seismic plus thermal loads.

c. Design seismic plus thermal plus interaction loads.

Accelerations of 0.08g and 0.2g were used for design (operating-basis) and no-loss-of-function (safe shutdown) earthquakes, respectively.

3.7.3.1.1.6 Pressurizer

A stress report for the pressurizer was completed and issued in 1969. The report contained a seismic analysis of the pressurizer shell, the support skirt, the support skirt flange and the pressurizer support bolts. Loads for these evaluations were developed by combining the internal pressure, thermal loads, weight, upper head nozzle loads (i.e., spray, safety, and relief nozzles), and static seismic loads. The seismic analysis was conducted generically for the heaviest Westinghouse pressurizer model. Two cases were analyzed: an operating-basis earthquake and a safe shutdown earthquake. However, for both cases the safe shutdown earthquake acceleration of 0.48g horizontal and 0.32g vertical were used for evaluation. The accelerations were applied statically at the center of gravity of the pressurizer.

In 1973, a more detailed evaluation was performed of the pressurizer skirt and shell (*Reference 10*). For that evaluation the loads applied to the skirt were the equivalent of 10 times the operating-basis earthquake loads and 14 times the safe shutdown earthquake loads outlined above. The results contained only the primary membrane and bending stresses. The 4-in. nozzles of the pressurizer were also evaluated. For the nozzle evaluations internal pressure stresses were combined with the stresses resulting from the pipe loads including seismic loads and the results were compared with ASME Code allowables. Design condition allowables were used for analysis involving operating-basis earthquake and emergency condition allowables were used for the safe shutdown earthquake.

The heaters for the pressurizer were qualified on a generic basis (*Reference 10*). The qualification procedure used an equivalent load of 37.5g for the safe shutdown earthquake and

30g for the operating-basis earthquake. The fundamental frequency of the heater rods was greater than 33 Hz.

3.7.3.1.2 Seismic Reevaluation

For the SEP reevaluation, the seismic input was defined by means of in-structure or floor response spectra which were generated either by the direct method or by means of a time-history analysis. The spectra were normally smoothed and the peaks broadened to account for modeling and material uncertainties.

In-structure response spectra were generated for both the interconnected building complex and the containment building. In both cases, in-structure spectral curves were smoothed, and the peaks were widened ±15% in accordance with current practice. As described in Section 3.7.2.1, two mathematical models of the interconnected building complex were analyzed to bracket the behavior of the braced frames: a half-area model that simulated buckled bracing; and a full-area model that simulated unbuckled bracing. Envelopes of spectra generated from the two models by the direct method were used for reanalysis of equipment. In-structure response spectra for the containment interior structures were generated from time-history analyses of the mathematical model.

Response spectra were generated at the equipment locations and floor centers of gravity indicated in Table 3.7-3 and shown in Figure 3.7-11. At each location, two orthogonal horizontal spectral components were computed at three different equipment damping ratios (3%, 5%, and 7%). Since the vertical dynamic amplification was judged to be negligible, all vertical floor spectra were considered to be the same as the ground input spectra with 0.13g peak acceleration.

The in-structure response spectra generated for equipment analysis are shown in Figures 3.7-12 through 3.7-28. The horizontal in-structure spectra of the containment interior structure are oriented in the directions of S62E and N28E. Spectra outside the containment building are in the north-south and east-west directions.

For mechanical and electrical equipment, a composite 7% equipment damping was used in the evaluation for the 0.2g safe shutdown earthquake. For piping evaluation, the equipment damping associated with the safe shutdown earthquake was limited to 3%.

For the SEP reevaluation, components were grouped as active or passive and rigid or flexible. Then, a representative sample of each group was evaluated to establish the seismic design factor of safety or degree of adequacy for that group. In this way, seismic design factors within groups of similar components were established without the detailed reevaluation of hundreds of individual components within each group.

A representative sample of components was selected for review by one of two methods:

A. Selection based on a walk-through inspection of the Ginna facility by the NRC SEP seismic review team which selected components as to the potential degree of seismic fragility for components of that category.

B. Categorization of the safe shutdown components into generic groups such as horizontal tanks, heat exchangers, and pumps; vertical tanks, heat exchangers, and pumps; motor control centers and motors.

Based on the detailed review of the seismic design adequacy of the representative components discussed above, conclusions were developed as to the overall seismic design adequacy of Seismic Category I equipment installed in Ginna Station.

The seismic analysis of the components selected for the SEP review, as well as the components that are representative of the generic groups of safety-related components is described in Section 3.9.2.2.4 for mechanical components and Section 3.10.2.1 for electrical components. Tables 3.9-12 and 3.10-2 contain the list of these components and the reason for their selection.

3.7.3.2 Basis for Selection of Frequencies

The components and distribution systems were designated as flexible or rigid in developing the magnitude of the seismic input for component evaluation. Designation of rigid or flexible components for Ginna was complicated by the fact that many components were supported in the auxiliary and reactor buildings by concrete structures, which had high fundamental frequencies between 15 and 25 Hz, while other components were supported by steel superstructures, which had fundamental frequencies between 6 and 11 Hz. Equipment supported at or near grade was subject to nearly the ground response, with a peak response acceleration in the 2 to 9 Hz range. Therefore, components that had fundamental frequencies greater than 20 Hz and were located on grade or supported by structural steel could be considered rigid since there was little amplification in this region of the applicable response spectra. Similar components supported by concrete structures would be at or near building resonance and were considered flexible. For flexible components whose fundamental frequencies were less than twice the dominant building frequencies, the seismic inertial accelerations were typically 5 to 15 times the safe shutdown earthquake peak ground acceleration, depending on:

- Potential resonance with the supporting building structure.
- Structure and equipment damping levels.
- Equipment support elevations.

3.7.3.3 Use of Equivalent Static Analysis

Equivalent static analysis was used for the seismic analysis of several components. For those components that were classified as rigid, with a fundamental frequency of 33 Hz or more, peak floor accelerations were used. For flexible components peak response acceleration from the appropriate in-structure response spectra were used.

3.7.3.4 Three Components of Earthquake Motion

Response spectra were generated at the equipment locations and floor centers of gravity. At each location, two orthogonal horizontal spectral components were computed. Since the vertical dynamic amplification was judged to be negligible, all vertical floor spectra were considered to be the same as the ground input spectra with 0.13g peak acceleration.

3.7.3.5 Combination of Modal Responses

The various Seismic Category I mechanical equipment and components were seismically qualified by analyses in which static loads equivalent to the accelerations in the response spectra were applied. As such, the question of combining modal responses did not exist. The same conclusion is true for those piping systems which were analyzed either using model techniques or by using equivalent static loads. The three piping systems that were analyzed using response spectrum are the (1) residual heat removal system, (2) main steam line, and (3) charging system. The original analyses used the square root of the sum of the squares of modal components. However, in response to NRC IE Bulletin No. 79-07, when a reanalysis was performed, the absolute sum of the modal components was used. Several additional seismic analyses of piping systems were performed subsequently. Either the square root of the sum of the squares or the absolute sum method were used for combining the modal responses. Both are acceptable.

3.7.3.6 Analytical Procedures for Piping

The original Ginna Station design did not utilize dynamic computer analyses for seismic qualification of Seismic Category I piping. The reactor coolant system piping was seismically qualified using a combination of model testing and analysis. Seismic Category I piping 2-1/2 in. nominal pipe size and larger was seismically qualified using equivalent static analyses.
Seismic Category I piping 2-in. nominal pipe size and smaller was seismically qualified using support spacing tables. Dynamic analysis of sections of the A residual heat removal and B main steam piping were performed solely to verify the equivalent static analysis method.

However, modifications or additions to piping systems at Ginna Station since initial operation were seismically qualified using dynamic analyses. Some small piping was seismically qualified using equivalent static analysis or spacing table techniques.

As a result of IE Bulletin No. 79-07, new dynamic analyses were performed for sections of the A residual heat removal, B main steam, and charging system piping. The reanalyses were based on as-built piping system isometrics and support information. The details of these analyses are described below and in Section 3.9.2.1.2.

Additional analyses were also performed for the pressurizer safety and relief lines. Details of the analytical methods and analysis are provided below and in Section 3.9.2.1.4.

Reanalysis of critical safety-related piping 2-1/2 in. and larger was performed under the Seismic Piping Upgrade Program discussed in Section 3.7.3.7.

<u>3.7.3.6.1 Residual Heat Removal System Line from Reactor Coolant System Loop A to Containment</u>

A sketch of this run is shown in Figure 3.7-29. Idealized lumped mass models were developed and analyzed dynamically. The analysis was made by assigning three translational and three rotational degrees of freedom to each lumped mass point with each mass point representing a geometrically proportional amount of the total system mass. Elastic characteristics of the system include the translational and rotational stiffnesses; the rotational elastic characteristics are carried into the reduced stiffness matrix that is inverted

and forms, with the mass matrix, the dynamic matrix. Following normal mode theory, the natural frequencies, mode shapes, and participation factors are computed to yield the dynamic system characteristics. These characteristics are then combined with the appropriate shock spectra to yield the D'Alembert reverse effective forces on the system for each mode. The modal forces are then used to compute the stresses per mode. The stresses are summed on a root mean square basis for final comparison to code allowable stresses.

More than 70 modes have been analyzed for their response to earthquake excitation. The Housner 0.5% critical damping ground response spectrum normalized to 0.2g was used. This spectrum was considered adequate because of the location of this pipe run, low in the containment.

For the location of maximum stress, the stress values were calculated at three points on the pipe cross section, the bottom, one side 90 degrees away, and half way between these two. First the stresses due to the two bending moments and one torsional moment on the pipe were calculated. Then for each of the three points, the root mean square of the stresses acting at the point for the significant modes (first three) was calculated. To this was added the dead weight stress, and then the result multiplied by the stress intensification factor, as the location of maximum stress was the end of an elbow. The pressure stress was added to this result in order to obtain the total additive longitudinal stress. The total maximum stress was calculated, considering the torsional shear stress and using the formula for maximum principal stresses.

Re-analysis of the Residual Heat Removal system piping was performed under the Seismic Upgrade program. Additional information may be found in Section 3.7.3.7.

3.7.3.6.2 Steam Line from Steam Generator B to Containment

A dynamic modal analysis was run on the steam line of loop B on lines similar to that just described. The lumped mass model of the piping, supports, and snubbers are shown in Figure 3.7-30.

Re-analysis of the Main Steam system piping was performed under the Seismic Upgrade program. Additional information may be found in Section 3.7.3.7.

3.7.3.6.3 Pressurizer Safety and Relief Lines

3.7.3.6.3.1 Analytical Methods

The analytical methods used to obtain a piping deflection solution consisted of the transfer matrix method and stiffness matrix formulation.

The piping system models, constructed for the WESTDYN computer program, were represented by an ordered set of data which numerically describes the physical system.

The spatial geometric description of the piping model was based upon the isometric piping drawings and equipment drawings. Node point coordinates and incremental lengths of the members were determined from these drawings. Node point coordinates are put on network cards. Incremental member lengths were put on element cards. The geometrical properties along with the modulus of elasticity (E), the coefficient of thermal expansion (a), the average

temperature change from the ambient temperature (delta T), and the weight per unit length (w) were specified for each element. The supports were represented by stiffness matrices which define restraint characteristics of the supports. Plotted models for various parts of the safety and relief valve discharge piping are shown in Figure 3.7-31, Sheets 1 through 5.

3.7.3.6.3.2 Transfer Matrix Method

The static solutions for deadweight and thermal loading conditions were obtained by using the WESTDYN computer program. The fundamental transfer matrix for an element is determined from its geometric and elastic properties. If thermal effects and boundary forces are included, a modified transfer relationship is defined as follows:

$$\begin{bmatrix} T_{11} & T_{12} \\ T_{12} & T_{22} \end{bmatrix} \begin{bmatrix} \Delta_0 \\ F_0 \end{bmatrix} + \begin{bmatrix} \delta_t \\ f_t \end{bmatrix} - \begin{bmatrix} \Delta_i \\ F_i \end{bmatrix}$$

(Equation 3.7-2)

or

$T_1B_o + R_1 = B_1$

where the T matrix is the fundamental transfer matrix as described above, and the R vector includes thermal effects and body forces. This B vector for the element is a function of geometry, temperature, coefficient of thermal expansion, weight per unit length, lumped masses, and externally applied loads.

The overall transfer relationship for a series of elements (a section) can be written as follows:

$B_1 = T_1B_o + R_1$

$B_2 = T_2B_1 + R_2 = T_2T_1B_o + T_2R_1 + R_2$

$B_3 = T_3B_2 + R_3 = T_3T_2T_1B_o + T_3T_2R_1 + T_3R_2 + R_3$

or

$$B_n = \pi \cdot \overset{n}{\underset{1}{T_r}} \cdot B_0 + \sum_{r=2}^{n} \left[\left(\pi \cdot \overset{n}{\underset{r}{T_r}} \right) \cdot R_{r-1} \right] + R_n$$

(Equation 3.7-3)

3.7.3.6.3.3 Stiffness Matrix Formulation

A network model was made up of a number of sections, each having an overall transfer relationship formed from its group of elements. The linear elastic properties of a section were used to define the characteristic stiffness matrix for the section. Using the transfer relationship for a section, the loads required to suppress all deflections at the ends of the section arising from the thermal and boundary forces for the section were obtained. These loads were incorporated in the overall load vector.

After all the sections were defined in this manner, the overall stiffness matrix, K, and associated load vector needed to suppress the deflection of all the network points was determined. By inverting the stiffness matrix, the flexibility matrix was determined. The flexibility matrix was multiplied by the negative of the load vector to determine the network point deflections due to the thermal and boundary force effects. Using the general transfer relationship, the deflections and internal forces were then determined at all node points in the system. The support loads, F, were also computed by multiplying the stiffness matrix, K, by the displacement vector, δ, at the support point.

The lumping of the distributed mass of the piping systems was accomplished by locating the total mass at points in the system which appropriately represented the response of the distributed system. Effects of the pressurizer motion on the piping system were obtained by modeling the mass and the stiffness characteristics of the equipment in the overall system model.

The supports were again represented by stiffness matrices in the system model for the dynamic analysis. Mechanical shock suppressors which resist rapid motions were considered in the analysis. The solution for the seismic disturbance employed the response spectra method.

From the mathematical description of the system, an overall stiffness matrix, K, was developed from the individual element stiffness matrices using the transfer matrix, K_R, associated with mass degrees of freedom only. From the mass matrix and the reduced stiffness matrix, the natural frequencies and the normal modes were determined. The modal participation factor matrix was computed and combined with the appropriate response spectra value to give the modal amplitude for each mode. Since the modal amplitude was shock direction dependent, the total modal amplitude was obtained conservatively by the absolute sum of the contributions for each direction of shock. The modal amplitudes were then converted to displacements in the global coordinate system and applied to the corresponding mass point. From these data the forces, moments, deflections, rotation, support reactions, and piping stresses were calculated for all significant modes.

The seismic response from each earthquake component was computed by combining the contributions of the significant modes.

3.7.3.7 Seismic Piping Upgrade Program

3.7.3.7.1 Program Scope

Commencing in 1979, a reanalysis of selected Class 1 piping systems was performed for the seismic piping upgrade program, which resulted from SEP Topic III-6.

The purpose of this program was to upgrade certain Seismic Category I Piping systems at Ginna Station to more current requirements and to provide a seismic data base for use with modifications, the inservice inspection program, and NRC requests for information.

Portions of the following piping systems were included in this program:

- Reactor coolant system
- Main steam
- Main feedwater
- Auxiliary feedwater
- Safety injection
- Residual heat removal
- Containment spray
- Steam generator blowdown
- Service water (SW)
- Component cooling
- Standby auxiliary feedwater
- Chemical and volume control
    1. Auxiliary spray
    2. Letdown
    3. Seal-water
    4. Charging

3.7.3.7.2 Piping Selection Criteria

The criteria for the selection of lines to be included in the program were as follows:

A. Only piping that is considered Seismic Category I as identified by the safety class and seismic boundaries shown on the Ginna Station P&IDs.

B. Main runs of piping included shall be based on the following criteria:

1. Main runs of piping which are 2-1/2 in. and larger and critical 2-in. piping.
2. Main runs that provide the fluid flow path to/or from equipment required for safe shutdown and loss-of-coolant-accident mitigation based on the Systematic Evaluation Program. Equipment does not include instrumentation.

3. Selected additional main runs, which are a primary part of the systems included in the upgrade program.

C. Branch lines included shall be based on the following criteria:

1. Branch lines shall be included in the analyses as necessary to determine the local effects of the branch lines on the main runs and to ensure adequate flexibility exists in the branch line to prevent local overstress in the branch due to main run displacements.
2. Branch lines whose section modulus is greater than 15% of the main run section modulus shall be included in the analysis for an appropriate distance and/or number of supports
3. Branch lines whose section modulus is less than 15% of the main run section modulus do not need to be explicitly included in the analysis.

3.7.3.7.3 Selected Lines

The lines selected to be analyzed and modified as necessary are identified below.

The load combinations, associated stress limits, and conclusions for these lines are discussed in Section 3.9.2.1.8. Pipe supports for these lines are discussed in Section 3.9.3.3.

3.7.3.7.3.1 Reactor Coolant System

a. Primary loop.
b. Surge line.
c. Pressurizer spray lines from the cold legs to the pressurizer.

3.7.3.7.3.2 Main Steam

a. The 30-in. lines from both steam generators through the penetrations and up to the main steam isolation valves.
b. Inlet piping up to safety and relief valves.

3.7.3.7.3.3 Main Feedwater

The 14-in. lines from the steam generators through the penetrations and up to check valves 3992 and 3993.

3.7.3.7.3.4 Auxiliary Feedwater

a. The discharge lines from the two motor-driven pumps and the turbine-driven pump up to the main feedwater connections, and branches to valves 4304, 4310, including by-pass lines containing valves 4493 and 4494.
b. The condensate and service water (SW) suction lines from the pumps to check valves 4014, 4016, 4017, and to valves 4013, 4027, and 4028.

3.7.3.7.3.5 Safety Injection

a. The 10-in. safety injection accumulator discharge lines to the cold legs.
b. Safety injection pump suction lines from the refueling water storage tank (RWST) through valves 896A and B and 825A and B to the three pumps.
c. The safety injection pump discharge lines from the three pumps to the safety injection accumulator discharge lines and to the two hot leg connections.
d. The boric acid lines from the boric acid storage tanks to the safety injection pump suction line.
e. The 4-in. alternate safety injection suction line from valves 1816A and B to the pump.
f. The 10-in. low-head safety injection suction from the refueling water storage tank (RWST) to valve 854.
g. The 6-in./8-in. header from the refueling water storage tank (RWST) to valves 857A, B, and C.
h. The 8-in. suction lines from containment sump B to valves 850A and B and the 6-in. branch lines to valves 1810A and B.
i. The low head safety injection lines from valves 852A and B to the reactor coolant system.

3.7.3.7.3.6 Residual Heat Removal

a. The 10-in. suction lines from the loop A hot leg to the two residual heat removal pumps.
b. From valves 850A and B to the residual heat removal pumps.
c. From valve 854 to the suction header.
d. The two pump discharge lines through heat exchangers and to the common 10-in. return.
e. The 10-in. return through penetration P111 and to the B cold leg.
f. The discharge cross-connect including valves 709C and D.
g. The heat exchanger bypass line including valves 712A and B.
h. The two lines from the residual heat removal heat exchanger outlets to valves 857A and B and 1816B.
i. The recirculation line from the residual heat removal return through valve 822B to the residual heat removal suction line.
j. The two lines from the residual heat removal return to valves 852A and B.

3.7.3.7.3.7 Containment Spray

a. The two suction lines from the refueling water storage tank (RWST) header to the spray rings.
b. The two containment spray pump discharge lines and spray rings.
c. The two eductor lines from the containment spray pump discharges to the pump suctions.
d. The spray additive lines from the tank through valves 836A and B and to the two eductors.

3.7.3.7.3.8 Chemical and Volume Control System

a. The auxiliary pressurizer spray line from the connection at the regenerative heat exchanger outlet line to the pressurizer spray line.

b. The letdown line from the reactor coolant system through the regenerative heat exchanger, through the nonregenerative heat exchanger, through valve TCV 145 to the volume control tank.

c. The 4-in. header from the volume control tank and the 3-in. suction lines to the three charging pumps.

d. The three charging pump discharge lines to the acoustic filter.

e. The 2-in. charging lines from the acoustic filter through the regenerative heat exchanger to both the hot and cold leg connections.

f. The 3-in. seal water header from the acoustic filter and the 2-in. lines to the reactor coolant pump seals.

g. The 2-in. seal-water return lines from the reactor coolant pump seals and the 3-in. return header through the seal water heat exchanger to the volume control tank. This includes ¾-in. piping through flow transmitters 175, 176, 177, and 178.

h. The 4-in. line from the refueling water storage tank (RWST) through valves LCV 112B and 358 to the charging pump suction header.

3.7.3.7.3.9 Steam Generator Blowdown

The 2-in. and 3-in. lines from the steam generators through the penetrations to the isolation valves.

3.7.3.7.3.10 Service Water System

a. The inlet piping to both diesel generators including the cross-connection between the diesels, the 16-, 14-, and 10-in. supply to the turbine building up to valve 4613.

b. The outlet piping from both diesel generators to an anchor point outside the diesel generator room.

c. The 20-in. supply lines and header inside the auxiliary building.

d. The 18-, 14-, and 6-in. supply lines from the 20-in. header to the two component cooling water heat exchangers and the spent fuel pool (SFP) heat exchanger.

e. The normal discharge lines from the component cooling water heat exchangers and the spent fuel pool (SFP) heat exchangers including the 20-in. discharge line inside the auxiliary building.

f. The 3-in. supply and normal discharge headers to and from the safety injection system pumps and equipment coolers in the auxiliary building (includes piping through valves 4738, 4739, and 4739A).

g. The 16-in. and 14-in. supply headers inside the intermediate building. Including piping through valves 4640, 4623, 4639, and 4756.

h. The 10-in. supply to the turbine building up to valve 4614.

i. The 4-in. supply lines to the preferred auxiliary feedwater pumps.

j. The 2-1/2 in. and 8-in. supply and discharge lines to and from the 1A, 1B, 1C, and 1D containment ventilation cooling coils and fan motors.

k. The 2-1/2 in. supply and discharge lines for the reactor compartment coolers, including piping through valves 4625, 4626, and 4624.

l. The 6 and 4-in. supply to the air conditioning water chillers up to the isolation valves 4663 and 4733.

m. The common discharge header for the ventilation coolers up to an anchor point outside the intermediate building.

n. The service water (SW) pump discharge piping inside the screen house including the 4-in. cross-tie.

o. The 4-in. supplies from the loop to the C and D standby auxiliary feedwater (SAFW) pumps including the 4-in. cross-tie.

p. The 4-in. test suction line through valves 9707A and B and the 1-1/2-in. branch line through valves 9720A and B.

q. The 1-1/2-in. supply to standby auxiliary feedwater room cooling units A and B.

r. The discharge from the standby auxiliary feedwater room cooling units to the 14-in. normal return line and to the 20-in. alternative discharge line.

3.7.3.7.3.11 Component Cooling Water

a. The 14-in. suction header and 10-in. suction lines to the component cooling water pumps. The component cooling water pump discharge lines to the component cooling water heat exchangers.

b. The 4-in. and 6-in. component cooling water surge tank line.

c. The 10-in. and 14-in. supply headers out of the component cooling water heat exchangers.

d. The 10-in and 14-in. supply lines to both residual heat exchangers.

e. The 10-in. and 14-in. return lines from the residual heat exchangers to the component cooling water pumps suction header.

f. The 2-in. supply and return lines to the residual heat removal pump coolers.

g. The 14-in. and 8-in. supply and return headers servicing the reactor coolant pumps and reactor supports.

h. The 3-in. and 4-in. supply and return lines to both reactor coolant pump motors.

i. The 6-in. supply and return lines for the reactor supports from the 2-in. headers to penetrations 130 and 131.

j. The 2-in. supply and return lines for the excess letdown heat exchanger from the 8-in. header to penetrations 124 and 126.

k. The 6-, 4-, and 2-in. supply and return lines for the nonregenerative heat exchanger and the seal-water heat exchanger.

l. The 2-in. supply and return lines for both the containment spray and safety injection pumps.

3.7.3.7.3.12 Standby Auxiliary Feedwater

The 3-in. discharge from standby auxiliary feedwater pumps (SAFW) C and D through the penetrations to the A and B main feedwater lines, including the 3-in. cross-tie and the 1-1/2-in. lines to both minimum flow orifices.

3.7.3.7.4 Codes and Standards

The original design of Seismic Category I piping at Ginna was done to USAS B31.1.

The piping code, USAS B31.1, was updated on June 30, 1973, revising the piping stress analysis formulas and stress intensification factors. The primary stress equations are similar to those given in the ASME Section III Code of that time. The stress intensification factors given in the 1973 version of the code were expanded to include more fittings than in the previous edition, as well as higher values for certain existing fittings. In the piping system Seismic Upgrade Program, the ANSI B31.1 Code, Summer 1973 Addenda, was used primarily, with the following exception. The piping criteria did not consider the B31.1 Summer 1973 Addenda stress intensification factors for butt and socket welds, since they are constrictively higher than the original design basis 1967 B31.1 stress intensification factors.

The design, materials, fabrication, installation, and examination of piping modifications required as a result of this reanalysis are done in accordance with ANSI B31.1.

3.7.3.7.5 Analytical Procedures

The analytical procedure used for the piping analysis is described in the following.

3.7.3.7.5.1 General

The piping/support systems are evaluated incorporating three-dimensional static and dynamic models which include the effects of the supports, valves and equipment. The static and dynamic analysis employs the displacement method, lumped parameters, stiffness matrix formulation, and assumes that all components and piping behave in a linear elastic manner.

The response spectra model analysis technique is used to analyze piping.

Seismic analyses incorporate the Gilbert Associates, Inc., developed response spectra for both the operating-basis and safe shutdown earthquake cases. Spectra are derived from buildings and elevations applicable to the individual analysis lines.

The seismic analyses are based on the operating-basis earthquake and safe shutdown earthquake being initiated while the plant is at the normal full power condition.

3.7.3.7.5.2 Damping Values

The seismic pipe stresses are determined using seismic loads generated considering the piping systems to have the following damping values.

Small diameter piping systems, diameter less than 12-in.

For operating-basis earthquake the damping value is 1%.

For safe shutdown earthquake the damping value is 2%.

Large diameter piping systems, diameter equal to or greater than 12 in.

For operating-basis earthquake the damping value is 2%.

For safe shutdown earthquake the damping value is 4%.

For a coupled system with different damping and different structural elements, such as would be the case in analysis with coupling between concrete structures and welded steel components, the method used for damping is either to (1) use the damping that results in the highest load, (2) inspect the mode shapes to determine which modes correspond with a particular structural element, and then use the damping associated with that element having predominant motion, or (3) use composite modal damping value for each mode, which is calculated by weighting the damping in each subsystem by the amount of strain energy in each subsystem.

An acceptable alternative to the listed damping values is to apply the values given in ASME Code Case N-411. These values are applicable to both Operating Basis Earthquakes and Safety Shutdown Earthquakes and are independent of pipe diameter.

3.7.3.7.5.3 Combination of Modal Responses

For piping systems interconnected between floors of a structure and/or building, the envelope of the respective floor response spectra is used in the seismic analysis.

The piping was analyzed for the simultaneous occurrence of two horizontal components and one vertical earthquake input component. The response spectra associated with each earthquake component are applied in each direction separately. The combined modal response for each item of interest (e.g., force, displacement, stress) resulting from each component analysis will be combined by the square root of the sum of the squares method.

For each seismic analysis, the total seismic response is obtained by combining the individual modal response (in each direction) utilizing the square root of the sum of the squares method. The combination of modal responses is in accordance with one of the following:

a. Regulatory Guide 1.92.
b. Subsection 3.7.3.4 of Westinghouse RESAR-41, as described below.
c. NUREG 1061 Volume 4, Section 2, as described below.

For systems having modes with closely spaced frequencies, the above method is modified to include the possible effect of these modes. The groups of closely spaced modes are chosen such that the difference between the frequencies of the first mode and the last mode in the group does not exceed 10% of the lower frequency. Combined total response for systems which have such closely spaced modal frequencies are obtained in accordance with Regulatory Guide 1.92 or, as an acceptable alternative, the following method.

Frequency groups are formed starting from the lowest frequency and working toward successively higher frequencies. No frequency should be included in more than one group.

The resultant unidirectional response for systems having such closely spaced modal frequencies is obtained by the square root of (a) the sum of the squares of all modes, and (b) the product of the responses of the modes in various groups of closely spaced modes and associated coupling factors. The mathematical expression for this method with R as the item of interest is:

$$R_i^2 = \sum_{j=1}^{N} R_{ij}^2 + 2 \sum_{j=1}^{S} \sum_{K=Mj}^{Nj-1} \sum_{l=K+1}^{Nj} R_{iK} R_{il} \varepsilon_{Kl} , for. l \neq K$$

(Equation 3.7-4)

where:

	R_i =	resultant unidirectional response for direction i; i=1, 2, 3
	R_{ij} =	absolute value of response of direction i, mode j
	N =	total number of modes considered
	S =	number of groups of closely spaced modes
	M_j =	lowest modal number associated with group j of closely spaced modes
	N_j =	highest modal number associated with group j of closely spaced modes
	K =	coupling factor with

$$\varepsilon_{Kl} = \left[1 + \left(\frac{\omega'_K - \omega'_l}{\beta'_K \omega_K + \beta'_l \omega_l}\right)^2\right]^{-1}$$ (Equation 3.7.5)

$$\omega'_K = \omega_K [1 - (\beta'_K)^2]^{1/2}$$ (Equation 3.7.6)

$$\beta'_K = \beta_K + \frac{2}{\omega_K t_d}$$ (Equation 3.7.7)

where:

	ω_K =	frequency of closely spaced mode K (rad/sec)
	β_K =	fraction of critical damping in closely spaced mode K
	t_d =	duration of the earthquake (seconds)

Total response, R_T is

$$R_T = \left[\sum_{i=1}^{3} R_i^{\,2} \right]^{1/2} \quad \text{(Equation 3.7-8)}$$

For "Multiple Supporting Piping Systems" utilizing the independent support motions response spectrum method of analysis, the total response for the system is obtained in accordance with NUREG 1061 Volume 4, Section 2, as summarized in the following:

aa. For inertial or dynamic components.

The inertial or dynamic component group responses for each direction are combined by the absolute sum method. Modal and directional responses for these component groups are combined by the square root sum of the squares method without considering closely spaced frequencies.

bb. For pseudostatic components (e.g., anchor motion).

Calculate the maximum absolute response for each support group and then combine their effects by the absolute sum method for each input direction. Directional responses are then combined by the square root sum of the squares method.

cc. For the total response.

Determine the total response by combining the dynamic and pseudostatic responses by the square root sum of the squares method. Since consideration of closely spaced frequencies need not be considered when applying this analysis method, either directional or modal components may be combined first.

dd. High frequency modes.

High frequency modes (>33Hz) are combined algebraically as described in NUREG 1061 Volume 4, Section B.2 of Appendix B. The effects of the high frequency modes are combined with the effects of the low frequency modes (≤33Hz) by the square root of the sum of the squares method.

3.7.3.7.5.4 Safe Shutdown Earthquake Stresses

The analyses performed for piping and supports do not include stresses resulting from safe shutdown earthquake induced differential motion. These stresses are secondary in nature, based on ASME code rules for piping (NB-3652, NB-3656, F-1360) and component supports (NF-3231). The safe shutdown earthquake, being a very low probability single occurrence event, is treated as a faulted condition. Therefore, consistent with present ASME philosophy, the secondary stresses associated with the safe shutdown earthquake induced differential motion are not evaluated when performing seismic analysis per the response spectrum method. The basic characteristic of these stresses is that they are self-limiting. Local yielding and minor distortions will satisfy the initial conditions that caused the stress to occur. Operating-basis earthquake induced differential motion is considered.

3.7.3.7.5.5 Small Piping Analysis

For small piping (2 in. and smaller) as an option to dynamic analysis, either the equivalent dynamic or static rigid range approach can be used. If the small piping system has a low operating temperature, then the pipe lines can be analyzed using equivalent static loads based on spacing table techniques. The static rigid range approach is used for rigid piping systems, which are defined as having natural frequencies greater than 33 Hz. In this case, the piping system is analyzed with static equivalent loads corresponding to acceleration in the rigid range of the applicable response spectrum curves. Both horizontal and vertical static equivalent loads are applied to rigid piping systems. The response of the piping system for two orthogonal horizontal directions and one vertical direction are combined on a square root of the sum of the squares basis.

For any piping that can be shown to be rigid (lowest natural frequency greater than 33 Hz), as an option to performing a dynamic analysis, the static rigid range approach may be used.

3.7.3.7.5.6 Branch Line Analysis

The following branch line analytical procedure and criteria are used.

a. The branch line is not included in the run model if its section modulus is 15% or less of the run section modulus.

b. For branch lines which have section moduli greater than 15% of the run section modulus, the branch line is modeled initially for a distance of 15 ft 0 in. If it is later determined by the piping analyst that additional modeling information is required, it is provided and included within the analysis model.

c. In the run analysis where the branch line has not been included, the branch allowable bending moments are included. Using B31.1 Summer 1973 Addenda, Formula 12, the branch allowable moment is expressed as follows:

$$= \frac{Z_B}{0.75i}\left[KS_h - \left(\frac{PD_0}{4 \cdot t_n}\right)_R\right]$$

M_{BR} = Branch Allowable Moment (Equation 3.7-9)

d. For branch lines that are not included in the model, supports within 10 ft of the run are noted since a support near the run pipe could affect the branch line flexibility.

3.7.3.7.5.7 Piping Beyond Scope of Upgrade Program

Piping which extends beyond the scope of the seismic upgrading program effort is included within the analysis only as it affects fluid lines within scope. In general, piping is modeled for a distance which covers a minimum of one rigid support in each of the three global directions. Case-by-case judgments are made when the above is insufficient or infeasible.

Out-of-scope piping is analyzed to the same general guidelines and criteria as the in-scope piping, once its inclusion has been deemed necessary. Analysis of nonseismic portions of the out-of-scope piping may be done to allowable stresses equivalent to the ASME Code Service

Level D allowables, providing the in-scope piping meets all seismic upgrade criteria requirements. Piping or support modifications are recommended for the out-of-scope segments when the qualification and/or safe operation of the upgraded piping mandates. Support load evaluations comply with the above guidelines and criteria established for the piping being supported.

3.7.3.7.6 Piping System Models

Piping Modeling Techniques for Static Analysis

The piping system models are represented by an ordered set of data, which numerically describes the physical system.

The spatial geometric description of the piping model is based upon the as-built isometric piping drawings and equipment drawings. Node point coordinates and incremental lengths of the members are determined from these drawings. Node point coordinates are input on network cards. Incremental member lengths are input on element cards. The geometrical properties along with the modulus of elasticity, E, the coefficient of thermal expansion, α , the average temperature change from ambient, delta T, and the weight per unit length, w, are specified for each element. The supports are represented by stiffness matrices, which define restraint characteristics of the supports.

A network model is made up of a number of sections, each having an overall transfer relationship formed from its group of elements. The linear elastic properties of the section are used to define the characteristic stiffness matrix for the section. Using the transfer relationship for a section, the loads required to suppress all deflections at the ends of the section arising from the thermal and boundary forces for the section are obtained. These loads are incorporated into the overall load vector.

After all the sections have been defined in this manner, the overall stiffness matrix, K, and associated load vector, to suppress the deflection of all the network points, is determined. By inverting the stiffness matrix, the flexibility matrix is determined. The flexibility matrix is multiplied by the negative of the load vector to determine the network point deflections due to the thermal and boundary force effects. Using the general transfer relationship, the deflections and internal forces are then determined at all node points in the system. The support loads, F, are also computed by multiplying the stiffness matrix, K, by the displacement vector,
δ , at the support point.

The models used in the static analysis are modified for use in the dynamic analyses by including the mass characteristics of the piping and equipment.

The lumping of the distributed mass of the piping systems is accomplished by locating the total mass at points in the system which will appropriately represent the response of the distributed system. Effects of the equipment motion are obtained by modeling the mass and the stiffness characteristics of the equipment in the overall system model when required.

The supports are again represented by stiffness matrices in the system model for the dynamic analysis. Hydraulic shock suppressors that resist rapid motions are considered in the analysis.

From the mathematical description of the system, the overall stiffness matrix, K, is developed from the individual element stiffness matrices using the transfer matrix, K_R, associated with mass degrees-of-freedom only. From the mass matrix and the reduced stiffness matrix, the natural frequencies and the normal modes are determined.

The effect of eccentric masses, such as valves and extended structures, are considered in the seismic piping analyses. These eccentric masses are modeled in the system analysis and the torsional effects caused by them are evaluated and included in the total system response. The total response must meet the limits of the criteria applicable to the safety class of the piping.

3.7.3.7.7 Valve Model

Valves are included in the piping system model. The model employed reflects non-rigid behavior as well as rigid behavior. For rigid valves, the model used consists of a rigid beam element from the center of the run pipe to the center of gravity of the valve. The mass of the valve should be located at the valve center of gravity. For non-rigid valves, the model should have two masses.

3.7.3.7.8 Equipment Model

Where the stiffness and mass of the equipment attached to the piping will influence the piping system being analyzed, the piping model must include the equipment effect. This is accomplished by including in the piping model a model of the equipment to the detail necessary.

3.7.3.7.9 Interaction Effects

Interaction of other piping systems is considered when their response will affect the response of the line being analyzed. The reactor coolant loop is included in the piping system model to the extent of detail required. If the lines being analyzed are relatively small diameter and/or low temperature, the reactor coolant loop need not be included in the model. This is because these lines are so flexible that the reactor coolant loop deflection will not include significant stresses in the lines, or that the reactor coolant loop response characteristics will not cause exciting forces different from those associated with the inner containment building.

Where branch piping is attached to the piping being analyzed, its effect on the piping of interest is accounted for by modeling in accordance with the criteria for branch lines given earlier.

3.7.3.7.10 Support Model

Supports are modeled as equivalent stiffness matrices within the piping analysis models (Section 3.7.3.7.6).

3.7.3.7.10.1 Deviations

Deviations from the analyzed support design parameters are permissible from an analysis standpoint provided the following acceptability guidelines are maintained.

a. Support stiffnesses.

1. Increasing the stiffness of a previously rigid support is acceptable. Rigid is defined in Section 3.9.3.3.3.3.
2. Revisions when original stiffness is below rigid are acceptable when revised values are ±15% of original stiffness.

b. Support locations.

Acceptable Deviations:

1.

Pipe Size	**Tolerance**[a]
≤4 in.	Greater of nominal pipe diameter or 3 in.
≤6 in.	Nominal diameter of pipe

a. Twice these tolerances permitted for spring hangers, constant force supports, and axial supports.

c.

Support directionality.

Acceptable deviation: ± 5 degrees.

Any noncompliances with these guidelines will be assessed on a case-by-case basis to determine the effect on the analysis results.

3.7.3.7.10.2 Support-Welded Attachments

Welded lugs are permissible for use on supports that do not act perpendicular to the pipe centerline and where slippage must be prevented. The design of acceptable welded attachments or lugs must be in accordance with the following geometric restrictions.

a. The attachment material, weld material, and pipe material have essentially the same moduli of elasticity and coefficients of thermal expansion.

$$\frac{L_1}{r} \le 0.5, \quad \frac{L_2}{r} \le 0.5, \quad \frac{L_1 \times L_2}{r^2} \le 0.075$$

b. (Equation 3.7-10)

where $2L_1$ is the width, $2L_2$ is the length of the welded attachment measured along the surface of the run pipe, and r is the mean pipe radius.

c. The attachment is made on straight pipe, with the nearest edge of the attachment weld located at a minimum distance of rt from any other weld or discontinuity. The mean pipe radius is r and t is the nominal pipe wall thickness.

d. $D_o/t \le 100$ where D_o and t are the outside diameter and nominal pipe wall thicknesses of the run pipe respectively.

e. The use of fillet welds for pipe attachments is normally acceptable. Full penetration welds will be specified in certain high temperature, high load situations.

Stanchions are small pipe segments welded to the run pipe and used for support. The support must be welded to the run pipe with a full penetration weld. The "branch" portion will have a zero pressure stress. The ratio of stanchion mean radius to pipe mean radius will govern the choice of applicable stress intensification factors used within the piping qualification. If this ratio is greater than 0.5, welding tee factors will be used; if it is less than or equal to 0.5, the larger of welding tee or branch factors will be used.

Supports requiring lugs or stanchions will be designed such that stress amplification is minimized. Exceptions to this criteria will be investigated on an individual basis.

An exception to the component standard supports stiffness capabilities is made in the case of U-bolt type supports, for the Seismic Upgrade Program effort. Finite element analysis evaluations provided the basis for U-bolt support stiffness values and load capabilities.

Rod hangers are generally single acting vertical supports; in the upward direction, they are susceptible to an early buckling condition. Stiffnesses, therefore, in the upward direction are minimal. Consideration of this condition will be made within the applicable analysis of piping systems with rod hangers included, such that the upward motion of a piping section at the location of these supports will cause support inaction. If stress acceptability is verified with support inactivity in the upward direction, the continued use of single acting rod hangers is satisfactory. If it is found that double-acting support is required for piping qualification, the replacement of rod hangers with struts will be recommended.

3.7.4 SEISMIC INSTRUMENTATION

A strong motion accelerograph is installed in the subbasement of the intermediate building at elevation 237 ft. This location was chosen rather than the basement of the containment since it more easily facilitates periodic surveillance of the instrument (this would be difficult should the instrument be located in the basement of the containment) and the retrieval of the shock record can more readily be made. The response of the accelerograph located in the basement of the intermediate building will be virtually the same as one located in the basement of the containment. The elevations of the basement floors of both the containment and intermediate building are within 2 ft of one another and both basement mats are supported upon the underlying Queenston formation.

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8. M. P. Singh, "Generation of Seismic Floor Spectra," Journal of Engineering Mechanics Division, ASCE, EM5, October, 1975.

9. M. P. Singh, "Seismic Design Input for Secondary Systems," ASCE Mini-Conference on Civil Engineering and Nuclear Power, Session 11, Boston, Massachusetts, Volume II, April 1979.

10. P. P. DeRosa, et al., Pressurizer Generic Stress Report, Sections 3.1, 3.2, 3.4, 3.7, Westinghouse Electric Corporation, Tampa Division, 1973.

11. T. M. Wiger, Framatome ANP, "Ginna Nuclear Power Plant, Control Rod Drive Mechanism Pressure Housing Assembly, Appurtenances ASME III, Class 1 Design Report" dated May 19, 2003.

Table 3.7-1
ORIGINAL AND CURRENT RECOMMENDED DAMPING VALUES

Structure or Component	Critical Damping (%) Ginna	Regulatory Guide 1.61 (Safe Shutdown Earthquake)	NUREG/CR-0098[a] (Yield Levels)
Prestressed concrete	2	5	5 to 7
Reinforced concrete	5	7	7 to 10
Steel frame	1 or 2.5	4 or 7	10 to 15
Welded assemblies	1	4	5 to 7
Bolted and riveted assemblies	2.5	7	10 to 15
Vital piping	0.5	2 or 3	2 to 3

a. See *Reference 6*.

Table 3.7-2
MODAL FREQUENCIES OF THE INTERCONNECTED BUILDING MODEL

	Frequency (Hz)	
Mode Number	Half-Area Model	Full-Area Model
1	1.8 (3.4, 12.9)	2.3 (7.4, 12.6)
2	2.0 (10.2, 0.2)	2.4 (8.5, 4.7)
3	2.1	2.8
4	2.4	3.1
5	2.6	3.2 (7.4, 0.6)
6	2.8	3.4
7	2.9	3.4
8	3.3	3.6
9	3.4	3.9
10	3.6	4.0 (6.3, 1.4)
11	4.0	4.3
12	4.2	4.3
13	4.2	4.6
14	4.4	4.6
15	4.7	5.4
16	5.6	6.7
17	6.1	6.9 (12.7, 6.4)
18	6.5 (6.4, 4.5)	7.0
19	6.6	7.3
20	6.7 (8.4, 8.5)	7.4
21	6.9 (10.3, 7.2)	7.5
22	7.0	8.0
23	7.8	9.7 (5.1, 8.3)
24	9.3	10.4
25	9.5 (5.4, 8.4)	10.6
26	10.4	10.9
27	10.8	11.1
28	11.1	11.7

	Frequency (Hz)	
Mode Number	Half-Area Model	Full-Area Model
29	11.2	12.1
30	12.2	12.8
31	13.5	14.0
32	13.8	16.4
33	16.4	16.7
34	17.8 (2.4, 6.6)	17.8 (2.3, 6.5)
35	18.5	18.6
36	19.3	19.5
37	21.1 (0.1, 27.1)	21.2 (0.1, 27.1)
38	22.9 (26.9, 0.1)	22.9 (26.9, 0.1)
39	27.0	27.2
40	33.5	33.6
41	41.2	41.2
42	45.1	45.7
43	57.8	57.8
44	60.4 (6.7, 0.0)	60.4 (6.7, 0.0)

NOTE:—Numbers in parentheses are the 10 largest modal participation factors in the east-west and north-south directions, respectively.

Table 3.7-3
EQUIPMENT AND LOCATIONS WHERE IN-STRUCTURE SPECTRA WERE GENERATED FOR THE SYSTEMATIC EVALUATION PROGRAM

Building	Equipment	Elevation (ft)
Containment interior structures	Pressurizer PR-1	253
	Control rod drive	253 and 278
	Steam generator SG-1A	250 and 278
	Steam generator SG-1B	250 and 278
	Coolant pump RP-1A	247
	Coolant pump RP-1B	247
Auxiliary building	Platform center of gravity	281.5
	Heat exchanger (35)	281.5
	Surge tank (34)	281.5
	Boric acid storage tank (40 B)	271
	Operating floor center of gravity	271
Control building	Basement floor center of gravity	250
	Relay room floor center of gravity	269.75
	Control room floor center of gravity	289.75

3.8 DESIGN OF SEISMIC CATEGORY I STRUCTURES

3.8.1 CONTAINMENT

3.8.1.1 General Description

3.8.1.1.1 Containment Structure

The reactor containment structure is a reinforced-concrete vertical right cylinder with a flat base and a hemispherical dome. A welded steel liner is attached to the inside face of the concrete shell to ensure a high degree of leaktightness. On the inside of the liner every weld seam has a leak test channel welded over it. The channels can be pressurized to design pressure for liner leak testing whenever the containment vessel is open. Exceptions were taken during the 1996 Steam Generator Replacement where two construction openings were created in the dome. The perimeter closure welds for both liner plate opening repairs have leak test channels on the outside of the liner plate. The thickness of the liner in the cylinder and dome is 3/8 in. and in the base is 1/4 in. The containment structure is 99 ft high to the spring line of the dome and has an inside diameter of 105 ft. The containment provides a minimum free volume of approximately 997,000 ft^3. An elevation and details of the containment structure are shown in Figures 3.8-1 through 3.8-5.

The cylindrical reinforced concrete walls are 3 ft 6 in. thick, and the concrete hemispherical dome is 2 ft 6 in. thick. These shell thicknesses are established to satisfy the requirements of the structural criteria as well as the shielding requirements. These thicknesses are nominal values. The true relevant engineering values are dependent on the specific location in the structure and the loading condition that is present. The concrete base slab is 2 ft thick with an additional thickness of concrete fill of 2 ft over the bottom liner plate. The containment cylinder is founded on rock (sandstone) by means of post-tensioned rock anchors which ensure that the rock then acts as an integral part of the containment structure. The hemispherical dome is reinforced concrete designed for all moments, axial loads, and shears resulting from the loading conditions described in this section. The cylinder wall is prestressed vertically and reinforced circumferentially with mild steel deformed bars. The base is a reinforced-concrete slab. The rock anchors are used for all vertical axial loads in the cylinder walls and thereby avoid the transfer of an imposed shear to the base slab. The structural systems for the containment structure are summarized as follows:

- Hemispherical dome - mild steel-reinforced concrete.
- Cylindrical walls.
    1. Vertical direction - prestressed concrete.
    2. Circumferential direction - mild steel reinforced concrete.
- Rock anchors - prestressed.

The design ensures that the structure has an elastic response to all loads and that it strains within such limits that the integrity of the liner is not prejudiced. The liner participates with the shell as it reacts to these loads and is designed to ensure the vessel vapor tightness.

The design of the structural elements are more fully described in Sections 3.8.1.2 and 3.8.1.3.

3.8.1.1.2 Waterproofing

No drainage system was provided under the containment structure. The maximum ground water elevation considered during the design of Ginna Station in the vicinity of the containment structure was 252 ft. The design-basis water level has since been revised to 265 ft msl (see Section 2.4.10.1). This compares with an elevation at the underside of the base slab of 231 ft 8 in. It is unlikely that tensile stresses will produce cracks in the outside concrete face because significant constraint is afforded by the irregular surface of the founding rock material. This rock has significant structural characteristics as described in Section 2.5. However, the concrete is not totally impermeable. For this reason, the design of the liner, test channels, backup bars (structural tees), anchors on test channels (refer to Figure 3.8-6) and the concrete cover were based upon accommodating the full hydrostatic head of water. A significant corrosion potential for embedded steel does not exist due to the close contact between the alkaline concrete and steel which provides a highly corrosive-resistant environment for the liner.

The basement floor elevation of the containment vessel is 235 ft 8 in. The exterior of the cylinder walls are covered from the edge of the ring girder to elevation 253 ft 0 in. with a membrane waterproofing. No waterproofing was placed between the foundation material (rock) and the base slab. The liner and liner anchorage at the base of the vessel were designed to withstand a theoretical pore pressure equal to the hydrostatic head of water, 7.7 psi. The site is not subject to significant fluctuations in the ground water elevations. Consequently, if the base liner is subject to the assumed water pressure, this pressure should remain essentially constant.

The net buoyant force due to the hydrostatic pressure acting on the containment base is transmitted by the base slab to the cylinder walls.

3.8.1.1.3 Rock Anchors

The side walls of the containment are anchored to the foundation rock with prestressed rock anchors. The anchors place a preload between the foundation rock and a ring beam at the base of the side wall. The tendons in the side walls are coupled to the rock anchors and extend to a location 12 ft 6 in. above the spring line to provide accessibility to the upper anchorage and to permit tensioning following the completion of the dome concrete work. A removable cover is placed over the top anchorage head for protection and to provide an expansion reservoir for the tendon protection system. Refer to Figure 3.8-7 for details of this enclosure.

The outer surface of the containment can be inspected, except in those limited areas where roofs, floors, and walls of adjacent buildings preclude access.

3.8.1.1.4 Construction Sequence

The sequence for the construction of the shell of the containment structure was as follows:

A. Excavation was completed and the exposed rock examined by a soils engineer to ensure its competence.

B. The concrete for the portion of the ring girder at the base of the cylindrical wall was placed. Sleeves and bearing places for the rock anchors were embedded in this concrete pour.

C. The holes for rock anchors were drilled through the embedded sleeves and into the rock. The anchor, which was completely fabricated in the shop, was inserted and the first stage grout placed. Following the required curing period the anchor was tensioned and the second stage grout inserted under pressure.

D. The concrete for the base mat was placed with embedded bars for the backup of liner welds. The outer concrete pour contains the tension bars (dowel at base of cylindrical walls). The base slabs for the sumps and pit also were installed with embedded bars for backup of liner welds. The liner for the walls of the sumps was then erected and used as an inner form for the placement of concrete.

E. The liner was erected starting on the base and continuing to the knuckle, the cylindrical wall, and the dome. All electrical and mechanical penetrations (i.e., sleeves for penetrations) were installed as liner erection progressed. Essentially all electrical and mechanical penetrations were shop assembled in the cylindrical wall plates. Provision was made to install the equipment access hatch and personnel air locks at a later stage of construction. Temporary openings were provided in the liner cylindrical wall for construction access requirements.

   The closure procedure for temporary openings in the liner was similar to that for steel tank construction. Initially, special reinforcement was provided around the periphery of the temporary openings. A sufficient width of plate extended beyond the limits of the concrete placement to preclude detrimental heatup of the concrete due to the welding of the closure plate. The welding procedures were identical to that used for all liner weld seams.

   The preparation of construction joints and placement of concrete in temporary openings is as described in Appendix 3B Attachment 1.

F. The tendon conduit was embedded in the second ring girder pour with provision made for installing the tendon and completing the coupling of rock anchor to the sidewall tendon. The enclosure about the coupling was welded to the anchor plate and a window removed to permit making-up the coupling. An expansion bellows was provided where differential motion will occur at the level of the elastomer pads.

G. The elastomer pads were installed and tendon conduit plus mild steel reinforcing placed. The mild steel reinforcing was temporarily supported from the tendon conduit and the stiffeners on the liner. Concrete placement at temporary openings was delayed and provision made to stagger reinforcement splices at these locations as well as elsewhere on the structure. For the cylindrical wall and dome, the liner was used as an inner form.

H. Where grade is adjacent to the structure, a retaining wall was erected to ensure no earth was placed against the cylindrical wall.

I. The concrete cylindrical wall was completed and temporary openings closed after they no longer were required for construction. The reinforcement rings about the equipment access hatch and personnel air locks were installed. The reinforcement about the equipment access hatch was not placed until after major components were installed.

The cylinder walls were placed with horizontal joints spaced at approximately 11-ft centers. Vertical joints were spaced at approximately 42.5-ft centers (i.e., the cylinder was divided into approximately eight equal pours). The final six lifts were poured with the spacing of vertical joints increased to approximately 57 ft (i.e., six approximately equal pours). Form ties consisting of 0.5-in. diameter threaded studs spaced at approximately 2-ft centers were welded to the liner (both plate and channel anchors) for attaching the liner to the outer form. The outer form was supported by cantilever construction from the lower pour. No attempt was made to stagger vertical or horizontal joints. A minimum delay of 3 days was maintained before placing new concrete against abutting pours. Initial and final concrete curing were by the wet method as specified in ACI 301-66.

The dome concrete (i.e., all concrete above the ledge at elevation 343 ft 2 in.) was placed as continuous rings with a chord width of approximately 4.2 ft. The final pour (center "dollar" section) consisted of an approximately 8-ft diameter section. All concrete was placed in one pour for the full thickness of the concrete shell. A galvanized expanded metal mesh located 1 in. inboard of the exposed face was used as an outer form on the greater sloped portion of the dome (i.e., up to an angle of approximately 55 degrees from the spring line). Form ties in the form of 0.5-in. diameter studs were welded to the liner plate and attached to the cage of reinforcing bars. A minimum delay of 3 days was maintained before placing new concrete against the previous concrete ring.

During the 1996 Steam Generator Replacement outage two construction openings were created in the containment dome. The removed liner plate sections were reused. Stiffener angles welded to the liner plate sections for rigging removal replaced the original 0.5 in. diameter studs. The reinforcing bars were supported off the stiffener angles and support chairs. The dome openings were then boarded and poured monolithically. Board form box-outs were used to place and consolidate concrete.

J. The tendons were installed in the embedded conduits and the sidewall tendon and the rock anchor were coupled. The remaining concrete pour in the ring girders was completed and the wax inserted into the conduit.

K. The concrete dome was completed and the sidewall tendons tensioned.

L. Following the tensioning of the tendons, the equipment access hatch with inset personnel air lock plus the second personnel air lock were installed. The containment was then ready for structural and leakage testing.

3.8.1.1.5 Steel Reinforcement

The principal dome reinforcement is continuous except for the anchorage at elevation 366 ft 8 in. which is provided in the form of a mechanical connection to a continuous circumferential plate. Additional steel to control spalling on the outer face of the shell is provided in the form of welded wire fabric. At the dome to cylinder discontinuity additional reinforcement is provided on both faces with 180-degree hooks with total anchorage provided to satisfy the requirements of ACI 318-63. Details are shown on Figure 3.8-5.

In the anchorage zone of the prestressing steel, the major steel provided to withstand bursting forces consists of continuous spirals. Radial reinforcement is provided with 180-degree hooks around the vertical flexural steel for anchorage. Vertical (meridional) reinforcement

used for flexural and temperature resistance is lap spliced in accordance with ACI 318-63 requirements on the basis of splice requirements at points of maximum tensile stress. Details are also shown on Figure 3.8-5.

All principal circumferential reinforcement is continuous except at the small penetrations where mechanical anchors are provided, as shown typically on Figure 3.8-4, and for a limited number of bars at the large openings, as described in Appendix 3B. Vertical (meridional) reinforcement is lap spliced (except for special large size bars which are Cadweld spliced) in accordance with ACI 318-63 requirements on the basis of splices at points of maximum tensile stress. At the base of the wall all vertical (meridional) reinforcement is provided with 90-degree hooks for anchorage. Details are shown on Figure 3.8-4.

3.8.1.2 Mechanical Design Bases

3.8.1.2.1 General

The containment safety design basis and principal design criteria are contained in Section 6.2.1.1. The containment vessel is a steel-lined concrete shell designed to ensure that it responds elastically to all loads and strains within such limits that the integrity of the liner is not prejudiced. The liner is anchored so as to ensure composite action with the concrete shell.

The containment structure is designed based upon limiting load factors which are used as the ratio by which accident and earthquake loads are multiplied for design purposes to ensure that the load/deformation behavior of the structure is one of elastic, low-strain behavior. This approach places minimum emphasis on fixed gravity loads and maximum emphasis on accident and earthquake loads. Because of the refinement of the analysis and the restrictions on construction procedures, the load factors primarily provide for a safety margin on the load assumptions. Load combinations and load factors utilized in the design which provide an estimate of the margin with respect to all loads are tabulated in this section.

3.8.1.2.2 Design Loads

The following loads were considered in the structural design:

- Internal pressure.
- Test pressure 69 psig.
- Live loads Roof loads plus pipe reactions.
- External pressure 2.5 psig.
- Wind load.
- Internal temperature.
    1. Accident.
    2. Operating - 120°F. (Reference 54 increased the maximum operating temperature to 125°F)
- Seismic ground accelerations.
- Dead loads.
- Prestressing loads.

The thermal loads on the containment vessel and their variation with time are developed on the basis of the transients discussed in Section 6.2.1.2. The seismic loads were evaluated as outlined in Section 3.8.1.3. The wind and snow loads used for the design of structures were those specified in State Building Code for the State of New York. The wind loads given in this code are as follows:

Height Above Ground (ft)	Pressure Load (psf)
0-15	12
16-25	15
26-40	18
41-60	21
61-100	24
101-200	28

The snow load specified in the code for the plant location is 40 psf for a flat roof. This value also was used in the design of the containment.

3.8.1.2.3 Design Stress Criteria

3.8.1.2.3.1 Limiting Loads

The design was based upon limiting load factors which were used as the ratio by which accident, earthquake, and wind loads were multiplied for design purposes to ensure that the load deformation behavior of the structure is one of elastic, low-strain response. The loads utilized to determine the required limiting capacity of any structural element on the containment were computed as follows:

- $C = 0.95\ D + 1.5\ P + 1.0\ T$
- $C = 0.95\ D + 1.25\ P + 1.0\ T' + 1.25\ E$
- $C = 0.95\ D + 1.0\ P + 1.0\ \underline{T} + 1.0\ E'$

Symbols used in the above equations were defined as follows:

C = Required load capacity of section.

D = Dead load of structure.

P = Accident pressure load - 60 psig.

T = Thermal loads based upon temperature transient associated with 1.5 times accident pressure.

T′ = Thermal loads based upon temperature transient associated with 1.25 accident pressure.

T = Thermal loads based upon temperature transient associated with accident pressure.

E = Seismic load based on 0.08g ground acceleration.

E′ = Seismic load based on 0.20g ground acceleration.

If the required resisting capacity on any structural component resulting from the wind load on any portion of the structure exceeded that resulting from the design earthquake, the wind load, W, was used in lieu of E in the second equation. The factor of 1.05 times dead load was used when it controlled in determining the required load capacity. All structural components were designed to have a capacity required by the most severe loading combination.

3.8.1.2.3.2 Load Factors

The load factors used in these equations make provision for safety of the containment structure in the same manner as does the ultimate strength design procedure in ACI 318. Because of the refinement of the analysis and the restrictions on construction procedures, the load factors in the design primarily provide for a safety margin on the load assumptions. The load factors utilized in the criteria were based upon the load factor concept employed in Part IV-B, Structural Analysis and Proportioning of Members - Ultimate Strength Design, of ACI 318-63. The load factor of 0.95 applied to the dead load represents the accuracy of dead load calculations (i.e., ±5%) considering the greater severity of reduced dead loads for tension members. The load factor applied to accident pressure loads was consistent with that suggested by Waters and Barrett (*References 1* and *2*) as the limit of low-strain behavior on prestressed concrete pressure vessels for nuclear reactors. This factor was also consistent with the proposed set of "French Regulations Concerning Concrete Reactor Pressure Vessels" wherein it was stated that: The design pressure shall not exceed 2/5 of the pressure calculated to bring about destruction of the structure by rupture of the cables. The load factor considering a tendon stress of 0.60 f_u at factored load would therefore equal 0.6 divided by 2/5 or 1.5. The load factor of 1.25 applied to the design earthquake load is consistent with that utilized in ACI 318 Part IV-B, Chapter 15.

The containment design includes the consideration of both primary and secondary stresses. When a structure experiences only elastic strains there is only a minimal relief of restraints causing secondary stresses. If a structure experiences increased strains beyond the elastic range, the restraints at any point will cease to be as significant due to local yielding in these regions and, if increased loads were applied until collapse of the structure was imminent, all restraints would be effectively removed and only membrane forces (primary stresses) should be experienced, unless premature shear failure were to occur. The design limit for the containment structure was conservatively established to ensure elastic, low-strain behavior at design loads thereby requiring design consideration of all secondary stress effects.

3.8.1.2.3.3 Maximum Thermal Load

The maximum expected values of T (thermal load) at any section are based upon the following conditions:

a. The maximum operating temperature inside the containment is 125°F and the minimum ambient temperature outside the containment is -10°F. The analysis used the original 120°F in containment. Reference 54 increased the allowable temperature to 125°F. The peak temperature will only be reached during hot outside temperatures such that this gradient is very conservative. See Section 3.8.2.2.2 of UFSAR for SEP

re-evaluation of containment.

b. The maximum temperature of the inner surface of the liner (inner face of insulation where the liner is insulated) will be that temperature associated with the factored load, 1.5 times accident pressure. This temperature is approximately 312°F. The design of the shell where the liner is insulated is based upon a maximum temperature rise of 10°F in the liner coincident with maximum pressure.

c. The maximum operating temperature at the basement floor elevation is 125°F and 5 ft below the floor elevation it is 50°F. The upper 2 ft of the basement slab were designed for a transient thermal gradient equal to 30°F. Thermal expansion of the basement slab approximately balances drying shrinkage.

   The steady-state operating thermal transient considered in the design for winter conditions (external ambient temperature equals -10°F) is shown on Figure 3.8-8. The steady operating thermal transient for summer conditions was not developed in detail in that it was concluded that such a condition would not affect the reinforcement requirements because a lesser gradient would exist.

   The transient thermal gradients through the containment shell for the insulated liner due to the design-basis accident (factored loads) was assumed for purposes of analysis to be the superposition of a liner increase of 10°F onto the operating thermal gradient described above. This is conservative as compared to the expected results described in Appendix 3E. The maximum concrete fiber temperature where the liner is uninsulated (dome) is 220°F in the region immediately in contact with the liner. The calculated shell elongation due to the pressure load exceeds the concrete fiber elongations due to the thermal load indicating that no restraint of concrete thermal growth occurs.

3.8.1.2.4 Load Capacity

3.8.1.2.4.1 Reinforced Concrete

The value of Young's modulus (E_c) for uncracked concrete was assumed to be 4.1 x 10^6 psi calculated on the basis of the equation in Table 1002(a) of ACI 318-63. The E_c and Poisson's ratio (ν_c) for cracked concrete were assumed to be zero. This latter assumption is considered to be substantiated by test data (*References 3 through 5*) for reinforcement experiencing stresses in excess of the 20,000 to 30,000 psi. (Refer to Appendix 3B regarding similar assumptions regarding the analysis of large openings.)

This structure is prestressed vertically only and the liner is insulated in the prestressed portion. The liner stresses (meridional direction) were calculated to be 4500 psi compression based upon a prestress force of 0.70 fs. The concrete strain due to creep and shrinkage was established as being 320 x 10^{-6} in./in. This increases the liner stress to 14,100 psi compression at the end of plant life.

Concrete reinforcement is intermediate grade billet steel conforming to ASTM A15-64 and A408-62T with a guaranteed minimum yield strength of 40,000 psi. Replacement reinforcement used for the Steam Generator Replacement dome opening repairs is ASTM A615 Grade 0. This reinforcement exceeds the minimum yield strength requirements of the original reinforcement. The design limit for tension members (i.e., the capacity required for

the factored loads) is based upon the yield stress of the reinforcing steel. No mild steel

reinforcement will experience average strains beyond the yield point at the factored load. The load capacity so determined is reduced by a capacity reduction factor, ϕ, which provides for the possibility that small adverse variations in material strength, workmanship, dimensions, and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in any under capacity. The coefficient ϕ is 0.95 for tension, 0.90 for flexure, and 0.85 for diagonal tension, bond, and anchorage. The coefficient ϕ of 0.95 for tension members compares with a coefficient of 0.90 utilized in ACI 318 for ultimate strength design of flexured members. However, in a tension member, unlike the case of a flexural member, only the variation of steel strength and not concrete strength is of concern. Also, the effect of reinforcement misplacement is not as critical as it is for a flexural member. Therefore, the capacity reduction factor of 0.95 is considered to be conservative.

The two equations developed previously for the loss-of-coolant accident and the loss-of-coolant accident combined with the design (operating-basis) earthquake could be written as follows for the mild steel reinforced sections:

C = 0.95 Y.P. = 0.95 D + 1.0 T + 1.5 P

C = 0.95 Y.P. = 0.95 D + 1.25 P + 1.0 T' + 1.25 E

To compare these equations with a working strength design the following equations are developed:

$$f = \frac{(D + P + T)0.95}{(0.95 \cdot D + 1.5 \cdot P + 1.0 \cdot T)} = 63\%$$

(Equation 3.8-1)

$$f = \frac{(D + P + T + E)0.95}{(0.95 \cdot D + 1.25 \cdot P + 1.0 \cdot T' + 1.25 \cdot E)} = 74\%$$

(Equation 3.8-2)

The new symbol in the above equation is defined as follows:

f = Ratio of the working stress to yield stress.

3.8.1.2.4.2 Prestressed Concrete

The design for the containment provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses, there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria, the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for ACI 318- 63 Chapter 17, Shear and Diagonal Tension -Ultimate Strength Design, and there is a basis for designing shear reinforcement.

The steel tendons for prestressing consist of high tensile, bright, cold drawn and stress-relieved steel wires conforming to ASTM A 421-59 T, Type BA, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete, with a minimum tensile stress of 240,000 psi.

The prestressed concrete is assumed to develop no tensile capacity in a direction normal to a horizontal plane. The design load capacity of tension elements is based upon a resultant condition of zero concrete stress due to the maximum combination of primary and secondary membrane forces. Any nominal secondary tensile stresses due to bending will be assumed to cause partial cracking. Mild steel reinforcing will be provided to control this cracking by limiting crack width, spacing, and depth. The load capacity so determined will be reduced by a capacity reduction factor, ϕ, which will be conservatively established as 0.95 which compares with a coefficient of 0.90 utilized in ACI 318 for ultimate design of flexural members. In a prestressed tension member only variations in the field-applied tensioning loads are of any concern. Tendon location and concrete strength variations are not critical as they are for flexural members.

Generally, if no tension stresses can be developed in the concrete, prestressed concrete tension members have a relatively low reserve strength above the point of zero stress. If cracking is initiated as the very low tensile stresses are developed in the concrete, all additional loads will be carried by the steel alone. Since the prestressing steel has a relatively small area of cross-section, the strain at any section increases markedly after cracking begins. For this reason, the containment design was conservatively based upon no complete cracking of any prestressed wall section.

Tensile stresses in the concrete resulting from diagonal tension will be permitted. The nominal shear stresses as a measure of this diagonal tension will be less than the maximum value stipulated in Chapter 17 of ACI 318.

The steel tendons are stressed during the post-tensioning operation to a maximum of 80% of ultimate strength and locked-off for an initial stress of 70% of the ultimate strength. The maximum effective prestress is determined, taking into consideration allowances for the following losses which are deduced from the transfer prestress:

- Elastic shortening of concrete.
- Creeping of concrete.
- Shrinkage of concrete.
- Relaxation of steel stress.

- Frictional loss due to intended or unintended curvature of the tendons.

In no event does the effective prestress exceed 60% of the ultimate strength of the prestressing steel or 80% of the nominal yield point stress of the prestressing steel, whichever is smaller. The design of all prestressed concrete elements for shear, bond, and other design considerations is in accordance with ACI 318-63 Chapter 26, Prestressed Concrete.

The prestressing force applied in the field was determined by measuring tendon elongation and also by checking jack pressure on a calibrated gauge or by the use of an accurately calibrated dynamometer. The cause of any discrepancy which exceeded 5% was ascertained and corrected. Elongation requirements were taken from load-elongation curves for the steel used.

With the exception of the large openings (refer to Appendix 3B) reinforcing bars are not draped around openings. Consequently, the minimum radius is the radius of the cylinder. The reinforcement about small openings is shown typically on Figure 3.8-4. The horizontal reinforcement is concentrated near the hole to accommodate stress concentrations. The tendons are draped only if required for clearances. The magnitude of prestress under construction and operating conditions is well within accepted limits based on ACI 318 requirements. The initial average membrane stress is 640 psi. Even a stress concentration factor of 3 results in acceptable stresses. The requirements for anchoring reinforcing bars are discussed in Section 3.8.1.4.5.4, Anchorage Stresses.

3.8.1.2.4.3 Liner

The liner is carbon steel plate conforming to ASTM A442-60T Grade 60 with a minimum yield of 32,000 psi. The liner plate thickness is 1/4 in. for the base and 3/8 in. for the cylinder and dome. Original liner welds in general were made from both sides of the plate and therefore backup strips were not used. In the base where the liner was welded to structural tees, the tees were continuous at all plate intersections.

During the 1996 Steam Generator Replacement outage, construction openings were created in the dome. Liner plate sections were removed during the replacement, prepped on the ground, then lifted and welded back in place. As required, ASTM A516 Grade 60 plate with a minimum yield of 32,000 psi was used in the liner repair. All seam welds for removed liner sections were made from the exterior only with the use of backing bars. The backing bars were left in place.

The load capacity is based upon the yield stress of the liner as reduced by the capacity reduction factor, ϕ, previously described. Sufficient anchorage is provided to ensure elastic stability of the liner. Anchorages are in the form of stagger welded channels on the cylinder and studs on the dome. Liner plate stiffener angles were used in lieu of studs at the locations where dome openings were repaired following the Steam Generator Replacement in 1996.

Insulation is provided for the side walls to a point 15 ft 0 in. above the spring line so as to limit the maximum liner temperature due to the loss-of-coolant accident and thereby avoid excessive compressive stresses in the steel plate.

All weld seams in the liner plate are covered with a test channel to permit testing of leaktightness. Except for the equipment access hatch, all penetrations provide a double barrier against leakage and can be pressurized to permit testing of leaktightness. The equipment access

hatch contains weld seams with no test channels. The liner plate on the base of the containment is welded to backup bars. These bars are continuous, as shown in Figure 3.8-6.

3.8.1.2.4.4 Rock

The containment is founded on rock (sandstone) for which the soils consultant recommended an allowable bearing pressure of 35 tons per square foot. The maximum bearing pressure occurs under the ring girder where the maximum bearing pressure was limited to 30 tons per square foot. This bearing pressure occurs under operating conditions and is reduced under incident conditions. The soils consultant also recommended a limit on the lateral resistance of the rock of 25,000 psf. The maximum lateral pressure, occurring at the ring girder under the combination of operating and incident loads is 24,000 psf. A detailed description of subsurface conditions is found in Section 2.5.

3.8.1.2.5 Codes and Standards

The design, materials, fabrication, inspection, and proof testing of the containment complied with the applicable parts of the following:

1. ASME Boiler and Pressure Vessel Code, Section III - Nuclear Vessels, Section VIII - Unfired Pressure Vessels, Section IX - Welding Qualifications.
2. Building Code Requirements for Reinforced Concrete (ACI 318-63).
3. American Institute of Steel Construction Specifications:
    a. Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted April 17, 1963.
    b. Code of Standard Practice for Steel Buildings and Bridges, revised February 20, 1963.
4. USAS N 6.2 - 1965, Safety Standard for Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors.
5. ACI 306-66, Specifications for Structural Concrete for Buildings.
6. ASTM C 150-64, Specifications for Portland Cement.
7. State of New York Department of Public Works Specification.
8. ASTM C 260-63T, Specifications for Air-Entrained Admixtures for Concrete.
9. ASTM A 15-64T, Specifications for Billet-Steel Bars for Concrete Reinforcement.
10. ASTM A 305-56T, Specifications for Minimum Requirements for Deformation of Deformed Bars for Concrete Reinforcement.
11. ASTM A408-64T, Specifications for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement.
12. ASTM C 94-65, Recommended Practice for Winter Concreting.
13. ACI 306-66, Recommended Practice for Winter Concreting.
14. ACI 605-59, Recommended Practice for Hot Weather Concreting.

15. ASTM A 421-65, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete.
16. ASTM C29-60, Method of Test for Unit Weight of Aggregate.
17. ASTM C 40-66, Method of Test for Organic Impurities in Sands for Concrete.
18. ASTM C 127-59, Method of Test for Specific Gravity and Absorption of Coarse Aggregate.
19. ASTM C 128-59, Method of Test for Specific Gravity and Absorption of Fine Aggregate.
20. ASTM C 136-63, Method of Test for Sieve or Screen Analysis of Fine and Coarse Aggregate.
21. ASTM C 39-64, Method of Test for Compressive Strength of Molded Concrete Cylinders.
22. ASTM C 192-66, Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory.
23. ASTM A 15-62T, Specifications for Billet-Steel Bars for Concrete Reinforcement.
24. ASTM A408-64, Specifications for Special Large Sized Deformed Billet-Steel Bars for Concrete Reinforcement.
25. ASTM A 432-64, Specification for Deformed Billet-Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength.
26. ASTM C 31-65, Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field.
27. ASTM C33-64, Specifications for Concrete Aggregates.
28. ASTM C42-64, Methods of Securing, Preparing, and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths.
29. ASTM C 131-64T, Method of Test for Abrasion of Coarse Aggregate by Use of the Los Angeles Machine.
30. ASTM C 138-63, Method of Test for Weight per Cubic Food, Yield, and Air Content (Gravimetric) of Concrete.
31. ASTM C 143-58, Method of Test for Slump of Portland Cement Concrete.
32. ASTM C 150-65, Specifications for Portland Cement.
33. ASTM C 172-54, Method of Sampling Fresh Concrete.
34. ASTM C 231-62, Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method.
35. ASTM C 260-65T, Specifications for Air-Entrained Admixtures.
36. ASTM C 494-62T, Specifications for Chemical Admixtures for Concrete.
37. ASTM C 173-58, Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method.
38. ACI 214-57, Recommended Practice for Evaluation of Compression Test Results of Field Concrete.
39. ACI 315-65, Manual of Standard Practice for Detailing Reinforced Concrete Structures.

40. ACI 347-63, Recommended Practice for Concrete Formwork.
41. ASTM D 287-64, Method of Test for API Gravity of Crude Petroleum and Petroleum Products (Hydrometer Method).
42. ASTM D 97-66, Method of Test for Pour Points.
43. ASTM D 92-66, Method of Test for Flash Point by Cleveland Open Cup.
44. ASTM D 88-56, Method of Test for Saybolt Viscosity.
45. ASTM D 937-58, Method of Test for Cone Penetrations of Petroleum.
46. ASTM D 512-62T, Methods of Test for Chloride Ion in Industrial Water and Industrial Waste Water.
47. ASTM D 1255-65T, Method of Test for Sulfides in Industrial Water and Industrial Waste Water.
48. ASTM D 992-52, Method of Test for Nitrate Ion in Industrial Water.
49. ASTM A 442-60T, Tentative Specifications for Carbon Steel Plates with Improved Transition Properties.
50. ASTM A 300-63T, Specifications for Steel Plates for Pressure Vessels for Service at Low Temperature.
51. ASTM A 36-63T, Specifications for Structural Steel.
52. SSPC-SP6-63, Commercial Blast Cleaning.
53. SSPC-SP8-63, Pickling.
54. SSPC-PA1-64, Shop, Field, and Maintenance Painting.
55. ASTM A 322-64A, Specification for Hot-Rolled Alloy Steel Bars.
56. ASTM A 29-64, Specification for General Requirements for Hot-Rolled and Cold-Finished Carbon and Alloy Steel Bars.
57. ASTM D 624-54, Methods of Test for Tear Resistance of Vulcanized Rubber.
58. ASTM D 676-59T, Method of Test for Indentation of Rubber by Means of a Durometer.
59. ASTM B 412-66T, Method of Tension Testing of Vulcanized Rubber.
60. ASTM D 573-53, Method of Test for Accelerated Aging of Vulcanized Rubber by the Oven Method.
61. ASTM D 395-61, Method of Test for Compression Set of Vulcanized Rubber.
62. ASTM D 746-64T, Method of Test for Brittleness Temperature of Plastics and Elastomers by Impact.
63. ASTM D 1149-64, Method of Test for Accelerated Ozone Cracking of Vulcanized Rubber.
64. ASTM D 471-66, Method of Test for Change in Properties of Elastometric Vulcanizates Resulting from Immersion in Liquids.
65. ASTM A 514-65, Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding.

66. ASTM A 441-66T, Specification for High-Strength Low Alloy Structural Manganese Vanadium Steel.
67. ASTM A 53-65, Specification for Welded and Seamless Steel Pipe.
68. ASTM A 435-65, Method and Specification for Ultrasonic Testing and Inspection of Steel Plates of Firebox and Higher Quality.
69. ASTM C 177-63, Method of Test for Thermal Conductivity of Materials by Means of the Guarded Hot Plate.
70. ASTM C 165-54, Method of Test for Compressive Strength of Performed Block-Type Thermal Insulation.
71. ASTM C 355-64, Methods of Test for Water Vapor Transmission of Thick Materials.
72. ASTM C 273-61, Method of Shear Test in Flatwise Plane of Flat Sandwich Constructions or Sandwich Cores.
73. ASTM D 1622-63, Method of Test of Apparent Density of Rigid Cellular Plastics.

The structural design also met the requirements established by the State Building Construction Code, State of New York, 1961.

3.8.1.2.6 Code and Standards Steam Generator Replacement (Dome Opening Repairs)

The design, materials, fabrication, inspection, and testing of the Steam Generator Replacement dome opening repairs complied with the applicable parts of the following. (The latest revision of the code in effect at the time of construction is applicable.)

1. ACI 211.1, Standard Practice for Selecting Proportions of Normal, Heavyweight, and Mass Concrete.
2. ACI 301, Specification for Structural Concrete for Buildings.
3. ACI 304, Guide for Measuring, Mixing, Transporting and Placing Concrete.
4. ACI 305R, Hot Weather Concreting.
5. ACI 306R, Code Weather Concreting.
6. ACI 318, Building Code Requirements for Reinforced Concrete.
7. ASTM C 33, Standard Specification for Concrete Aggregates.
8. ASTM C 39, Compressive Strength of Cylindrical Concrete Specimens.
9. ASTM C 40, Standard Test Methods for Organic Impurities in Fine Aggregate for Concrete.
10. ASTM C 88, Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate.
11. ASTM C 94, Standard Specification for Ready-Mixed Concrete.
12. ASTM C 109, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens).

13. ASTM C 117, Standard Test Method for Materials Finer than 75mm No. 200 Sieve in Mineral Aggregates by Washing.
14. ASTM C 123, Standard Test Method for Lightweight Pieces in Aggregates.
15. ASTM C 127, Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate.
16. ASTM C 128, Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate.
17. ASTM C 131, Standard Test Method for Resistance to Degradation of Small-Size Aggregate by Abrasion and Impact in the Los Angeles Machine.
18. ASTM C 136, Standard Method for Sieve Analysis of Fine and Coarse Aggregates.
19. ASTM C 138, Standard Method of Test for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete.
20. ASTM C 142, Standard Test Method for Clay Lumps and Friable Particles in Aggregates.
21. ASTM C 143, Standard Test Method for Slump of Hydraulic Cement Concrete.
22. ASTM C 150, Standard Specification for Portland Cement.
23. ASTM C 151, Test Method for Autoclave Expansion of Portland Cement.
24. ASTM C 172, Standard Practice for Sampling Freshly Mixed Concrete.
25. ASTM C 173, Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method.
26. ASTM C 191, Test Method for Time of Setting of Hydraulic Cement by Vicat Needle.
27. ASTM C 192, Standard Method of Making and Curing Concrete Test Specimens in the Laboratory.
28. ASTM C 231, Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.
29. ASTM C 260, Standard Specification for Air-Entraining Admixtures for Concrete.
30. ASTM C 289, Standard Test Method for Potential Reactivity of Aggregates (Chemical Method).
31. ASTM C 295, Recommended Practice for Petrographic Examination of Aggregates for Concrete.
32. ASTM C 311, Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
33. ASTM C 494, Standard Specification for Chemical Admixtures for Concrete.
34. ASTM C 535, Standard Test Method for Resistance to Degradation of Large-Size Aggregate by Abrasion and Impact in the Los Angeles Machine.
35. ASTM C 566, Standard Method of Test for Total Moisture Content of Aggregate by Drying.
36. ASTM C 586, Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method).

37. ASTM C 617, Standard Method of Capping Cylindrical Concrete Specimens.
38. ASTM C 618, Standard Specification for Coral Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
39. ASTM E 4, Standard Methods of Verification of Testing Machines.
40. ASTM E 70, Standard Test Method for pH of Aqueous Solutions With the Glass Electrode.
41. Corps of Engineers - U.S. Army (CRD): CRD C 119, Method of Test for Flat and Elongated Particles in Coarse Aggregates.
42. National Ready-Mixed Concrete Association (NRMCA): Check List for Ready Mixed Concrete Production Facilities.
43. Occupational Safety and Health Administration (OSHA): Safety and Health Regulations for Construction.
44. American Association of State Highway Transportation Officials (AASHTO): T-26, Standard Method of Test for Quality of Water to be Used in Concrete.
45. New York State Department of Transportation (NYSDOT): Standard Specifications, Construction and Materials.

3.8.1.3 Seismic Design

3.8.1.3.1 Initial Seismic Design

The containment is a Seismic Category I structure. It was originally analyzed as a single lumped mass cantilever beam system to determine its natural frequency. For the containment structure, the damping factor as a percent of critical damping was assumed to be 2.0%. The resultant load developed from the maximum horizontal response is distributed in a triangular manner with the base of the triangle at the top of the structure. The stress criteria for the containment and all reinforced concrete members in tension were as described in Section 3.8.1.2.3 based on the response to a ground motion of 0.08g acting in the vertical and horizontal planes simultaneously. Design of the containment was checked to ensure that the combined stresses resulting from gravity, incident, and seismic loadings based on the response to a ground motion of 0.20g acting in the vertical and horizontal planes simultaneously are within the stress limits described in Section 3.8.1.2.3.

The natural period of the first harmonic was determined using an analysis consisting (for horizontal motion) of a cantilever fixed at the base with the mass lumped at the centroid of the structure. Bending stiffness was established based on a Young's modulus of 4.1×10^6 psi and shear stiffness was established based on a shear modulus of 1.8×10^6 psi. No rotation of the foundation material was considered. The natural period of the first harmonic was calculated to be 0.22 sec for horizontal motion and 0.07 sec for vertical motion.

The resultant base shear was established on the basis of the maximum response acceleration (0.46g) for the maximum hypothetical design earthquake considering 2% of critical damping. The resultant load was conservatively assumed to be distributed in the form of an inverted triangle extending the full height of the structure. The resulting maximum meridional forces are as shown in Figure 3.8-9.

3.8.1.3.2 Seismic Reanalysis

As a check on the initial seismic design of the containment it was reanalyzed using normal mode theory with a number of lumped masses. A check was also made on the containment considering the rock foundation as an elastic medium with rotation and translation of the containment considered. This flexible foundation modeling of the containment changed the total shear and overturning moment on the structure by less than 5% as compared to the rigid foundation model. The base shear for the modal analysis on a rigid foundation resulted in an equivalent containment response acceleration of 0.26g as compared to the 0.46g used in design. Comparable results were obtained with respect to overturning moments. As a result of the somewhat more rigorous modal analysis, the containment design can be shown to be highly conservative. A detailed description of the modal analysis follows:

A. The containment structure is modeled as a cantilever consisting of lumped masses connected by weightless springs. This model is shown in Figure 3.8-10.

B. The normal modes are calculated using the computer program SAND. This program is a modified version of a program developed by the Jet Propulsion Laboratory for the dynamic analysis of lumped mass systems. Shear deformations and rotational inertia are included in the program SAND.

C. The input required for SAND consisted of the modal coordinates, member properties, and material properties. These are shown in Figure 3.8-10 and Table 3.8-1. The masses are calculated by the program using a density of 160 lb/ft^3. This is representative of heavily reinforced concrete.

D. The response in each mode is read from the response curves determined for the site, as given in Section 3.7. The deflections, accelerations, and member forces are computed in each mode and are then summed on a square root of the sum of the squares basis. This computation is executed by the computer program SPECTA.

E. The natural frequencies and response are summarized in Table 3.8-1. The mode shapes are plotted in Figure 3.8-11 and the shear forces and bending moments in Figure 3.8-12.

F. The effects of ground motion were investigated by considering the rock as an elastic medium with coefficients similar to concrete:

$E = 3.0 \times 10^6$ psi

$r = 0.2$

The fundamental frequency was reduced from 6.95 Hz to 6.28 Hz. The alterations to the deflections, accelerations, shear forces, and moments were insignificant, being less than 5%.

3.8.1.4 Containment Detailed Design

3.8.1.4.1 Stress Analysis

3.8.1.4.1.1 Analysis Methods

The analysis of the containment structure for operating plus incident load was based upon shell theory analogy.

The containment structure was analyzed for seismic loads as a cantilevered beam with all mass assumed concentrated at the center of gravity. Both shear as well as bending stiffness were considered in determining the fundamental frequency. The total horizontal inertial load was determined from the response curves given in Figures 3.7-1 and 3.7-2 for 2% damping for the fundamental frequency of the cantilevered beam. This total horizontal load was distributed over the height of the containment structure in the form of an inverted triangle to determine the inertial overturning moment. The vertical seismic component was assumed to be unamplified due to the high axial stiffness of the containment.

Stresses induced by the horizontal and vertical components of seismic motion were combined algebraically. The seismic shear distribution assumed was that given for a hollow thin-walled cylinder with shear flow perpendicular to the containment radius and the maximum shear flow equal to twice the average value.

3.8.1.4.1.2 Analysis Results

The results of this analysis for the following loading combinations are shown in Figures 3.8-13 through 3.8-15.

a. Operating plus incident load.

b. Operating plus incident plus design earthquake loads.

c. Operating plus incident plus maximum potential earthquake.

The displacement resulting from the seismic excitation will produce a base shear which is transferred via the base mat to the side walls of the structure by the radial reinforcement. During an incident these bars should be in tension. As the lateral load (i.e., earthquake shear) is imposed, these bars will react similar to a wheel with prestressed spokes with a load applied to the hub and the rim restrained from moving. In this design these members are assumed to have no shear resistance. The load transfer from the radial bars, which established longitudinal shear stresses in the wall, will occur by means of varying circumferential membrane forces in the lower portion of the wall.

The side wall, at loads resulting from the factored pressure (1.5 P), will be uncracked in a horizontal plane due to membrane prestress forces. The only cracking that occurs will be partial cracking due to secondary flexure. The depth of these cracks will be limited by the mild steel reinforcement. At the design pressure there will then be sufficient uncracked section of concrete to limit radial shear stresses to less than the maximum allowable value stipulated in ACI 318-63. Details of the radial shear analysis are provided in Section 3.8.1.4.5.1.

The amount of prestressing force provided in the meridional direction of the cylinder is determined to ensure no resultant tensile stress due to the factored load combinations described in Section 3.8.1.2. Consequently, radial cracking is predicted to be only a result of flexure which is similar to the basis for the derivation of concrete shear capacity and shear reinforcement requirements stipulated in ACI 318-63 for flexural members. The derivation of shear reinforcement requirements at the base to cylinder discontinuity is described in Section 3.8.1.4.4.

The capacity to resist membrane shears is affected by the concrete cracking. Refer to Appendix 3B for the discussion of membrane shears in the vicinity of the large openings.

For the cylindrical portion of the vessel resistance to the vertical shears resulting from the earthquake, loading will be developed in the circumferential reinforcement by dowel action (*Reference 6*). The resulting principal stress in the reinforcement will not exceed 0.95 x yield stress as provided in the design criteria. This design further ensures no failure of the adjacent concrete in bearing. Details of the longitudinal shear analysis are provided more fully in Section 3.8.1.4.5.2.

In the dome, all membrane and shear stresses resulting from the earthquake loading will be developed in the mild steel reinforcing.

The loading on the concrete shell of the containment following an accident must be transmitted to it through the liner. The liner attempts to expand under the combined influence of the temperature and pressure. Since the containment structure may be classed as a thin shell, (the diameter to thickness ratio is 30), it is considered that it would have been valid to treat the temperature rise in the liner as an equivalent pressure increase.

The analysis as performed considered an equivalent liner force occurring at the location of the liner. Such equivalent liner forces were established based on no thermal strain relief at points where concrete is uncracked. Where the liner is insulated, the liner temperature increase was assumed to be 10°F due to accident conditions. Based upon no relief of thermal strains and uncracked concrete, the effect of this temperature rise was converted to an axial force plus a moment about the centroid of this insulated section. As a design conservatism, the elastic expansion of the concrete shell under pressure and temperature loads was not used to reduce the temperature induced stresses.

3.8.1.4.1.3 Analysis for Steam Generator Replacement Dome Openings

In order to support the 1996 Steam Generator Replacement, significant structural analyses were performed to support the creation and restoration of containment dome openings. A finite element approach was used for the structural analyses to determine the containment shell structure's capabilities to support applicable loads during and after the dome opening construction and restoration. The structural evaluation of the concrete shell of the dome was based on ACI 318-63 Code Part IV-B, Ultimate Strength Design. The structural evaluation of the liner system of the dome was based on the 1965 ASME Boiler and Pressure Vessel Code, Section III. These codes are consistent with original design and are shown in Section 3.8.1.2.3.3 and the table in Section 3.8.2.1.1.2.

3.8.1.4.2 Rock Anchors

3.8.1.4.2.1 Rock Anchor Design

The basic criterion for the determination of anchor length was that the pull of the anchor is resisted only by the submerged weight of rock and that the rock offers no tensile strength. This criterion further assumes that the rock breaks out at an angle of 45 degrees to the bond development length of the tendon. This criterion also allowed for any additional loads on the

rock imposed from the inside of the containment vessel. The hold-down capability of the rock in the rock anchor design took into consideration the circular geometry of the vessel.

The design of the rock anchors was based upon the simplified assumption that the rock breaks out at an angle of 45 degrees to the axis of the tendon with the apex of the angle at mid-height of the first stage grout. This implies that the rock failure mode is one of diagonal tension. This assumption of a half-angle of 45 degrees for rock is supported by *References 7*, *8*, and *9*.

Further verification of the conservative nature of this assumption was demonstrated by the rock anchor tests described in Section 3.8.1.7.

The sockets for the rock anchors are percussion drilled into the rock through steel pipe sleeves which are welded into the underside of the bearing plates for the rock anchors and extended through the ring girder. The sockets in the rock plus the pipe sleeves are filled with a neat cement grout in two stages after the rock anchors are installed. Protective steel covers, as shown on Figure 3.8-1, are welded to the bearing plates for the rock anchors to enclose the sidewall tendon to rock anchor couplings. The tendon conduit extending above this enclosure is 6-in. diameter schedule 40 pipe with threaded couplings. This tendon conduit is threaded into a half coupling that is welded to the top of the protective steel cover. In order to permit the required conduit movement, stainless steel bellows are provided. The tendon conduit, including the protective steel cover, is bulk filled with the corrosion protection system described in Section 3.8.1.4.3.4. This filler material is injected through a connection in the protective steel cover. The exterior surface of the containment structure was waterproofed from the edge of the ring girder to elevation 253 ft 0 in. to provide corrosion protection.

3.8.1.4.2.2 Preinstallation Grouting Test

Prior to installing any rock anchors, a test was performed by grouting a rock anchor in a water filled, clear, 6-in. diameter tube. This rock anchor contained ninety 1/4-in. diameter wires with the grout tube and bottom hardware all identical to that proposed for the permanent installation. This test demonstrated that the grout did flow so as to completely encase the tendon. However, it also indicated that the use of bleeder holes near the bottom of the grout pipe, as well as the grout pipe terminating above the bottom of the hole, tended to produce an unacceptable dispersion of the grout. This condition was remedied by deleting the bleeder holes and extending the grout pipe with the addition of a bevel to the bottom of the hole. No tests could be made on the completeness of grouting of permanent rock anchors. However, procedures used for grouting did comply with those found to be satisfactory in the test.

The side wall tendons are coupled directly to the rock anchors. Lift-off readings were made on the side wall tendons that provide a measure of the prestress force at the fixed end (i.e., upper anchor head for the rock anchors). However, in the bonded tendon, it was not possible to measure the prestress in the full rock anchor tendon.

These criteria are identical with those used for dams in the United States and Europe. Confirming information was also obtained from The Cementation Company Limited of Great Britain, a specialty firm whose activity in recent years has been devoted, in large measure, to the prestressing of both existing and new dams, especially in South Africa and Australia.

3.8.1.4.2.3 Previous Applications

Large capacity, post-tensioned anchors designed on this basis have previously been used in a number of dams in Europe, Africa, Australia, and the United States to provide stability for the structures. One of the early applications was the anchoring of the Cheurfas Dam in France 1935. Similarly, prestressed rock anchors have been used for tie backs on retaining walls on a permanent as well as temporary basis and for suspension bridge anchorages. Major structures for which prestressed rock anchors were used are listed in Table 3.8-2. A list of some major applications of the BBRV ninety 1/4-in. diameter wire prestressed rock anchor assemblies is given below.

- Wanapum Dam, Washington; Mayfield Dam, Washington: Rock anchors and trunnion anchors; rock anchors for penstock slope stabilization.
- Boundary Dam, California: Rock anchors for rock stabilization.
- John Hollis Bankhead Dam, Alabama: Rock anchors for dam stabilization.
- Ice Harbor Dam, Washington: Rock anchors.
- Mangla Dam, West Pakistan: Trunnion girder anchorage, main spillway.

The design is based upon the use of the BBRV system developed originally in Switzerland and used extensively for rock anchor applications.

3.8.1.4.2.4 Rock Hold-Down Capacity

Laboratory tests on core representative of rock in the approximate area and depth of the rock anchor installation indicate a bulk specific gravity of the rock of 2.54. Since the rock participating with the rock anchors is below the ground water table, the submerged weight of rock of 96 lb/ft^3 (2.54-1.0) x 62.45) is used in determining the hold-down capability.

The bond development length (first stage grout) for the ninety 1/4-in. diameter wire tendons is computed as follows:

For 0.60 f_u = 635 kips

$$P = \frac{\left(80/60 \times 635000\right)}{\pi \times 6 \times 170 \times 12} = 22.0\,ft.$$

(Equation 3.8-3)

Each rock anchor was initially tensioned to 80% of ultimate strength and the jacking force was then reduced at lock-off to 70% of ultimate. The bond stress assumed between rock and grout is 170 psi. This value was determined to be conservative as demonstrated during the test performed on reduced scale rock anchors and also as reported by the Swiss Federal Laboratory for the Testing of Material (*Reference 10*) and as documented in Grolversuchemit Spannankern an Talsperran der Asterreichen Bunderbahnen und die Anwendung der Vorspannbouweise auf den Talsperrenban, Von A. Ruttner, Wien, Austrian Engineering Journal, 1964. Test data obtained for the John Hollis Bankhead Dam, Warrior River,

Alabama, also confirm the conservatism of a bond development length developed on the basis of the average bond stress of 170 psi between grout and rock.

The diameter of the drilled hole for each rock anchor is 6 in. The assumed breakout angle of 45 degrees to the vertical is most conservative as demonstrated during the reduced scale rock anchor test and in *Reference 7*.

Weight of rock in kips/ft circumference = $0.096d^2$

$$\text{Internal pressure in kips/ft circumference} = \frac{0.072 \cdot pd(2r - d)}{r}$$

(Equation 3.8-4)

The depth, d = 26.5 ft, was established based on preliminary design. No surcharge beyond the internal pressure of the containment vessel was considered to be effective in determining the rock anchors hold-down capability. Therefore, for varying internal pressures the rock hold-down capacity uniform around the circumference of the vessel, was as follows:

Rock Hold-Down Capacity (Kips per ft circumference)	**Internal Pressure (psig)**
0	67.4
60	240.4
69	266.4
75	283.7
90	327.0

3.8.1.4.2.5 Hold-Down Factor of Safety

For the combination of operating plus incident loads (i.e., load combination a) in Section 3.8.1.2.3, the uplift per foot circumference is constant at 259.0 kips/ft which is less than the assumed rock anchor capacity of 327.0 kips/ft. Therefore, the factor of safety on pull-out against the factored load is 1.26. For the structural proof test, uplift per foot circumference was constant at 182.0 kips/ft which was less than the rock anchor capacity of 266.4 kips/ft for a factor of safety of 1.47.

For the combination of operating plus incident plus design earthquake loads (i.e., load combination b), the maximum uplift per foot circumference is 274.1 kips/ft and the minimum is 150.5 kips/ft. This considers horizontal and vertical components of ground motion occurring simultaneously and their effects added algebraically. Due to the group action of anchors, the overcapacity of the rock against lateral loads can be represented by the factor of safety against overturning. This factor, using the rock hold-down capacity based on the pressure load of 75 psig, is 2.38.

For the combination of operating plus incident plus maximum potential earthquake loads (i.e., load combination c), the maximum uplift per foot circumference is 289.2 kips/ft and the minimum is 25.4 kips/ft. The factor of safety against overturning, using the same consideration, is 1.96.

Consideration was also given for seismic loading without internal pressure. For the 0.1g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) there is no uplift.

Minimum downward component is 0.9 kips/ft. The factor of safety against overturning is 4.62. For the 0.2g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) the maximum uplift is 69.2 kips/ft. The factor of safety against overturning is 2.31.

3.8.1.4.2.6 Installation

The tendons are anchored into the rock socket with an expanding grout. The grout contained an additive designed to reduce the water requirement of the cement, to have a slightly expanding action, and to retard the initial set. The expansion based upon original grout volume is 8% ± 2%. This expansion is accomplished by the reaction of aluminum powder with the alkalies of the cements. This reaction results in liberation of hydrogen gas in the form of small bubbles which have an expanding effect. Tests have verified that the molecular form of the hydrogen in the alkaline medium will not adversely affect the steel.

The top (movable) anchor head for the rock anchor is coupled to the bottom (fixed) anchor head of the side wall tendon, as shown in the fully engaged position in Figure 3.8-16. Dimensions and material are as shown. The bushing provides for coupling the smaller diameter fixed head to the larger movable (i.e., tensioning) head. The coupling has right-hand threads on each end.

During construction, after the rock anchors were tensioned, the coupling was set in place on the top head of the rock anchor. When the sidewall tendon was inserted in the conduit, the coupling was threaded onto the bottom head of the sidewall tendon to the end of thread. The coupling was then turned down onto the top head of the rock anchor resulting in all threads on both anchor heads being fully engaged as shown in Figure 3.8-16. The design of the tendon hardware ensures that the hardware remains elastic up to the ultimate capacity of the wires. Therefore, at the effective prestress force of 60% of the ultimate strength of the tendon, average strains in the coupling are designed to be no greater than 60% of the yield strain of the coupling material.

3.8.1.4.3 Tendons

3.8.1.4.3.1 General Design

The design for the containment provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria, the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for ACI 318-63 Chapter 17, Shear and Diagonal Tension - Ultimate Strength Design, and there is a basis for designing shear reinforcing.

The design also provides for anchoring the cylindrical walls to rock with anchors which will be post-tensioned tendons anchored into grouted sockets in the rock. The anchors are designed to resist all membrane stresses in the cylindrical wall. A sufficient physical separation is provided between wall and base slab to ensure that there is no transfer of vertical reaction to the base slab.

In order to produce minimum practical base restraint and to most effectively use the rock anchors (i.e., no moment applied to the ring girder), the design provides for the development of a hinge at the cylinder to base transition using an elastomer pad. The elastomer pad permits a predictable rotation of the hinge with the only restraint to rotation being a minimal resistance due to compression on the pad. The elastomer (neoprene) pad was selected for the hinge because of its predictability of behavior, maintenance-free properties, and ability to withstand environmental conditions far more severe than that associated with the Ginna design. Detailed background data on the use of neoprene bearing pads is included in Section 3.8.1.4.4.3.

Under the dead load of the containment and the application of the prestress force, the elastomer pad will compress vertically approximately 0.08 in. Upon being subjected to the most severe loading combination, the tendon elongates and the pad reverts back to essentially its original thickness (i.e., pre-stress force equals or is slightly greater than membrane forces due to this loading combination). This elongation must extend over a sufficient length of tendon to ensure no yielding of the steel. In an effort to minimize the

increase in wire stresses under load, the tendon is unbonded for the entire length from coupling between rock anchor and anchorage of tendon at the top of the side wall.

A large amount of vertical reinforcement is provided near the outer surface of the wall at the lower elevations. This steel is provided to resist bending moments which occur in the wall due to the base restraint. Mild steel reinforcement provided for flexure is shown in Figures 3.8-4 and 3.8-5. Since the wall has a steel liner on the inside, the minimum mild steel reinforcement required for crack control has been provided on the outside only, in the amount of 0.19% of the concrete cross-sectional area. The prestressing tendon is positioned at the center of the wall section, thus causing the participation of the prestress force to be minimal in resisting bending moments. The design requires all bending or shear stresses to be resisted by mild steel reinforcement, thus making the design quite conventional in the region of bending and shear.

Due to the initial tendon force (0.6 f'_s) the maximum average concrete membrane (meridional) stress is 640 psi compression and the liner (meridional) stress is 4500 psi compression. Considering a concrete creep and shrinkage of 320 x 10^{-6} in./in., the final average concrete membrane (meridional) stress is 550 psi compression and the liner (meridional) stress is 14,100 psi compression. This implies that a linear temperature gradient of 39°F through the concrete shell (i.e., a temperature on the liner side 39°F below the exterior fiber temperature) would result in a zero stress on the inner fiber. This situation is not considered credible.
During MODE 6 (Refueling), the refueling and purge system which has no cooling coils, could not reduce the interior temperature below the external ambient temperature. The containment recirculation fan coolers (CRFC) could possibly reduce the internal ambient temperature but not to the extent required to exceed the foregoing gradient. Therefore a reversal of stresses is not possible and no concern exists regarding crack control on the inner face. As noted above, a minimum mild steel reinforcement has been provided on the outside face in the amount of 0.19% of the concrete cross-sectional area. This amount exceeds the frequently used minimum amount of steel for crack control of 0.15%. The structure has liner insulation (except for a region of the dome) and will consequently not be subject to rapid temperature changes due to fluctuations in the interior ambient temperature.

The sole purpose of prestress is to balance vertical tensile membrane forces in the wall thus allowing confidence in the use of the provisions of ACI 318, Section 1701 and 1702, for shear reinforcement design. Therefore, the prestressing requirements would be those of a tension member rather than of a bending member.

All side wall tendons, can be removed or retensioned. Two tendons are permanently accessible for either operation, while the remainder can be reached by removing concrete at approximately elevation 228 ft (see Figure 3.8-2) to obtain access to the coupling enclosure. Any tendon can be uncoupled from the rock anchor for removal by opening a window in the coupling enclosure.

The two permanently accessible tendons are located on the south side of the containment vessel, and have the coupling enclosure exposed in the auxiliary building sump (Figure 3.8-2). A bolted door on the coupling enclosure permits removal and inspection of the tendons without removing concrete.

A failure of an unbonded tendon or tendons in the upper portion of the wall would result in a loss of prestress in the section of the wall subjected to bending and shear; however extensive failures of this type would cause a tensile failure in the wall, thus making a secondary shear failure at the base of little consequence.

3.8.1.4.3.2 Seismic Considerations

In evaluating the relative safety of a tendon for a prestressed concrete structure subject to seismic loads, consideration was given to the stresses in the tendon (the ninety 1/4-in. diameter wires) and to the tendon anchorage.

The design for Ginna is based upon a dynamic analysis using a basic ground acceleration of 0.2g. The design does not consider the ultimate strength and plastic deformation of the structure but considers only an elastic response with damping selected on the basis of such a response.

Other considerations that are generally recommended for seismic design and are incorporated in the design are (1) to provide a symmetrical structure thereby avoiding the torsional effect produced by structure rigidity and (2) include sufficient rattle space between the containment shell and adjacent structures, including the structures within the containment, to avoid any possible physical interaction as the structures deflect independently under the seismic load.

By using unbonded tendons, high local strains or elongations can be distributed over the length of the tendons. Another problem is the control of cracking in the concrete. In *Reference 1*, T. C. Waters and N. T. Barrett state that an adequate amount of bonded reinforcement or the bonding of a portion of the prestressing tendons will ensure that cracking of the concrete is uniformly distributed and that concentrations of large local tensile strains at particular points will be avoided. In the Ginna design where cracking might occur due to flexure produced by discontinuities, bonded mild steel reinforcement is used to control crack spacing and width. Where flexural stresses are minimal, bonded mild steel reinforcement is also provided to control the spacing and width of cracks, thereby serving to increase the ultimate capacity of the structure. The concrete containment is not susceptible to a low temperature brittle fracture. This conclusion is consistent with information provided in the First Supplement to the PSAR.

For a flexural member, there may be merit in localizing a wire failure in that the loss of pretress force might not extend over a region where maximum flexural capacity is required. This would be especially true for a failure at or near an anchorage. However, the design provides for prestressing tension, not flexural, members and there is no similar advantage in localizing the failure of a tendon in a tension member.

The behavior of the anchorage hardware is of prime importance when the element is subjected to reversal of loading produced by the dynamic loads from an earthquake. The anchorage system for this design, the BBRV (buttonhead) system, was chosen because of its positive anchorage and excellent properties when subjected to cyclic loadings. The BBRV system used parallel wires with cold formed buttonheads at the ends which bear upon a perforated steel anchor head, thus providing a positive mechanical means for transferring the prestress force. The buttonheads on the wire are formed by cold upsetting to a nominal

diameter of 3/8 in. on the 1/4 in. diameter wire. Professor Fritz Leonhardt (*Reference 11*) reports that "Extensive tests show that this BBRV 'buttonhead' provides a reliable anchorage, even under dynamic loading conditions, if an anchor of softer steel (ST 52 to ST 90), provided with an appropriate bore (opening for wire) is employed." The anchor heads for the Ginna design are fabricated from C1141 steel, which is a softer steel than the wire heads approximately equivalent to ST 70 covered under the German Specification DIN 17 100.

Fatigue tests were conducted by the Swiss Federal Testing Station (EMPA) in 1960 on individual 7-mm wires with upset heads and on tendons consisting of eighteen 7-mm wires each. The anchorage heads for the tendons were for 22-wire units but the number of wires was limited by the capacity of the testing apparatus. The tests on individual wires indicate that 7-mm wire with upset heads is capable of sustaining 2,000,000 stress application cycles with an upper limit of about 1301 kg/mm^2 (180 ksi) when the lower limit is 95 kg/mm^2 (135 ksi). Several tests were conducted on the 18-wire tendons. The results of the one test with stress limits most similar to that used for design of prestressed concrete are summarized below.

With a lower limit of 95 kg/mm^2 (135 ksi), the tendon withstood over 2,040,000 stress application cycles to an upper limit of 111 kg/mm^2 (158 ksi) without any of the wires fracturing. Only after the upper limit was raised to 113 kg/mm^2 (160 ksi), did one of the wires break after an additional 113,000 stress application cycles. The rate of stress applications was 350 cycles per minute.

Cutting tolerance for the test tendon was plus or minus 0.5 mm. The ratio of tolerance to total wire length for the test tendon is 1/2377, which compares with 1/3210 for the rock anchors and 1/4800 for the side wall tendons. The ultimate strength of the wire being tested was 160 kg/mm^2 (225 ksi).

Therefore, it is concluded that dynamic loads, considering especially pulsating loads resulting from an earthquake, do not jeopardize the buttonhead anchorage.

The tendon bearing plates are 18.5 in. in diameter with a 5.5 in. center hole. Considering uniform bearing, the concrete bearing pressure due to the initial tendon force (742 kips) is 3040 psi. This compares with an allowable stress (ACI 318-63, Equation 26-1) of 3720 psi. The maximum splitting force (*Reference 11*) due to the initial tendon force considering no concrete tension is 58.0 kips, based upon tension extending from 6 in. to 30 in. below the bearing plate. The required reinforcing is 1.45 in.^{2}/ft compared with the furnished 5/8-in. diameter spiral at 2-in. pitch with an area of 1.86 in.^{2}/ft. The calculated spalling force (*Reference 12*) is 22.2 kips/tendon for which No. 7 reinforcing bars were provided at 12.75-in. centers.

Bond development of the spiral reinforcement is not considered relevant. The reinforcement for spalling is anchored in excess of ACI 318-63 requirements. Experience indicates that long-term loadings will not degrade the integrity of the anchorage zone.

A Seismic Committee was established by the Prestressed Concrete Institute to develop guidelines for the design of prestressed concrete structures for seismic loads. In their report (*Reference 13*), the Prestressed Concrete Institute provides detailed guidelines for the design of

prestressed concrete structures for seismic loads. These guidelines apply to bonded and unbonded tendons. The Ginna design has been reviewed in light of this report and has been found to comply with all guidelines.

3.8.1.4.3.3 Stressing Procedure

Stressing of tendons is accomplished by hydraulic jacks and pumping units which are equipped with dial gauges that indicate the pressure in the system within plus or minus 2%. The stressing procedure is as follows:

a. Stress by pumping until the required overstressing force is reached with backup provided by direct measurement of differential displacement of the tendon head and bearing plate made to the nearest 1/32 of an inch.
b. Insert shims, filling the space as completely as possible.
c. Reduce pressure to seat the anchor head on the shims.
d. Take lift-off reading and record.
e. Adjust shims as necessary.

The pattern and sequence of post-tensioning was established so as to provide basically for initially tensioning every 40th tendon of the total 160 tendons and then in a systematic manner to tension the tendons approximately midway between previously tensioned tendons. This approach minimizes the loss due to elastic shortening. The elongation of the side wall tendons during the stressing operation was approximately 8 in.

The philosophy behind this sequence of post-tensioning was as follows:

aa. To provide in each stage of stressing an essential symmetric loading on the containment cylindrical wall and neoprene pad at the base.
bb. The prestress load was to be applied as far as practical symmetrically with respect to the two large access openings.
cc. The curved tendons around the large access openings were to be retensioned after 1000 hours in order to counteract the time dependent losses due to shrinkage, creep and steel relaxation. The retensioning was required in order to fulfill minimum prestress requirements up to the end of plant life, which is 40 years.

The highest tendon stresses occur during the jacking operation which, in effect, pretests the tendon including all hardware prior to the application of a pressure load. The effective prestress considering all losses (i.e., 60% of ultimate stress) is 144,000 psi. Upon subjecting a tendon to the most severe loading combination (design-basis accident plus maximum earthquake), the tendon stress increases by 4.6%, i.e., 6,600 psi.

The effective prestress forces were developed in all tendons in accordance with normal industry practice. All tendons were initially tensioned to 80% of ultimate stress and then locked-off at 70% of ultimate stress. Basically all tendons are straight. A limited number have a minor curvature where they are draped around small penetrations. The tendons in all cases are located in a relatively large (6-in. diameter) rigid conduit which was sized to permit

the bottom anchor head to pass through. Any wobble and friction losses will be less than 24,000 psi or 10% of the ultimate stress. The remaining losses consist of elastic shortening, concrete shrinkage and creep, creep of the elastomer pads, and steel relaxation. Anchorage losses are negligible for the length of tendon being used. The tendons are protected to ensure that there are no loss of wires due to corrosion.

The tendon temperature never sufficiently exceeds that resulting from plant operation and high ambient temperatures external to the containment. The average daily temperature of the tendon will, therefore, never exceed approximately 90°F.

The prestressing sequence for the rock anchors was generally as follows:

i. Initially, every fourth anchor was tensioned. Horizontal spacing of anchors, as shown in Figure 3.8-2 is 2 ft 1 9/16 in.
ii. Secondly, every second tendon not included in item 1 was tensioned.
iii. Finally, all remaining anchors were tensioned.

The tensioning of side wall tendons was done using a minimum of four jacks spaced generally about the circumference of the structure. Stressing positions were alternated to prevent concentrations of multiple stressed tendons adjacent to multiple unstressed tendons. This was accomplished by tensioning tendons in a sequence wherein the tensioned tendon was approximately equidistant between previously tensioned tendons. The four jacks were used so that the resultant of the prestress force remains approximately symmetrical around the circumference of the structure.

3.8.1.4.3.4 Corrosion Protection

A steel conduit (6-in. diameter Schedule 40 pipe) is embedded in the side wall concrete to permit insertion of the prestressing steel tendon and in addition provide electrical shielding against stray ground currents. The conduit is specially designed where it passes through the elastomer pads so as not to jeopardize the action of the hinge by using a bellows-type expansion joint.

The 6-in. ϕ threaded pipe is screwed on to a 6-in. ϕ half coupling. This connection meets the criteria specified in the standard code for Power Piping, USAS B31.1.0 - 1967, and as such provides a leak-proof joint.

The wire was protected prior to fabrication to ensure that the surface was free from any imperfections other than a light oxide film. Prior to shipment, the tendon was protected with a coating of NO-OX-ID "490," manufactured by the Dearborn Chemical Division of W. R. Grace and Company. The NO-OX-ID "490" provides a light coating satisfactory for temporary protection. Following insertion of the tendons in the conduit, the conduit was filled with NO-OX-ID "CM" so as to provide bulk filling of the void in the conduit. An expansion reservoir is provided at the top anchorage as shown on Figure 3.8-7. Access to this reservoir is provided as shown on Figure 3.8-17. The tendon conduit is filled by pumping the NO-OX-ID "CM" in at the level of the tendon coupling and venting from the top anchorage.

The water table is approximately 16 ft above the bottom of the tendons. The tendon and its conduit are approximately 110 ft high. This leaves a hydraulic head of filler material of 94 ft which is equivalent to about 36 psi above the highest point of the water table. This ensures that there is no water seepage into the conduit.

This is an underestimation of the pressure required to displace the filler material in that it is based upon the material having the viscosity of water with no friction loss and a specific gravity of 0.9. During the actual placement of the filler material, a pressure of 42 to 45 psi was required to pump the material after it was agitated.

The radial tension bars, as shown on Figure 3.8-2, are protected against corrosion as follows:

a. In the cylinder wall the bars are coated with grease. (The grease ensures there is no bond development).

b. In the base slab, the bars are inserted in a pipe sleeve for a length of 2 ft 10 in. The annular space between bar and pipe sleeve is filled with the corrosion protection system described above for the side wall tendons.

c. The remaining length of the bars in the base slab are in intimate contact with the concrete.

The buttonheads at the rock anchor heads are encased in grout to provide continuity of environment along the full length of the wire. The movable (top) anchor heads for the side wall tendons are protected by covering the head with the NO-OX-ID "CM" made to prevent rain water from entering the conduit by the expansion reservoir. The top anchor heads can be inspected for corrosion by unbolting the cover on the expansion reservoir shown on Figure 3.8-7 and removing the wax covering the heads. The wax can also be sampled by this method.

NO-OX-ID "CM" casing filler is composed essentially of a selected paraffin-base refined mineral oil, blended with a microcrystalline petroleum-derived base (petrolatum) of definite melting point and penetration range. Additives consisting of lanolin, and sodium petroleum sulphonates are incorporated as water-displacing surface-active agents and corrosion inhibitors. The proportion of oil to microcrystalline wax in the formulation is adjusted to give a pour or gelling point within the range of 110 to 120°F. The oil and wax are highly refined long-chain saturated paraffinic petroleum derivatives, resistant to oxidation and chemical or physical degradation, within the temperature ranges to which they will be exposed in this service. The lanolin is a polar substance which enhances inhibitor performance and wetting of the metal surface by the microwax blend. The petroleum sulphonate is a surface-active, water displacing corrosion inhibitor of long tested merit. (See Table 3.8-3.)

Quality Control Tests

Quality control determinations on required raw materials and on the finished NO-OX-ID "CM" protective coating included those tests already being done in the standard raw material inspection procedures, plus additional controls requested on chloride, sulfide, and nitrate content. The latter tests included the following:

aa. Chlorides - The initial screening test on both raw materials and finished produce was the sensitive Beilstein Test. This is a flame determination using an oxidized copper carrier.

A green or blue-green color appears in the flame if chlorides (halides) are present. This test detects as little as 0.5 ppm halide. If a positive Beilstein indication is obtained, a confirming test is made on water extracts of the product, using standard titration or colorimetric procedures described in ASTM D-512-62T. (Note: A positive Beilstein test may be obtained when halides are not present, because of interferences from traces of pyridines, thiourea, thiocyanate, etc. This is the reason for a confirming titration on water extracts, following a positive Beilstein indication).

bb. Sulfides - The method used was a water extraction followed by a total sulfide determination. Zinc acetate was added to the extraction water to precipitate sulfides. Sulfides present were then measured in accordance with Paragraph 8 of ASTM D-1255. This method detects as little as 0.1 ppm sulfide. An alternate colorimetric procedure also was available in which sulfides are volatilized from an acidified extraction solution, to create a colored spot on zinc acetate paper. Spot intensity is measured to determine sulfide. The extraction procedure is described in ASTM D-1255.

cc. Nitrates - The method used was a water extraction followed by colorimetric measurement, based on ASTM D-992-52. Either the Brucine or phenoldisulfonic acid procedures were used. Either can detect as little as 0.01 mg/l nitrate.

dd. Cathodic Protection - All of the tendons are connected to the liner of the containment and then to the copper grounding system. Also, electrically connected to the grounding system is the mild steel reinforcement below the high ground water level. Permanent and stable potential reference cells are installed at significant locations to measure the corrosion potential.

At the time of containment construction, Durichlor anodes were installed around the perimeter of the vessel. Protective current can be applied from these anodes and regulated as needed to maintain a protective potential if cathodic protection is found necessary by measurements from the reference cells.

In addition, sacrificial steel cable has been installed next to all bare copper cables. Also, four potential bridge pipe test stations were installed on the rock anchor system to measure the magnitude of the earth potential current gradient caused by current flow into or out of the rock anchors and to provide a basis for regulating any applied current from the anodes.

A British study of the problem indicates that cold drawn perlitic wire of the type employed in the Ginna containment is not susceptible to stress corrosion cracking failure (*Reference 14*).

3.8.1.4.4 Hinge Design

3.8.1.4.4.1 Tension Bars

A hinge was developed at the base of the cylinder wall by supporting the wall vertically on a series of elastomer bearing pads and anchoring the wall horizontally into the base mat with radially positioned, high-strength steel bars. The bars are approximately 20 ft long and 1-3/8 in. in diameter with two anchor plates and with a minimum ultimate tensile strength of 145,000 psi and a yield strength of 130,000 psi. The bars conform to ASTM A322-64a and ASTM A29-64 and are spaced approximately 1 ft-1 in. on centers at the centerline of the side-

wall. The anchor plates conform to AISI C-1040 and can develop 100% of the bars' ultimate strength. The bars are unbonded over a predetermined length to provide for an elongation of the bar under load consistent with that required for the rotation of the wall with the elastomer pad acting as a hinge. The only rotational restraint on the base of the wall is that produced by the resistance of the elastomer pads to deformation. Actual tension bar stresses resulting from the factored loads are as follows:

Loading Combination	Bar Force, kips	Bar Stress, ksi	% Yield Strength
A	130	87.5	67
B	149	100.0	77
C	170	114.3	88

The effect of the base to cylinder discontinuity is based upon equations developed for the analogy of a semi-infinite beam on an elastic foundation (*References 15* and *16*) in which the spring constant for the circumferential bars and liner is taken as the foundation modulus. As such, the hoop stiffness is generated independently of the concrete. The elastic modulus of the uncracked concrete is assumed to be equal to

$$4.1 \times 10^{6}\ psi\ (E = \omega^{1.5}\sqrt{33}\, f'_c\ from\ ACI\ \ 318\text{-}63)$$

(Equation 3.8-5)

The assumption on a single elastic modulus, which is considered to be a reasonable upper limit, is conservative in that it results in the highest discontinuity stresses.

Except for participation in anchoring the radial tension bars at the base of the cylinder, the base slab is not an integral part of the containment shell for this design. The loads on this slab, which is more properly described as a cap on the rock, are those from the interior structures.

A simple check was made, based on an assumed 45-degree bearing distribution, to ensure that rock bearing pressures do not exceed the limits listed in Appendix 2C.

The means for transferring the radial reaction at the base of the cylinder into the foundation rock is shown in Section 1-1 of Figure 3.8-2. The base reaction is transferred from the radial dowels into the ring girder and thickened portion of the base slab and thence, as a lateral load, on the rock outboard of the ring girder. The concrete for the ring is placed directly against the rock. The load is transferred to the rock on the interface from elevation 231 ft 8 in. to elevation 224 ft 8 in. The maximum allowed lateral pressure is 25,000 pounds per square foot as stipulated in Appendix 2C. Where no lateral rock support is available at the auxiliary building sump, a special beam and struts are required to span this area as shown in Sections 2-2 and 3-3 of Figure 3.8-2.

The details of the expansion joint in the tendon conduit at the hinge are shown in Figure 3.8-18. This is a stainless steel bellows as conventionally used on process piping for expansion joints. The bellows are 6-in. diameter, stainless steel pipe bellows complying with requirements of the ASA B31.1 Code for Pressure Piping. They provide movement capability for the rigid tendon conduit at the hinged joint to ensure a sealed tendon enclosure which retains the grease corrosion protection around the tendon and also seals against contaminants gaining access to the tendons. The bellows also provide essentially no resistance across the hinged joint to the movements. The inside diameter of the bellows is approximately 5.6 in. and the diameter of the tendon bundle is approximately 3 in. With 1.3 in. clearance and maximum predicted horizontal movement at design loads of 0.2 in., margin is available to preclude contact.

3.8.1.4.4.2 Liner Knuckle

The liner design at the hinge provides for a base to cylinder transition in the form of a knuckle with a 10-in. radius. This detail provides sufficient flexibility as the sidewall moves with respect to the base during the tensioning of the sidewall tendons and under the application of the design loads.

The stresses in the liner base to sidewall transition knuckle have been determined for the following cases. The analysis was based on the method described in *Reference 17*.

a. Under the application of the prestress force plus the dead weight of the vessel, the sidewall moves vertically downward 0.08 in. with respect to the base. The maximum bending stress in the knuckle due to this motion is 25 ksi.
b. Under loading combination a, the sidewall moves vertically upward 0.08 in. with respect to the base, and radially outward 0.08 in. The maximum bending stress in the knuckle following the movement is 10 ksi and membrane stress is 1.2 ksi. This loading combination represents the most severe loading on the knuckle.

The calculated stress for the tension bars at the base of the cylinder listed in Section 3.8.1.4.4.1 were based upon the assumption that the stiffness of the base is a function only of the tension bars. A study was made to validate this assumption. It was found that the liner knuckle offers negligible restraint to radial motions but does offer very significant restraint to lateral (horizontal earthquake) motions.

The dimensions of the liner knuckle are shown on Figure 3.8-19. The method of solution involved the use of a shell computer program based on the solution described in *Reference 17*, wherein stresses were determined on the basis of a lateral translation of Point A (Refer to Figure 3.8-19). It was conservatively assumed that the support lines for the knuckle remain circular. For the lateral motion the calculated spring constant of the knuckle is 785,000 k/in. Based upon a maximum earthquake shear force at the base of the cylinder of 12,080 k it is determined that the maximum shear stress in the knuckle is 16.4 ksi. Bending stresses are small.

3.8.1.4.4.3 Elastomer Bearing Pads

Each bearing pad is a flat pad 1.628-in. thick, made of two layers of 55 durometer hardness neoprene between three steel shims. The outer shims are 16 gauge and the middle shim is 10 gauge carbon steel.

The pads are placed between the cylinder walls and the ring beam. Because of the ability of neoprene to deform, it provides an effective medium of load transfer. By conforming to surface irregularities uniform bearing is provided. No lubrication or cleaning is necessary for the bearing. The pad dimensions are 9 in. x 42 in. and two pads were placed between each pair of pre-stressing tendons.

Each pair of pads will carry a maximum load of 371 tons resulting in a bearing pressure of 980 psi. This pressure is reduced to 840 psi after prestress losses occur. Both pressures are well within allowable values. A pad under load should not exceed a vertical deflection greater than 15% of the thickness. The steel shims being used reduce the calculated strain to 5.2, as further verified by the tests reported in Section 3.8.1.7.1. The creep of neoprene pads is dependent on the hardness of the neoprene which was the reason for using low hardness (55 durometer) pads. Creep as verified by tests is estimated to be 13% of initial deflection.

On most of the circumference of the containment, the elastomer pads are accessible or could be made accessible by removing insulation to view from one side.

Specifications for the elastomer pads are summarized in Section 3.8.1.6.6.

Neoprene pads have been in use since 1932 so that the practice at the time of the Ginna containment design was based on over 30 years of experience and research. These pads were first used in France in the late 1940s as the load transfer bearings between piers and beams. In the United States and Canada, development more or less paralleled the use of precast, pre-stressed concrete beams because of the problem of seating such beams. By 1957, concrete bridges had been built with neoprene bearings in Texas, New Hampshire, Rhode Island, and Ontario. At the time of the Ginna containment design, thousands of bridges and buildings throughout the world have been built using neoprene bearing pads.

The neoprene pads will have a local effect on seismic shears at the base. This effect however is comparable to Saint-Venant effects which are present locally at any discontinuity. The seismic design of containment for shear and moment loads as a cantilever beam is not affected by the neoprene pads since the cylindrical shell is tied to the base by means of the vertical pre-stressing.

The effect of vertical cracking of the containment shell under pressure loading will tend to reduce the stiffness of the containment which in turn, for the modal analysis discussed in Section 3.8.1.3, will increase the period and response of the structure. However this same cracking will tend also to increase structural damping and thereby reduce the structural response. Considering the large design margin contained in the actual seismic design of the containment as compared to that dictated by the more rigorous modal analysis presented in Section 3.8.1.3, the local perturbations caused by use of neoprene pads are not sufficient to affect design adequacy.

A typical properties specification for bridge bearing pads (the hardness Shore A 50 approximately applying to the pads to be used for the containment) is given by the American Association of State Highway Officials as follows:

Original Physical Properties			
Hardness Shore A	50 ± 5	60 ± 5	70 ± 5
Tensile, minimum psi	2500	2500	2500
Elongation at break, minimum %(ASTM D-412)	400	350	300
Ozone, 1 ppm in air by volume, 20% strain, 100 ± 2°F, 100 hours	No cracks	No cracks	No cracks
Compression set 22 hours at 158°F, maximum %	25	25	25
Oven Aged 70 hours at 212°F			
Hardness pts. change maximum	0 to ±15	0 to ±15	0 to ±15
Tensile, % change maximum	±15	±15	±15
Elongation, % change maximum	-40	-40	-40
Low Temperature Stiffness at -40°F			
Young Modulus, maximum psi	10,000	10,000	10,000
Tear. Die C lb/ in minimum	225	225	225

3.8.1.4.5 Concrete

3.8.1.4.5.1 Radial Shear

The maximum value of radial shear is 253 psi and this occurs 3 ft above the highest stressed radial tension bar under the combination of operating incident and maximum credible earthquake loads (load combination c). The critical section for shear is taken 3 ft above the radial tension bar level to conform with the requirements of ACI 318, Section 1701. The ultimate shear capacity of the reinforced wall without shear reinforcement as defined in ACI 318 1701 is 126 psi. Shear reinforcement is required and is provided according to the requirements of Section 1702 as No. 7 bars at 11-in. centers. Thus, under the conditions of 60 psi internal pressure and 0.2g simultaneous earthquake (load combination c), the shear capacity of the containment wall is sufficient to resist the maximum shear stress which occurs at only one position on the circumference.

Under the combination of operating and incident loads (load combination a) the maximum shear stress which occurs uniformly around the wall is 183 psi, which is 78% of the ACI design code capacity of 253 psi. Under the combination of operating, incident, and design earthquake loads (load combination b), the maximum shear stress occurring at one point in the containment wall is 222 psi, which is 88% of the design capacity.

The detailed analysis for shear design under load combination 3 is as follows:

The ultimate shear capacity of the wall is

$v_c = \phi\ 1.9\ f'_c + 2500\ (P_w\ V_d/M) = 126$ psi

The actual maximum shear stress is

$v = 109000 / (12 \times 36) = 253$ psi

whence the shear carried by stirrups is 127 psi.

Placing stirrups at 11-in. centers, the required cross-sectional area of bar using 0.85 yield stress is:

$$A_v = \frac{V'_u\ S}{\phi \cdot f_y \cdot d} = \frac{127 \times 36 \times 12 \times 11}{0.85 \times 40,000 \times 36} = 0.494 in^2$$

(Equation 3.8-6)

No. 7 bars having an area of 0.60 in.^{2} per bar are therefore placed at 11-in. centers.

3.8.1.4.5.2 Longitudinal Shears

Under the combination of loads resulting from the simultaneous occurrence of maximum earthquake and loss-of-coolant accident, the internal pressure of 60 psi will produce vertical

cracks in the cylindrical wall (maximum concrete tensile stress would be 970 psi). The capacity of the wall to resist longitudinal shears across these cracks due to the seismic loads with internal pressures is developed by the dowel action of the circumferential reinforcement.

In determining the capacity of the circumferential reinforcing bars as dowels, first the capacity of the concrete in bearing is checked and then the capacity of the bars in combined tension and shear is checked.

The concrete strength is calculated to limit the capacity to transfer shear to a dowel capacity of 38.7 kips per bar or an average shear stress of 9.7 ksi in the reinforcing bar (*Reference 6*).

In considering the strength of the reinforcing to resist shear stresses due to the dowel action and to resist tensile stresses due to the pressure load, the Mohr circle method is used to combine stresses. It is recognized that the failure mode of mild steel is one of shear. The strength envelope on the Mohr circle is a straight line parallel to the normal stresses axis at a shear stress magnitude of 19.0 psi (1/2 x 0.95 yield stress). The tabulation in Table 3.8-4, broken down as to factored load combinations, shows the allowable shear stress for a given tensile stress (due to pressure load) and the allowable tensile stress for a given longitudinal shear stress (due to lateral seismic load).

As indicated in Table 3.8-4, in every case where there is dowel action there is a margin of safety on the shear capacity of the reinforcing steel. In all cases, however, the capacity of the bar in shear is limited by the concrete in bearing and not by the steel in combined shear and tension. It should also be noted that this analysis considers only the outer ring of circumferential reinforcement for which the tensile stress is maximum.

This entire analysis is developed on the capability of the circumferential reinforcement to resist longitudinal shears with no reliance placed upon the liner capability or aggregate interlock. It is recognized that the longitudinal shear will be resisted by the interaction of dowels and liner but that the composite action will not jeopardize the integrity of the liner.

3.8.1.4.5.3 Horizontal Shear

The horizontal shear due to lateral seismic load is transferred to the cylindrical wall of the containment through the horizontal radial tension bars provided at the base. The bars act in a manner analogous to spokes of a wheel in transferring shear.

The forces in the bars have been analyzed by assuming the wall to be a stiff ring. This analysis gives an overestimate of bar force and leads to a conservative radial bar design. However, a wall section acting as a horizontal ring at the base of the vessel must also be checked as a ring for bending and shear stresses that result from differential radial tension bar forces. The worst condition for this effect will occur with 0.2g earthquake resulting in a maximum differential force between any one bar and the adjacent one of 1.55 kips. This force differential produces a moment and shear in the wall section (considering a one foot height of ring) of 1.66 kips-ft and 1.55 kips respectively. From the circumferential bar layout in the region of the wall adjacent to the radial bars this moment and shear will be resisted by a minimum of four 18S bars.

Assuming a totally cracked wall section in this region (which is not the case as circumferential hoop tensions are very small in this region) the capacity of these four 18S bars in shear is 155 kips compared to the calculated shear of 1.55 kips and in bending is 303 kips-ft compared to a computed moment of 1.66 kips-ft. Thus the wall has more than adequate capacity to resist the small moments and shears produced by any radial force differentials in tension bars.

There are two general types of bond failure. ACI 318-63 addresses the most common type of bond failure produced by a splitting type failure (i.e., concrete cracking longitudinally along the bar). The second type is that produced by shearing the concrete by the bar deformations or by shearing off the bar deformations.

It is recognized that cracks normal to the bar will reduce the bond capacity. This conduction is analogous to that occurring in a flexural member where reinforcement is subjected to tensile stresses. The code advises that splicing at points of maximum tensile stress should be avoided wherever possible but provides for using a reduced allowable bond stress where such a splice is unavoidable (refer to ACI 318-63, Section 805). Such a condition is not uncommon, as evidenced by common practice for splicing bars in negative moment regions of rigid frames.

Cracking parallel to a reinforcing, although undesirable, is controlled by the strength across the crack provided by reinforcement usually associated with an orthogonal arrangement of bars. This condition is the basis for concern for splices occurring close together for a series of bars where spirals or closely spaced stirrups are suggested for use.

It should be noted that the development of rebar bond in a prestressed structure is less severe than in conventional reinforced concrete structures such as buildings, chimneys, and tanks. On this structure the reinforcement for which bond development is required to effect the anchorage consists only of the steel required to accommodate rotational strains or to control cracking. The interrupted reinforcement where bond is relied upon does not serve as primary membrane reinforcement.

Although temperature changes may affect the crack width on the containment during MODE 5 (Cold Shutdown), it is not considered to significantly change during plant operation. Because of the time lapse between construction and plant operation, the change in strains due to concrete shrinkage is extremely small. Because of the conservative design limits established to ensure an elastic response to transient loads, the crack widths should not change due to the design earthquake loads.

3.8.1.4.5.4 Anchorage Stresses

The stresses for the anchorage of the tendons and the dome reinforcement in the vicinity of the dome to cylinder transition were analyzed and compared with *Reference 11*. The maximum bursting stress caused by the tendon anchorage is 180 psi, compared with an allowable stress of 300 psi. The maximum spalling stress is 465 psi which required the addition of reinforcement. The maximum concrete compression under maximum load at the zone between the anchorages of the tendon and the dome reinforcement is 650 psi, compared with an allowable stress of 1250 psi. The anchorages for tendon and reinforcement are separated so as to minimize overloads of anchorage stresses.

The design provides for a factor of safety of 2.2 times the factored load against shear failure at this location. Details of the anchorage zone in the dome to cylinder transition are shown in Figure 3.8-5.

3.8.1.4.5.5 Shell Stress Analytical Procedures

The analytical procedures used for the stress analysis of the shell are summarized in the following paragraphs.

Base to Cylinder Discontinuity

The analysis considered a stiffness circumferentially of 116.5 lb/in.^{2}

$(k = (A_s)(E_s/2) = 116.5\ \text{lb/in}^2)$

Based upon the analogy of a semi-infinite beam on an elastic foundation (*References 15* and *16*) it can be shown for the model described in Figure 3.8-20 that:

$$\text{Deflection}: y = \frac{e^{-\beta X}}{2\beta^3 EI_z}[P_0 \cos\beta X - \beta M_0 (\cos\beta X - \sin\beta X)]$$

(Equation 3.8-7)

$$\text{Rotation}: \phi = \frac{e^{-\beta X}}{2\beta^2 EI_z}[-P_0 \sin\beta X + \cos\beta X + 2\beta M_0 \cos\beta X]$$

(Equation 3.8-8)

$$\text{Moment}: M = \frac{e^{-\beta X}}{\beta}[P_0 \sin\beta X - \beta M_0 (\sin\beta X + \cos\beta X)]$$

(Equation 3.8-9)

$$\text{Shear}: v = e^{-\beta X}[P_0 (\cos\beta X - \sin\beta X) + 2\beta M_0 \sin\beta X]$$

(Equation 3.8-10)

Symbols that are not defined on Figure 3.8-20 are as follows:

E = Young's modulus for beam material

I_z = Moment of inertia of beam

$$\beta = \sqrt[4]{\frac{k}{EI_z}}$$

(Equation 3.8-11)

k = Foundation modulus

It can also be shown that:

Hoop Force: $F_\theta = r\,(p - ky)$

$$\text{Base reaction: } P_0 = \frac{2p\ \beta^3 EI_z}{k}\ (\beta M_0)$$

(Equation 3.8-12)

Symbols not previously defined are as follows:

r = average radius of shell

p = internal pressure

All stress resultants, shears, and moments were calculated on the basis of the foregoing equations. Because of the use of the hinge, the moment at the base of the cylinder (M_o) consists only of the restraining moment produced by the elastometer bearing pads and pseudo-moment applied to ascertain the effect of thermal stresses.

No inclined bars (i.e., bent shear bars) are used on the containment structure. As shown on Figure 3.8-4, stirrups are used at the base of the cylinder to an elevation 10 ft 5 in. above the base. This structure is prestressed vertically and, with the hinge design at the base, is subject only to bending stresses and not to tensile membrane stresses in the longitudinal direction. Therefore, the stirrups are anchored in concrete subject to only vertical cracks due to membrane loads. As shown on Figure 3.8-4, Section 9-9, the stirrups are continuous around the structure. Consequently, anchorage is provided both by bond and by the mechanical attachment to the vertical bars on the inside face.

In general, there are two types of bond failure (*References 18* and *19*). In one type of bond failure the concrete surrounding the bar splits along the reinforcing steel. In the other, the splitting does not occur but the concrete between the deformations in the reinforcement is sheared off, thus leaving a round hole in solid concrete. For the splitting failures, the tensile strength of concrete, distance between bars, and the magnitude and distribution of lateral stress acting on the bars are important variables affecting the bond strength. The bond limits, including lapped splice requirements in ACI-318, are based upon tests in which the failures were splitting type failures. Since the bond tests were made on beams, there was an absence of lateral confining stresses. The bond strength for splitting failures would most certainly be lower than the bond strength where the failure is the shearing off of the concrete between the reinforcing steel deformations. Confinement caused by lateral pressure can change the failure from "splitting" to "shearing" and increase the bond strength considerably (*Reference 19*).

The exact increase due to lateral pressure is not known because the tests were run on small size specimens that would have little to do with any actual bond stress situation occurring in practice. It is known that in simple beam tests, the effect of the confinement at the support increases the bond strength. Where confinement is included in the design, the actual bond strength would appear to be higher than the design values permitted by ACI-318. Consequently, for the configuration of stirrups used in the cylinder to base juncture it is considered that ACI-318 design limits on anchorage provide a conservative basis for the design.

Dome to Cylinder Discontinuity

The analysis was based upon general shell theory (*Reference 20*) using the model shown in Figure 3.8-21. At a distance sufficiently removed from the discontinuity it can be shown based upon membrane theory that:

$$\delta_c - \delta_d = pr^2 \frac{1 - \frac{\nu_c}{2}}{E_c t_c} - \frac{1 - \nu_d}{2E_d t_d}$$

(Equation 3.8-13)

Symbols not previously defined are as follows:

δ_c = Normal displacement of cylinder

δ_d = Normal displacement of dome

ν_c = Poisson's ratio for cylinder

ν_d = Poisson's ratio for dome

E_c = Young's modulus for cylinder

E_d = Young's modulus for dome

t_c = Shell thickness of cylinder

t_d = Shell thickness of dome

In calculating the quantities Q_o and M_o it is assumed that the bending is of a local character and, therefore, that the bending is of importance only in the zone of the spherical shell close to the joint and that this zone can be treated as a portion of a long cylindrical shell. It can therefore be shown that

$Q_o = 1/Z\ (\delta_c - \delta_d)$

$M_o = 1/Y\ (\delta_c - \delta_d)$

where Z and Y are functions of dome and cylinder stiffnesses.

Base, Cylinder, and Dome

The calculated stress resultants ($N\phi$, $N\theta$), stress couples ($M\phi$, $M\theta$), meridional shears ($V\phi$), and radial displacements (δ_R) for dead load, final prestress, operating temperature (winter and summer), internal pressure, accident temperature, and earthquake are as listed in Table 3.8-5. These loads were combined as shown in Table 3.8-6. The results for the load combinations are as shown in Appendix 3C.

The physical constants used in the analysis described above were as follows:

Uncracked concrete

$E = 4.1 \times 10^6$ psi

$G = 1.8 \times 10^6$ psi

$\nu = 0.15$

Cracked concrete

$E = 0$

$G = 0$

$\nu = 0$

Rebar/liner

$E = 29 \times 10^6$

Shrinkage and creep for the prestressed concrete were assumed to be 320×10^{-6} in./in.

For the data tabulated, the analytical model considered was always the cracked model associated with the accident condition.

On the basis of the foregoing data the liner stresses at selected load combinations (refer to Table 3.8-6 for load combinations) are as follows:

	Cylinder (X = 60 ft)		
Load Combination	**$\sigma\phi$**	**$\sigma\phi$**	**Dome Apex**
1	-14.3 ksi	-2.6 ksi	-2.4 ksi
3	-10.7 ksi	+0.1 ksi	-0.2 ksi
29	-2.9 ksi	+27.0 ksi	

The discontinuity stresses between the dome and cylinder were determined by considering the following:

a. That the dome concrete cracks in tension and the cylinder concrete cracks vertically in tension. The radial deformations of the cylinder and the dome are conservatively assumed to be a function of the reinforcing steel alone. The steel areas across the discontinuity are established so as to develop a compatibility of stresses and therefore also of deflections.

b. That neither the upper part of the cylinder nor the lower portion of the dome concrete cracks and that the difference in deflection of the cylinder and the dome some distance from the discontinuity is a function only of the concrete properties. The solution, as developed in Theory of Elasticity, by Timoshenko and Goodier (*Reference 21*), assumes that the lower portion of the dome behaves in a manner similar to that of a cylinder (i.e., the discontinuity moments and shears are rapidly dissipated and become minimal at a limited distance from the discontinuity). For this condition only a nominal shear and moment (4 k/ft and 18 k ft/ft) would be developed due to the most severe factored loads.

c. That the radial deformation of the cylinder some distance from the discontinuity is a function of the cracked concrete section, and the radial deformation of the dome is a function of the uncracked section. The probability of vertical cracks in the cylinder propagating into the dome is remote. The discontinuity shears and moments resulting from the condition are excessive and require the assurance by development of planes of weakness in the concrete that cracking will occur uniformly across the discontinuity.

The discontinuity stresses can be calculated with greater confidence based upon a model of cracked concrete above and below the transition. To ensure that a condition does not exist where either the pressure load produces significant cracking of concrete in the dome at the discontinuity or vertically in the cylinder, crack initiators are used to permit a uniform propagation of tension cracks in the concrete at the discontinuity.

The safety against shear (or tension) failure at the dome-cylinder intersection was investigated by the following two approaches:

aa. An ultimate strength solution based on the Mohr-Coulomb failure criteria for concrete and plane failure surfaces.

bb. An elastic solution in which the stresses were calculated at the point of maximum splitting tensile stress given by Leonhardt (*Reference 11*). The principal stresses at the point were obtained and the stability of the section verified by assuming a direct relationship between tensile and compressive strengths which was obtained from several investigators.

The first approach indicated a collapse load 2.16 times larger than the factored load of (0.95 D + 1.5 P) while the second solution led to a safety factor of 2.12 referred to the same load. On the basis of this analysis, it is concluded that the factor of safety against shear (or tension) failure at the dome-cylinder intersection is greater than the overall safety factor of the containment structure.

The section between anchorage plates for the tendons and the dome reinforcement was also checked using the analogy of a corbel and reinforcement provided as recommended by Kriz and Raths (*Reference 22*).

The dome reinforcing bars are mechanically anchored in the precompressed zone below the top anchors of the tendons. This mechanical anchorage is in the form of Cadweld connections arc welded to a continuous mild steel plate. No bond development is required to fulfill the design requirements. ACI 318-63 limits on splicing are developed upon bond requirements based on a splitting type of failure (*References 18* and *19*). These requirements are not relevant to the design of the containment anchorage.

It was not necessary to stagger the dome anchor plate from an engineering standpoint. Common practice in regular reinforced concrete structures is to stagger splices and, if possible, the anchorage of reinforcing steel. However, in this instance, anchorage is developed by mechanical means in a region of membrane compression. This conservative anchorage environment negates the need for the staggering of splice plates.

3.8.1.4.6 Insulation

The liner insulation is Vinylcel as manufactured by Johns-Manville. This material is a closed-cell polyvinyl chloride foam insulation with low conductivity, low water absorption, and high strength. The insulation is 1.25-in. thick with a density of 4 pcf.

The function of the liner insulation is to limit the mean temperature rise of the liner to 10°F at the time associated with the maximum pressure as shown on the transient for the factored pressure (90 psig). For this determination the containment vessel internal ambient temperature is assumed to be 120°F and 100% relative humidity and the external ambient temperature is assumed to be minus 10°F. The insulation is covered with a metal sheeting. The insulation is capable of withstanding periodic compression of 60 psig within a temperature range of 40°F to 120°F and a single compression to 69 psig within the same temperature range, both without any detriment or change to the insulation properties.

The results of a series of tests which have been performed are included in Section 3.8.1.7.1. Also included in that section are the results of an analog study of the insulation when subjected to the pressure and temperature transients associated with an internal pressure of 90 psig.

Hypothetical Local Insulation Failure

If a local failure of insulation is hypothesized at a typical piping penetration, the circumferential liner stress at the point of failure is calculated as a compression stress of 6.3 ksi at design pressure and temperature. This stress compares with a tensile stress for the insulated liner of 18.5 ksi. Due to this secondary effect, the tensile stress of the mild steel reinforcement would be locally increased but that would not alter the ultimate capacity of the section.

The vertical liner stress would increase locally at the point of insulation failure until the plate yielded in compression. The consequential loss of prestress would be distributed over the full height of the wall. Considering a 2-ft dimension for the area without insulation, the loss of

prestress of the affected tendon would be approximately 1%. The loss of prestress for the entire vessel would consequently be minimal.

The effect of insulation failure at a penetration would be to produce yielding of the sleeve circumferentially in compression and longitudinally in bending.

3.8.1.4.7 Liner

3.8.1.4.7.1 Vibrations

The main sources of liner vibrations are vibrating pipes which pass through the liner. The vibrations from these pipes are transferred to the liner from the penetration sleeves. The piping systems expected to vibrate are the following:

Pipe	Frequency of Vibration
Main steam line	13 Hz
Feedwater line	13 Hz
Charging line	1.8 to 18 Hz
Cooling pump seal water line	1.8 to 18 Hz

During a plant design life of 40 years such penetrations may be subject to full stress reversals under operating conditions which are in excess of 2,000,000 cycles. The inner end plate and sleeve of these penetrations were designed for this condition using the stress limitations of the ASME Nuclear Vessel Code.

Regarding cyclic loads due to earthquakes, the anticipated number of cycles (50 to 250) will not require reduction in the stress limits. However, as these vibrations are carried into the concrete shell through the sleeve, which is an extremely stiff member relative to the liner, the degree of participation of the liner in absorbing these vibrations is small, being a function of the sleeve movements at the sleeve liner weld connection. Due to the rigidity of the penetration and its method of fixture to the concrete sleeve, movements at this weld interface are negligible.

3.8.1.4.7.2 Anchorage Fatigue Analysis

The sidewall liner is anchored to the concrete with steel channels of 3-in. depth on approximately 4-ft 3-in centers. The channels are intermittently welded to the liner. The channels ensure elastic stability of the liner under potential compression loads and also provide the required capacity to resist instability due to vacuum loads. The steel channels had the added function of stiffening the liner during erection.

3.8.1.4.7.3 Base Slab Liner

Backup bars in the form of structural tees were embedded and anchored into the 2-ft 0-in. thick base slab as shown on Figure 3.8-6. These backup bars, all of which are continuous,

were placed flush with the top concrete surface. The liner plate was placed on the concrete surface and the butt joint made as shown on the typical joint detail on Figure 3.8-6. Tolerance on height is ±3/8 in. and out-of-flatness is 1/4 in. in 10 ft.

After nondestructive testing of this weld (liquid penetrant examination), the test channels were installed and leak tested. The nominal 24-in. concrete cover was then placed and the test channels were again pneumatically tested. The liner seams and the channel to liner welds were found to be leaktight. No grout was placed between the base slab and the liner.

A nominal 24-in. concrete cover was placed over the liner. Therefore, the liner is located at mid-thickness of the concrete. The walls of the reactor cavity are assumed to act as a shear key with the required capacity to transfer earthquake loads. Consequently, the test channels should not be subject to a significant shear load.

The concrete cover placed on top of the liner does not necessarily ensure intimate contact between the liner plate and the base slab over the entire plan area, but does ensure that sufficient bearing exists to adequately distribute vertical loads from columns and walls to the base slab. All shear loads are assumed transferred by means of the walls of the reactor cavity, which acts as a shear key. Refer to Figure 3.8-3 for reactor cavity wall details.

3.8.1.4.7.4 Liner Stresses

The maximum nominal liner stress (meridional direction), considering shrinkage and creep of concrete, is 14,100 psi compression.

The liner was reinforced about all openings in accordance with the ASME Unfired Pressure Vessels Code (i.e., by replacing the cut-out area of 3/8-in. liner plate). Normally this involved the use of a common 3/4-in. plate for a group of penetrations. Minimum spacing of penetrations conforms to ASA N 6.2-1965, Safety Standard for Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors. The liner stress concentration at the hole is determined based upon elasticity solutions for a flat plate of constant thickness subjected to a biaxial stress field.

The combination of stresses from all effects is combined in accordance with the ASME Nuclear Vessels Code, Article 4, and evaluated on the basis of the allowable peak stress intensity, which for the liner material is 60,000 psi.

The data provided in Table 3.8-4 and the description contained in Section 3.8.1.4.5.2 do not consider the liner as resisting earthquake shears. It can be shown that the principal stress resultant is oriented nearly horizontal in that the shear component is small relative to the axial components. Nevertheless, the same model previously used where dowel action was considered was reanalyzed to determine the interaction between concrete, reinforcing bars, and liner. This analysis conservatively assumed that the liner and concrete shell acted compositely.

The maximum longitudinal shear at the base of the cylinder (i.e., on an axis normal to the direction of ground motion) due to the 0.2g ground acceleration is 67.2 k/ft. The shear modulus of the liner G_L [E/2(1 + V)] equals 11,200 ksi. The effective shear modulus of the

concrete wall is based on pure shear on the uncracked concrete plus the dowel action of the horizontal reinforcement across the hypothesized vertical crack. The conservative assumption was made that the shear stiffness across the crack is not increased by aggregate interlock.

The dowel stiffness is established on the basis of a load-slip relationship of 3000 kips/in. which is a linear relationship for the motions calculated in this study. The shear modulus of the cracked wall section G_W equals

$$\frac{G_c}{1+\frac{A_c G_c}{3000\cdot L}}$$

(Equation 3.8-14)

where terms are as defined on Figure 3.8-22.

The results of this study are summarized as follows:

Cracking Spacing L (in.)	**Gw (ksi)**	**Linear Shear τ_L (psi)**	**Concrete Shear τ_C (psi)**
25	188	5200	87
12	95	7700	65
8	65	9000	53

As a check on the allowable liner shear stress, a Mohr's circle was used based upon a critical shear stress of 16 ksi (1/2 σ γ) as shown on Figure 3.8-23.

It is thus shown that the allowable shear stress exceeds the calculated shear stress based upon these conservative analytical models. It should be reiterated that these calculated stresses in no way represent expected response to the loading being considered, but instead represent an upper bound based upon a simplified model.

3.8.1.4.7.5 Liner Buckling

The liner anchors in the cylinder are 3-in. deep channels spaced horizontally at approximately 4 ft 4 in. on centers. The liner is analyzed as a flat plate, which is a conservative assumption in that the liner will have to buckle against its own curvature. For analysis it is assumed that the liner is fixed at the angles and that there will not be any differential radial moments of the boundaries. The liner anchors are designed and spaced so that the critical buckling stress will be greater than the liner stress under operating or incident conditions. In the case of a cylinder, considering conservatively a uniaxial stress field, the critical buckling stress is 99,000 psi, which compares with a maximum stress of approximately 4000 psi.

Details on the channels attached to the liner as anchors are shown in Figure 3.8-24.

The containment structure was designed to use reinforcing bars with a minimum yield stress of 40,000 psi, as this basis leads to stress levels in the liner which ensures that it does not yield when the containment is at test pressure. The calculated maximum tangential liner stress in the cylinder due to the test pressure load is 26,500 psi (tension). This compares with a calculated liner stress due to the factored accident loads (1.5 P = 90 psig) of 28,700 psi (tension). The thermal gradient is considered in developing these stresses for accident conditions, but not for test conditions. In neither case is the calculated stress equal to nor greater than the yield stress. The meridional liner stress in the cylinder under both test and accident conditions is compressive; this and the meridional or circumferential stresses in the dome are lower than those listed above.

Cylinder Liner

In view of the large shell radius to liner thickness (630/0.375 = 1680) and shell radius to support spacing (630/52 = 26) ratios, a flat plate idealization is considered to be fully justified.

The steel liner is therefore considered to be a flat, thin, isotropic plate supported with line supports against a rigid wall as shown on Figure 3.8-25.

The buckling pattern of the panel plate is a wave surface. Therefore, the equations derived for a wave surface are used where the deformation pattern of the panel plate is as shown on Figure 3.8-25.

From the large deflection analysis of clamped plates under biaxial compression it can be shown that:

$$\frac{W_0^{\,2}}{a} = \frac{1}{\frac{1966}{400} + \frac{9}{4}\nu - \frac{1066}{400}\nu^2}\left[\frac{2}{3}\frac{t^2}{a} + \frac{3}{4\pi^2}(1+\nu)(\epsilon_1+\epsilon_2)\right]$$

(Equation 3.8-15)

Since W_O equals zero at the onset of buckling

$$\left[\frac{2}{3}\frac{t^2}{a} + \frac{3}{4\pi^2}(1+\nu)(\epsilon_1+\epsilon_2)\right] = 0$$

(Equation 3.8-16)

Therefore, under operating conditions, when $\in_2 = -\nu\in_1$

$(\in_1)_{CR} = -9.65\ (t/a)^2$

$(\sigma_1)_{CR} = E\in_1 = -9.65 \in (t/a)^2$

For this structure wherein plate thickness is 3/8 in. and spacing between vertical anchors is 49.5 in.

$(\sigma_1)_{CR} = -9.65 \times 30 \times 10^3 (0.375/49.5)^2 = 16.6$ ksi

This applies for operating conditions only. A similar analysis is also performed for accident conditions wherein $\in_1$ is compression and $\in_2$ is tension.

Using the notation $f = N_1/N_2 = P_1/P_2$ and where $\in_1/\in_2 = (P_2 - \nu P_1)/(P_2 - \nu P_2)$, it can be shown that

$$(\sigma_1)_{CR} = -11.6 \frac{1 - f\nu}{(1-\nu)(1+f)}$$

(Equation 3.8-17)

Therefore, if f is negative, as would be the case for this structure, the critical buckling stress $(\sigma_1)_{CR}$ continues to increase as σ_2 increases in tension. In summary

F	**$(\sigma^1)_{CR}$**	**σ_2**
0	-16.6 ksi	0
-0.125	-19.6	+2.4
-0.25	-23.8	+6.0
-0.375	-29.8	+11.2
-0.50	-38.1	+19.0

For this structure with the insulated liner the operating condition represents the most severe condition for the stability analysis.

From *Reference 23* it is shown that for an initial displacement Y_o and the initial deflection curve, defined as:

$Y = Y_o/2[1 - \cos 2\pi (X/L)]$

that the equivalent liner strain equals

$\in_L = 1/4 (\pi_O/L)^2$

For this structure it can then be shown that for varying amounts of Y_o the resulting liner strains ($\in 2$) are as follows:

Yo	Yo/L	$\in 2$	$\sigma 2$(psi)	N2 (lb/in.)
0.1 in	2.02×10^{-3}	1.01×10^{-5}	303	11.4
0.2 in	4.04×10^{-3}	4.00×10^{-5}	1200	45.0
0.3 in	6.06×10^{-3}	9.09×10^{-5}	2727	102

The welded connection between the anchor and the liner consists of a staggered 3/16 in. fillet weld on both sides of the flange; of 1.5 in. length in 4 in. This weld has a shear capacity of approximately 2.5 k/in., which obviously is sufficient capacity for possible liner dimensional imperfections.

The liner anchor connection is designed for the differential shear load, caused by a buckled liner panel, which is equal to the load in the adjacent panel under normal operating of the plant. Under internal pressure loading, the liner will be in tension in the hoop direction.

Deviation in liner anchor spacing within normal erection practice for pressure vessels will not affect liner stability or liner anchor design. Liner hoop compressive stresses are negligible during winter operation of the plant. The liner is insulated and thermal stresses are insignificant. Therefore, a local poor or inadequate weld between liner and anchor will not cause any danger with respect to liner stability.

The effect of a liner panel erected out of roundness between two adjacent anchor points can be defined as follows:

a. Under operation of the plant, the liner hoop compressive force in the neighboring panel can be transferred directly in shear to the nearest liner anchor. (See above.)
b. Under internal pressure loading, the liner hoop tensile force will be redistributed to other parts of the liner, and possibly also to the hoop reinforcing steel until the liner is being engaged to resist additional hoop stresses as the pressure load increases.

Variations in liner material yield strength are not significant in that predicted operating/accident loads are always significantly less than minimum yield. The calculated liner stresses are tabulated in Appendix 3C.

The interior of the liner below elevation 346 ft (15 ft above the dome springline) in the dome area and the cylinder can be inspected after the insulation has been removed. The liner in the dome above this elevation can be directly inspected.

Dome Liner

See Section 3.8.2.3 for a discussion of the dome liner stress analysis.

3.8.1.4.7.6 Liner Corrosion Allowance

No corrosion allowance has been included in the design of the liner, which has a minimum thickness of 0.25 in. The exposed surface of the liner has been given a protective coating of paint. The cylindrical portion is protected by insulation.

The outer surface of the steel is in direct contact with the concrete, which provides adequate corrosion protection due to the alkaline properties of concrete. The external underground surface of the concrete shell has a membrane waterproofing system to act as a seal for protection against underground water.

3.8.1.5 Penetrations

3.8.1.5.1 General

All penetrations through the containment reinforced concrete pressure barrier for pipe, electrical conductors, ducts, and access hatches are of the double barrier type. Typical electrical and pipe penetrations are shown on Figure 3.8-26.

In general, a penetration consists of a sleeve embedded in the reinforced concrete wall and welded to the containment liner. The weld to the liner is shrouded by a test channel which is used to demonstrate the integrity of the joint. The pipe, duct, or access hatch passes through the embedded sleeve and the ends of the resulting annulus are closed off, generally by welded end plates. Piping penetrations have a bellows type expansion joint mounted on the exterior end of the embedded sleeve where required to compensate for differential motions. The only exceptions to providing an annulus about piping occurs for the three drain lines from sump B. Details of these penetrations are shown on Figure 3.8-27.

All welded joints for the penetrations including the reinforcement about the openings (i.e., sleeve to reinforcing plate seam) are fully radiographed in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels, except that nonradiographable joint details are examined by the liquid penetrant method. For fully radiographed welds, acceptance standards for porosity are as shown in Appendix IV of the Nuclear Vessels Code. The remaining liner weld seams are examined by spot radiography. (The ASME Unfired Pressure Vessels Code states that porosity is not a factor in the acceptability of welds not required to be fully radiographed.) Verification of leaktightness is by means of pressurizing test channels.

Penetrations are designed with double seals so as to permit individual testing at design pressure. In this case, an adulterant gas method is used. An air distribution system is provided for periodic testing.

All penetrations are provided with test canopies over the liner to penetration sleeve welds. Each canopy, except those noted below, is connected to, and pressurized simultaneously with, the annulus between to the penetration pipe and sleeve when under test. The exceptions are the canopy for the fuel transfer penetration, which must be pressurized independently of the annulus because of the separation posed by the transfer canal liner; and the three pipe penetrations in sump B, in which only the canopies are pressurized as there are no annuli.

For details of small penetrations analysis, refer to Section 3.8.1.5.6.

3.8.1.5.2 Electrical Penetrations

There are generally five types of electrical cable penetrations required, depending on the type of cable involved:

Type 1 High voltage power, 4160 V.

Type 2 Power, control and instrumentation: 600 V and lower.

Type 3 Thermocouple leads.

Type 4 Coaxial and triaxial circuits.

Type 5 Fiber Optic

All five types of penetration designs are a cartridge type, basically as shown on Figure 3.8-28. The cartridge length and the support of cables immediately outside containment are designed to eliminate any cantilever stresses on the cartridge flange.

Type 1 penetrations use a rubber insulation copper rod. This insulated rod passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge. High alumina insulating bushings are used as an alternative to provide the double barrier.

Type 2 penetrations use single or multi-conductor mineral insulated cable with a metallic sheath. The cable passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge. The ends of the mineral insulated cable are potted with an epoxy resin compound.

Type 3 penetrations are similar to Type 2 except that the conductors are thermocouple material. The sealing and terminations are identical to Type 2 penetrations.

Type 4 penetrations are used principally for coaxial and triaxial circuits. Each cable passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge similar to that employed in the other penetration types. Inside the cartridge, between the double barrier, a plug and receptacle connection is provided to block leakage through the cable itself.

The Type 5 penetration assembly consists of a stainless steel header plate/extension tube, feedthrough modules containing fiber optic conductors, and a stainless steel support plate system for the feedthrough modules. Additional components such as a penetration monitoring tube/plug and fill valve, are provided for leakage surveillance of the penetration. The feed-through modules containing the fiber optics pass through the header plate and are secured and sealed to the plate with specially designed Midlock stainless steel compression fittings. These fittings are installed from the outer face of the header plate and are concentric with the feedthough modules.

These penetrations are designed to permit as much shop fabrication and testing as possible and minimize on-the-job fabrication. At the same time, double barrier protection and accessibility for in-place testing is maintained.

In general, shop fabrication and quality control are used in all penetration designs where practical. For example, penetration sleeves are shop welded to certain liner plates in specified locations, and transition welds between carbon and stainless steel are shop welds.

3.8.1.5.3 Piping Penetrations

Piping penetrations are provided for fluid-carrying pipes and for air purge ventilating piping. Most pipes penetrating the containment connect to equipment inside and outside of the containment, and are for either high temperature or moderate- to low-temperature service. Other pipes, such as for purge air, connect the containment volume to the outside atmosphere.

In all cases, a piping penetration consists of an embedded sleeve with the ends welded to the penetrating pipe. Provision is made for expansion with bellows type joints forming a testable compartment in the case of hot lines. Further, in the case of the high-temperature pipe lines, the penetrations are designed so that the temperature of the concrete around the penetration does not exceed ASME III, Division 2, Subsection CC-3340, Item (a) limits. For normal or any other long-term period concrete temperatures shall not exceed 150°F except for local areas around the penetration, which are allowed to have increased temperatures not to exceed 200°F. For accidents or any other short term period the temperatures shall not exceed 350°F for the inner surfaces in containment except local areas are allowed to reach 650°F from steam or water jets in the event of a pipe failure. The high-temperature pipe lines use a forced air cooling system, connected to cooling coils integrated with the penetration sleeves. The cooling coils are in the form of an embossing welded directly to the inner surface of the penetration sleeve as shown on Figure 3.8-29. The cooling air exit temperature is monitored and can be related to the concrete-to-sleeve interface temperature. A prototype test was performed under simulated operating conditions to verify assumptions made for hydraulic and thermal calculations. In addition, provisions are made to insert and monitor thermocouples at approximately mid-thickness of the concrete wall at the concrete to sleeve interface in most of the air cooled penetrations (12 of 15), and these enable exhaust air temperature and maximum concrete temperature to be correlated.

The modes of isolating these pipes during a high-pressure containment incident are covered in Section 6.2.4.

3.8.1.5.4 Access Hatch and Personnel Locks

An equipment hatch, constructed of welded steel and having a double-gasketed flange and bolted dished door, is located near grade. The equipment access opening has a diameter of 14 ft.

All major components were moved into the containment prior to installation of the hatch. The hatch barrel is embedded in the containment wall. All weld seams at the joint between the barrel and the liner have test channels for periodic leak testing. For components of the hatch, including barrel and door, test channels are not provided. Details of the equipment hatch are shown in Figure 3.8-30.

An equipment hatch closure plate is available for use when in the MODE 5 (Cold Shutdown) or MODE 6 (Refueling) modes when the equipment hatch is removed. The plate is bolted to

containment in place of the equipment hatch. The closure plate has a hatch door that provides an emergency means of containment egress and provision for temporary services needed during an outage to be brought into containment while still providing containment closure. The closure plate is designed to maintain containment closure during a fuel-handling accident, prohibiting excessive radiological releases. It is designed to withstand a pressure load of +0.5 psi to -0.5 psi. Plant operating procedures restrict the containment pressure differential to 0.5 psig when the closure plate is in place. The plate has a gasket system that when bolted down provides an airtight mechanical fit. No leak testing is required. The closure plate and its storage supports are Seismic Category I. As an alternative during MODE 5 or MODE 6, the equipment hatch opening can be isolated by an installed retractable overhead door. The retractable door is attached to a concrete enclosure built around the equipment hatch opening outside of containment.

Two personnel accesses are provided. One personnel hatch penetrates the dished door of the equipment hatch. The other is located diametrically opposite the equipment hatch. Each personal hatch is a hydraulically-latched double door, welded steel assembly. An equalizing valve connects each personnel hatch with the interior of the containment vessel for the purpose of equalizing pressure in the personnel hatch with that in the containment. Hatch closures are of the double-tongue, single gasket type. The access locks are properly interlocked to ensure door closure at all times, as defined in Section 12.3.2.2.7, with annunciation in the control room, except as allowed in Technical Specification 3.9.3 (Containment Penetrations). Details of the personnel hatch are shown on Figure 3.8-31.

For details of the analytical approach for large opening reinforcement design refer to Appendix 3B.

<u>3.8.1.5.5 Fuel Transfer Penetration</u>

A fuel transfer penetration is provided for fuel movement between the refueling transfer canal in the reactor containment and the spent fuel pool (SFP). The penetration, as indicated by Figure 3.8-32, consists of a stainless steel pipe installed inside a larger pipe. The inner pipe acts as the transfer tube and connects the reactor refueling canal with the spent fuel pool (SFP). The tube is fitted with a standard stainless steel flange in the refueling canal and a stainless steel sluice gate valve in the spent fuel pool (SFP). The outer pipe is welded to the containment liner and provision is made, by use of a special seal ring, for freon gas leak testing of all welds essential to the integrity of the penetration.

The fuel transfer penetration, like all other penetrations, is anchored in the containment shell. Because this anchor point moves when the containment vessel is subjected to load, expansion joints are provided where the penetration is connected to structures inside and outside of the containment vessel. Since the penetration is located on a skewed angle, not normal to the containment shell, the expansion joints are subjected to both radial and tangential (lateral) motions. The expansion bellows inside the containment vessel provide a water seal for the refueling canal and accommodate thermal growth of the penetration from the anchor, as well as the pressure and earthquake produced motion of the anchor (the containment shell). The gasketed expansion joint accommodates motion of the sleeve within the containment shell relative to the portion of the sleeve anchored in the wall of the refueling canal in the auxiliary

building. Section A-A on Figure 3.8-32 indicates a pipe to detect leakage of ground water into the penetration through the gasketed joint. The expansion bellows inside the auxiliary building performs the same function as described for that within the containment.

3.8.1.5.6 Typical Penetration Analysis

3.8.1.5.6.1 Loss-of-Coolant Accident

The concrete temperature adjacent to piping penetrations is limited to 200°F (see Section 3.8.1.5.3). The penetrations for high-temperature pipe lines employ air-cooled coils integrated with the penetration sleeves. The test of a prototype penetration indicated that sufficient margin existed in the design to permit an 80-min period of no coolant flow before the temperature at the interface with the concrete reached 150°F. Backup fans are provided for the air coolant with a capacity of 100% of the design requirement. The concrete shell is not designed for the two-dimensional thermal gradients in the area of the piping penetrations. The typical one-dimensional thermal gradients used in the design are shown in Figure 3.8-8.

The radial deformation of a hole in a plate subjected to a stress field is determined by performing an integration of the tangential strains around the periphery of the hole (*Reference 21*). The increase in the diameter of a hole (δ_D) due to a biaxial stress field (S and S") at a location in the direction of this stress field (S) is as follows:

$$\delta_D = \frac{1}{E}\int_0^{\pi}(S - 2S\cos 2\theta - [S'' - 2S''\cos(2\theta - \pi)])r\sin\theta d_\theta$$

(Equation 3.8-18)

$\delta_D = (2/3)(r/E)(5S - S')$

This corresponding elongation of the plate which would occur if the hole were not present over a length, r, is

$\delta = [2(S + \nu S')/ E]\ r$

The above derivation neglects the stiffening effect of the penetration sleeve and thus overestimates the hole distortion.

The average liner stress (horizontally) due to a loss-of-coolant accident, defined as S, is a tensile stress of 14.1 ksi. (The liner is thickened from 3/8 in. to 3/4 in. around the penetration.) The average liner stress (vertically), defined as S, is a compression stress of 10 ksi.

The maximum increase in diameter of the hole, which is in the horizontal direction for this 10-in. diameter penetration, is then:

$$\delta_D = \frac{\frac{2}{3} x5 \cdot (5x14.1 - 10)}{30x10^3} = 0.006710 in.$$

(Equation 3.8-19)

To simplify the analysis and to provide a conservative result, it is assumed that this deformation is uniform around the circumference of the penetration sleeve. Based upon this assumption:

Maximum moment sleeve = f/4 λ per inch.

Radial deformation due to constant line load, r = $fr^2\lambda / 2E\ t$.

Maximum hoop stress in sleeve = $fr\lambda / 2t$.

In the above equations:

f = line load at the liner sleeve interface

r = radius of sleeve

λ = $3(1-v^2)/ R_2^2\ t^2$

ν = Poisson's ratio

R_2 = mean radius of sleeve

t = wall thickness

The material used for the penetration sleeves is SA-106, grade B, with a minimum yield strength of 31,000 psi at 300°F and an allowable stress intensity (S'_m), per the ASME Nuclear Vessels Code of 20,000 psi at 300°F. The stresses produced at the liner-penetration sleeve interface are defined in the ASME Nuclear Vessels Code as secondary bending and membrane stresses and are therefore limited to a maximum value of 60,000 psi (3 S'_m).

For the 10-in. diameter penetration sleeve using Schedule 80 pipe

$$\Delta r = \frac{0.00671}{2} = \frac{fx5^2 x0.746}{2x30x10^6 x0.594}$$

(Equation 3.8-21)

f = 6400 lb/in. circumference

Maximum bending stress f_b = 6400 x 6 / (4 x 0.746 x 0.594^2) = 36,500 psi

Maximum hoop stress f_t = 6400 x 5 x 0.746 /(2 x 0.594) = 20,200 psi

Therefore, both the maximum bending and hoop stresses are less than the allowable stress of 60,000 psi. Thus, the use of Schedule 80 (10-in. nominal diameter pipe of SA-106, grade B) material was satisfactory for this penetration sleeve.

The material used for the end plates is SA-201, grade B, with a minimum yield strength of 28,350 psi at 300°F and an allowable stress intensity (S'_m) per the Nuclear Vessels Code of 18,000 psi at 300°F.

For a typical 6-in. diameter pipe penetrating the liner through a 10-in. diameter sleeve, the resulting moment and axial force at the anchor on the pipe, which is the end plate, from a thermal flexibility analysis based on normal operating conditions are 1500 lb-ft and 200 lb. Using an end plate thickness of 3/4 in., the maximum bending stress due to the applied moment is 6840 psi and due to the axial load is 4800 psi. The sum of the stresses (11,640 psi) is less than the allowable value.

3.8.1.5.6.2 Loss-of-Coolant Accident Plus Earthquake

A typical 6-in. diameter pipe line is analyzed for the combination of 0.2g ground motion and the loss-of-coolant accident (60 psig). The one pipe line generates an equivalent static force of 1500 lb due to the excitation by the 0.2g ground motion.

This force is resisted at the anchorage by a combination of shear and compression on the sleeve. For this given load, two extreme conditions were analyzed, one with the resulting load applied parallel to the axis of the sleeve and the other with the load applied normal to the axis of the sleeve.

For the case with the load applied normal to the penetration axis and the sleeve of Schedule 80 - 10-in. diameter pipe, the maximum shear stress is 1530 psi and the maximum bending stress is 2470 psi. Due to internal pressure of 60 psig, the axial load on the penetration is 4710 lb. The resulting stresses in the sleeve are a maximum compression of 2775 psi and a minimum compression of 2165 psi.

For the case with the ground motion parallel to the axis of the penetration sleeve, the resulting stresses in the sleeve are a maximum compression of 374 psi and a minimum compression of 305 psi.

From this analysis, the seismic loads on a 10-in. diameter penetration sleeve arising from approximately 100 ft of 6-in. diameter pipe produce small stresses in the penetration elements.

The deformation of the penetration as previously determined is then applied to the liner sleeve and bending and hoop stresses are calculated. This approach is most conservative in calculating tensile stresses since the hole deformations are calculated neglecting the restraining effect of the sleeve and the sleeve stresses are considered to be a function of the total hole deformation.

For a typical piping penetration the stresses calculated on this basis are as follows:

	Leak Rate Test	Loss-of-Coolant Accident
Average membrane stress in liner adjacent to sleeve	+18.8 ksi	+14.1 ksi
Maximum circumferential stress in sleeve	+28.0	+20.2
Maximum bending stress in sleeve	+50.6	+36.5

The review of penetrations indicates that the maximum tensile stresses in the penetration elements occur during the leak rate test and not during the simultaneous occurrence of the loss-of-coolant accident plus the earthquake. By defining leaktightness (i.e., the area of holes in the liner) as a function of tensile stress in the penetration elements, it can be shown that the leakage would be greatest during the test.

3.8.1.5.7 Penetration Reinforcement Analyzed for Pipe Rupture

The penetrations for the main steam, feedwater, blowdown, and sample lines are designed so that the penetration is stronger than the piping system and that the containment is not breached due to a hypothesized pipe rupture combined, for the case of the steam line, with the coincident internal pressure. These penetrations were analyzed for the bending moments, torques, shears, and axial loads transmitted by the pipes. The penetration sleeves were analyzed based upon elasticity theory with the maximum principal stress not exceeding yield stress. The piping connected directly to the primary coolant system, not including the sample lines, are anchored in the shield walls around the steam generators. One isolation valve is located on either side of the anchor (shield wall). The penetrations through the shield walls are designed as anchors to ensure that one hypothesized pipe rupture will not jeopardize both valves. The major components (i.e., the reactor vessel, steam generators, reactor coolant pumps, and pressurizer) are supported so as to ensure that the severance of a primary coolant pipe does not produce coincident severance of the steam system piping (Section 3.6). Therefore, the containment mechanical penetrations designed for the pipe rupture condition do not consider coincident loads from the loss-of-coolant accident. The pipe capacity in flexure is assumed to be limited to the plastic moment capacity based upon the ultimate strength of the pipe material. For the main steam and feedwater penetrations special reinforcement is required, as shown on Figures 3.8-29 and 3.8-33. This reinforcement provides for transferring shears, torque, and moments into the concrete wall through the liner. Steel elements of the containment and penetrations are designed on the basis of stresses not exceeding yield stress based on using a load factor of 1.0. Concrete elements are designed based upon the ultimate strength design provisions of ACI 318-63.

The piping was designed based on the Code for Pressure Piping ASA B31.1-1955, which was the current standard when the piping was designed. The code was also used to design all piping systems required for safe shutdown under the loss-of-coolant accident conditions.

3.8.1.6 Quality Control and Material Specifications

3.8.1.6.1 Concrete

3.8.1.6.1.1 Ultimate Compressive Strength

The minimum ultimate compressive strength for a standard cylinder of concrete used in the design was as follows:

Containment shell	5000 psi in 28 days.
Other	3000 psi in 28 days.

3.8.1.6.1.2 Quality Control Measures

The specifications for the original structural concrete for Ginna Station required the following quality control measures:

A discussion for the replacement concrete placed during the 1996 Steam Generator Replacement is provided in Section 3.8.1.6.1.6.

Preliminary Tests

The Westinghouse Atomic Power Division obtained the services of a Testing Laboratory which, prior to the contractor commencing concrete work, made preliminary determinations of controlled mixes, using the materials proposed and consistencies suitable for the work, in order to determine the mix proportions necessary to produce concrete conforming to the type and strength requirements called for herein or on the drawings. Aggregates were tested in accordance with the latest editions of the following ASTM Specifications: C29, C40, C12, C128, and C136. Compression tests conformed to ASTM Specifications C39-64 and C192-65. The contractor submitted to the Testing Laboratory, a sufficient time before concrete work commenced, all concrete ingredients required by the Testing Laboratory for the preliminary tests.

The proportions for the concrete mixes were determined by Method 2 of Section 309 of Proposed ACI 301 and as previously specified.

The engineer had the right to make adjustments in concrete proportions if necessary to meet the requirements of the specifications.

In the event the contractor furnished reliable test records of concrete made with materials from the same sources and of the same quality in connection with current work, then all or a part of the strength test specified previously could have been waived by the engineer, subject, however, to any provisions to the contrary of building codes or ordinances of the governing authority.

Field Tests

During concrete operations, the Testing Laboratory had an inspector at the batch plant who certified the mixed proportions of each batch delivered to the site and sampled and tested periodically all concrete ingredients. Another inspector at the construction site inspected

reinforcing and form placements, took slump tests, made test cylinders, checked air content, and recorded weather conditions. Except as noted, test cylinders were molded, cured, capped, and tested in accordance with Proposed ACI 301 except that one of the three cylinders was tested at 3 days and the remaining two at 28 days. For the containment shell, a set of four cylinders was made for each 50 cubic yards or fraction thereof placed in any one day.

One cylinder was tested at 3 days, another cylinder at 7 days, and the remaining two cylinders at 28 days. Slump tests were made at random with a minimum of one test for each 10 cubic yards of concrete placed. Also, slump tests were made on the concrete batch used for test cylinders.

In the event that concrete was poured during freezing weather or when a freeze was expected during the curing period, an additional cylinder was made for each set and was cured under the same conditions as the part of the structure that it represented.

Test Evaluation

The evaluation of test results were in accordance with Chapter 17 of Proposed ACI 301. Sufficient tests were conducted to provide an evaluation of concrete strength in accordance with the specification.

Deficient Concrete

Whenever it appeared that tests of the laboratory cured cylinders failed to meet the requirements set forth in the specification, the engineer and/or Testing Laboratory had the right to:

a. Order changes to the proportions of the mix to increase the strength.
b. Require additional tests of specimens cured entirely under field conditions.
c. Order changes to improve procedures for protecting and curing the concrete.
d. Require additional tests in accordance with "Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," ASTM C42-64.

If these tests failed to prove that the questionable concrete was of the specified quality, the contractor replaced the concrete work as directed.

3.8.1.6.1.3 Concrete Suppliers

Initially, concrete for Ginna Station was supplied from the Penfield Plant of the Manitou Construction Company. This plant was a relatively new "Rex" plant made by Rex Chain Belt Inc. of Milwaukee. Its capacity was about 100 cubic yards per hour. Operation was partially automated and controlled from a central console.

Punched cards were prepared for the various mixes to be supplied. The operator inserted the proper card for the mix required, set a dial for the quantity of concrete desired, and the machine measured out the ingredients automatically. Measurements could be observed on 2-ft diameter indicating dials in the control room as follows:

Cement: 0-6000 lb in 5-lb graduations.

Sand and gravel:	0-30,000 lb in 30-lb graduations.
Water:	0-3000 lb in 3-lb graduations.

The ingredients for the mix could easily be measured and recorded to within 1% of the true values. The State of New York purchased concrete from this plant. Their inspectors made periodic checks and required aggregate measurements within 2% and cement measurements within 1%. All provisions for storage precision of measurement complied with ASTM C94-64, Standard Specifications for Ready-Mixed Concrete.

The bulk of the concrete for the containment was supplied from the Walworth Plant of the Manitou Construction Company.

Technical details of this plant were as follows:

- Rex type AD dry batch plant.
- 100 yards/hr - maximum 150 yards/hr.
- Six-compartment aggregate bin.
- Eight-compartment batcher with dial scale.
- Two-compartment 600 bbl. cement silo.
- Eight-yard cement batcher with dial scale.
- 640-gallon water weight batcher with dial scale.

The plant provided fully automatic batching using a punch card system. All weights as well as time of batch were recorded on the card. Accuracy of the scale was ±0.5%. In a 1-day run, the accumulated weights reconciled to within 5 lb as an average. All recording scales had visual dials which could be observed by the inspector. Moisture probes were embedded in the bins to determine moisture and automatic compensations were made to maintain the proper water-cement ratio. Temperature of the concrete was controlled by heating with closed steam pipes located in the bins or cooling by control of aggregate temperature. Only Type II cement was being stored and used at the Walworth Plant.

3.8.1.6.1.4 Concrete Specifications

The Ginna specification for structural concrete included the Proposed ACI Standard Specifications for Structural Concrete for Buildings, as prepared by ACI Committee 301 and presented in the Journal of the ACI, February 1966, Proceedings, Volume 63, No. 2. At the time the specification was issued, ACI 301-66 was not yet formally released. Nevertheless, ACI 301-66 contained no significant changes from the proposed standard used for the Ginna specifications. The proposed ACI standard was either equaled or exceeded in all cases. Significant requirements that supplement or differ from those in the proposed ACI standard include the following which has been extracted from the Ginna specification:

The structural concrete for the containment shell including the ring girder, cylindrical walls, and dome shall have a minimum ultimate compressive strength of 5000 psi in 28 days.

The determination of the water-cement ratio to attain the required strength shall be in accordance with Method 2, Section 308(b) of Proposed ACI 301.

All cement shall be Portland Cement conforming to "Specification for Portland Cement," ASTM C150-64, Type II ...the cement shall be confined to a single brand with an established reputation for being uniform in character and shall be acceptable to the engineer.

All structural concrete shall be considered subject to potentially destructive exposure and shall contain air in amounts conforming with Table 304(b) of Proposed ACI 301.

A water-reducing densifier shall be added to all structural concrete with a required ultimate compressive strength equal to or greater than 3000 psi at 28 days.

Admixtures containing calcium chloride shall not be used.

Maximum water-cement ratio for various strengths of concrete shall be as follows:

Compressive Strength (psi at 28 days)	Gallons of Water/Sack of Cement
5000	5
3000	6

Ready-mixed concrete shall be mixed and transported in accordance with Specifications for Ready-Mixed Concrete, ASTM C94-65. The minimum amount of mixing in truck mixers loaded to maximum capacity shall be 70 revolutions of the drum or blades after all of the ingredients, including water are in the mixer. The maximum number of revolutions at mixing speed shall be 100; any additional mixing shall be at agitating speed.

The concrete shall be delivered to the site and discharge shall be completed within 1.50 hours or before the turn has been revolved 300 revolutions, whichever comes first, after the introduction of the mixing water to the cement and aggregates or the introduction of the cement to the aggregates. In hot weather the 1.50 hour time limit shall be reduced.

The proportion of water in each strength mix shall be adjusted at least every week as required by the content of surface moisture on the aggregates. Except for this adjustment, no changes in quantity of mixing shall be made without the approval of the engineer.

Each batch of concrete shall be recorded on a ticket which provides the date, actual proportions, concrete design strength, destination as to portion of structure and identification of transit mixer.

Concrete shall be protected against adverse weather conditions in accordance with Recommended Practice for Winter Concreting, new ACI 306-66, and Recommended Practice for Hot Weather Concreting, ACI 605-59, except that accelerators such as calcium chloride and antifreeze compounds shall not be used.

Curing methods detailed in proposed ACI 301 shall be used except that a method other than a curing compound shall be used for initial and final curing of concrete in the containment shell.

For the containment shell, a set of four cylinders will be made for each 50 cubic yards of fraction thereof placed in any one day.

Slump tests will be made at random with a minimum of one test for each 10 cubic yards of concrete placed.

Construction joint surfaces shall be prepared for the placement of concrete thereon by cleaning thoroughly with wire brushes, water under pressure, or other means to remove all coatings, stains, debris, or other foreign material.

The chloride content of mixing water shall not exceed 100 ppm and turbidity shall not exceed 2000 ppm.

On construction joint surfaces in the containment vessel, including all vertical joints in the cylindrical shell and all joints in the dome, an epoxy-resin compound shall be used to bond the new concrete with the abutting pour.

The limitation in Proposed ACI 301 for a maximum slump of 2 in. was not enforced. Enforced slump limitations were as listed in Table 305(a) of Proposed ACI 301.

A listing of all codes and standards referenced in specifications for the containment construction is included in Section 3.8.1.2.5. ACI 301-66 referenced above, provides that:

> The hardened concrete of joints in the exposed work, joints in the middle of beams, girders, joints, and slabs and joints in work designed to contain liquids shall be dampened but not saturated, then thoroughly covered with a coat of neat cement. The mortar shall be as thick as possible on vertical surfaces and at least 1/2-in. thick on horizontal surfaces. The fresh concrete shall be placed before the mortar has attained its initial set.

3.8.1.6.1.5 Admixtures

The ingredients of the structural concrete for the containment include the following admixtures:

a. Air-entraining admixture - This admixture is Darex AREA as manufactured by Grace Construction Materials and is a sulfonated hydrocarbon type with a cement catalyst conforming to ASTM C260.

b. Water-reducing retarder - This admixture is Plastiment as manufactured by Seka Chemical Corporation and is a non-air-entraining, water-reducing retarder with an active ingredient which is a metallic salt of hydroxylated carboxylic acid. This admixture conforms to ASTM C-494, Type D.

No user testing of admixtures was performed.

3.8.1.6.1.6 Replacement Concrete for the 1996 Steam Generator Replacement

Repair of the dome openings following the 1996 Steam Generator Replacement was accomplished using the existing liner plate sections, new reinforcing bars and new concrete. The replacement concrete, its constituents, batching, placement, and testing activities were considered safety-related. Design specifications for "Material Testing Services", "Purchase of Safety-Related Ready-Mixed Concrete" and "Forming, Placing, Finishing and Curing of Safety-Related Concrete" (Bechtel Documents 22225-C-101(Q), 22225-C-311(Q) and 22225-C-302(Q)) controlled the work. Concrete mix designs were developed and tested to comply with the design specification of 5000 psi minimum compressive strength @ 7 days, slump 3" to 6" and air entrainment of 6% ± 1.5%. All mix design constituents were tested to meet design specifications. Independent verification testing was performed in addition to concrete supplier testing required for mix design qualification. B.R. Dewitt Inc. supplied the ready mixed concrete. Provisions for storage of specific mix design quantities of aggregate and cement were made prior to the pour date. The final design mix is listed below:

Constituent	**Weight (per cu. yd.)**
Cement	850 lb
Fly Ash	130 lb
Fine Aggregate[a]	915 lb
Coarse Aggregate[a, b]	1680 lb
Rheobuild 1000[c]	113 oz
MB-VR[c]	19 oz
Water	315 lb

a. Weight is based on saturated, surface dry condition.

b. A 1:1 blend of ASTM C 33 #5 and #7 stone may be used to provide a gradation conforming to #57 stone.

c. Admixture dosage may be adjusted within manufacturer's limits to meet field conditions.

The amount of "superplastizer" or high range water reducing admixture which was required for workable concrete was determined through mock-up testing. A containment dome mockup structure representing the full size actual dome opening with surrounding portions of dome was constructed for opening construction and repair activities. The mix design concrete was placed, cured and tested in the mock-up by the same methodology used on the actual containment prior to the 1996 Steam Generator Replacement outage. The mock-up proved valuable in adjusting admixtures for workability, maintaining truck mixing revolutions within acceptable limits, accessing forming and consolidation techniques, and verifying the mix design parameters.

The mock-up also proved valuable in determining logistical support such as: number of inspectors, technical support from admixture and ready-mix concrete suppliers, pumping controllers, labor support and batch plant communications.

In mid-May of 1996 concrete was placed in both containment dome openings using a Putzmeister BSS 44 series concrete pumper. The dome openings were boarded with reusable forms. Block outs for concrete placement and vibration were provided at approximately 4 ft on centers. After initial set the forms were stripped and the concrete was rubbed out and curing compound was applied. The design strength of the placed concrete was verified with all compressive cylinder breaks exceeding 5000 psi at 7 days.

3.8.1.6.2 Mild Steel Reinforcement

The concrete reinforcement used was deformed bar of intermediate grade billet-steel conforming to the requirements of ASTM A15-64, Specifications for Billet-Steel Bars for Concrete Reinforcement, with deformations conforming to ASTM A305-56T, Deformed Bars for Concrete Reinforcement. Special large size concrete reinforcing bars were deformed bars of intermediate grade billet-steel conforming to ASTM-A408-64, Specifications for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement. Reinforcing steel conforming to these specifications has a tensile strength of 70,000 psi to 90,000 psi and a minimum yield point of 40,000 psi. The large diameter reinforcing bar used in the 1996 Steam Generator Replacement dome opening repair was ASTM A615 which is an equivalent of the original reinforcement. The reinforcing was produced safety-related.

All splicing and anchoring of the concrete reinforcement was in accordance with ACI 318-63. The special large size bars were spliced by the Cadweld process with splices staggered as described below. Exceptions to this splicing process were made in the repair of the 1996 dome openings in limited locations. Where physical constraints prohibited the use of cadwelds (mostly in the hexagon opening corners), #18S reinforcing bars were welded together using a prequalified weld procedure.

The intermediate grade reinforcing steel is the highest ductility steel commonly used for construction. Certified mill reports of chemical and physical tests were submitted to the engineer, Gilbert Associates, Inc., for review and approval. Each bar was branded in the deforming process to carry identification as to the manufacturer, size, type, and yield strength, as shown in the following examples:

- B - Bethlehem.
- 18 - Size 18S.
- N - New billet steel.
- Blank - A-15 and A-408 steel.
- 6 - A-432 60,000 psi yield.
- 7 - A431 75,000 psi yield.

Because of the identification system and because of the large quantity, the material was kept separated in the fabricator's yard. In addition, when loaded for mill shipment, all bars were properly separated and tagged with the manufacturer's identification number.

Visual inspection of the bars was made in the field for inclusions.

The specifications stipulated that "Arc welding concrete reinforcement for any purpose including the achievement of electrical continuity shall not be permitted unless noted otherwise on the drawings."

The concrete cover required for reinforcing steel is tabulated in Table 3.8-7. A comparison is made between values for this plant and ACI requirements.

3.8.1.6.3 Cadwell Splices

Tension splices for bar sizes larger than No. 11 were made with Cadweld splice. To ensure the integrity of the Cadweld splice the quality control procedures provided for a random sampling of splices in the field. The selected splices were removed and tested to destruction. For details of the destructive testing of Cadweld splices, refer to Section 3.8.1.7.1.

Where the special large size bars (i.e., 14S and 18S) were spliced, the Cadweld process was used so that the connection could develop the required minimum ultimate bar strength. Where the Cadweld splice was used, including the cylinder and dome, the splices were staggered a minimum of 3 ft. An exception to this practice was in the vicinity of the large openings. Where reinforcing bars were anchored to plates or shapes, such as is the case for the dome bars anchored into the cylinder and the interrupted hoop bars at penetrations, the Cadweld splices all occur in one plane. In addition to this, the cadweld splices made in 1996 for the Steam Generator Replacement dome opening repair were not staggered. This is typical around the perimeter sides of both dome openings. The dome openings were laid out such that at each side or face of the opening, two out of three layers of the #18S reinforcing bars project into the hole. Lapped splices are detailed in accordance with ACI 318-63.

Where Cadweld splices were used to anchor reinforcing bars to a structural steel member, as shown typically on Figure 3.8-4, a procedure of testing coupons was used to demonstrate that the welding process was under control. This procedure required each welder to initially make coupons, as shown on Figure 3.8-34, as a qualification procedure. The procedure was repeated at a frequency of one coupon for each 100 production units. Each coupon required testing of two Cadweld connections.

In addition, the welding procedure complied with the specifications of the American Welding Society and provided for 100% visual inspection of welds.

A sampling of 20 splices was initially tested to destruction to develop an average ($\bar{X}$) and standard deviation (σ). Thereafter sufficient samples were tested to provide 99% confidence that 95% of the splices met the specification requirements. As additional data became available, the average ($\bar{X}$) and standard deviation (σ) were updated and the quantity of samples revised accordingly.

The distribution established on this basis permitted the development of the lower limit below which no test data should fall. If the result of any test fell below this limit, the subsequent or previous splice was sampled. If this result was above the lower limit, the process was considered to be in control. If this result was again below the lower limit, the process average had changed and an engineering investigation was required to determine the cause of the excess variation and reestablish control.

3.8.1.6.4 Radial Tension Bars

Bars were received by Stressteel Corporation from Bethlehem or U.S. Steel along with certified mill reports of chemical and physical tests. The high-strength alloy steel bars were proof stressed to the minimum specified yield stress of 130,000 psi and then stress relieved in an oven at 700°F for 5 to 6 hours. Chemical test reports on each mill heat of steel used for bars and load-strain curves certifying physical properties of the stress relieved bars were provided. Other bar steel fabricated in the Stressteel plant was of equal or higher strength. Furthermore, the physical appearance of the bar steel, including smooth surfaces and threaded end, completely eliminated possible substitution with other construction materials in the field.

3.8.1.6.5 Containment Liner

3.8.1.6.5.1 Fabrication and Workmanship

The details of the fabrication and workmanship, with certain exceptions, conformed to the requirements of the ASME Nuclear Vessels Code for Class B Vessels. These exceptions included the following:

a. Materials - The steel plate for the main shell including the hemispherical dome, cylindrical walls, and base conformed to ASTM A442, Grade 60, and met the impact test requirements of ASTM A-300, except that the Charpy V-specimens were tested at a temperature of at least 30°F lower than the lowest service metal temperature. For the main liner shell, the lowest service metal temperature was calculated to be 48°F. Rolled sections including test channels and stiffeners conformed to ASTM A36.

b. Weld Inspection - Longitudinal and circumferential welded joints within the main shell, the welded joint connecting the hemispherical dome to the cylinder, and any welded joints within the hemispherical dome were inspected by the liquid penetrant method and spot radiography all in accordance with the ASME Unfired Pressure Vessels Code.

c. Opening Reinforcement - The liner is reinforced about all openings in accordance with ASME Unfired Pressure Vessels Code.

The ASTM A442, Grade 60, material has a specified minimum elongation in 8 in. of 20% and in 2 in. of 23%.

Quality control measures required by these standard specifications included the following:

ASTM A442

One tension test and one bend test shall be made from each plate as rolled. In addition, mill test reports will be obtained for heat.

ASTM A300

Each impact test value shall constitute the average value of three specimens taken from each plate as rolled (Note 3) with not more than one value below the specified minimum value of 15 ft-lb, but in no case below 10 ft-lb. Because of the material thickness, subsize specimens are used thereby altering the above-mentioned impact values to 12.5 and 8.5 ft-lb, respectively.

ASTM A131

Two tension and, except as specified in Paragraph (b), two bend tests shall be made from each heat of structural steel and steel for cold flanging, unless the finished material from a heat is less than 25 short tons when one tension and one bend test will be sufficient. If, however, material from one heat differs 0.15 in. or more in thickness, one tension test and one bend test shall be made from both the thickest, and the thinnest material rolled, regardless of the weight presented. When so specified in the order, a bend test may be taken from each plate of structural steel as rolled. Two tension and two bend tests shall be made from each heat of rivet steel.

When material is ordered for cold flanging and is subject to test and inspection by a ship classification society, one bend test shall be required from each plate as rolled.

3.8.1.6.5.2 Penetrations

The specifications for the containment liner further required that "The materials for penetrations including the personnel and equipment access hatches, as well as the mechanical and electrical penetrations, shall conform with the requirements of the ASME Nuclear Vessels Code for Class B vessels. All materials for penetrations shall exhibit impact properties as required for Class B Vessels."

The material for the penetrations conformed to ASTM A201-61T, Grade B Firebox, Tentative Specification for Carbon-Silicon Steel Plates of Intermediate Tensile Ranges for Fusion-Welded Boilers and Other Pressure Vessels, which was modified to ASTM A300-58, Standard Specification for Steel Plates for Pressure Vessels for Service at Low Temperature.

Quality control measures required for ASTM A201 included the following:

Two tension tests, one bend test, and one homogeneity test shall be made from each firebox steel plate as rolled. One tension test and one bend test shall be made from each flange steel plate as rolled.

3.8.1.6.5.3 Welding

The specifications for the containment liner further required the following quality control measures for welding:

The qualification of welding procedures and welders shall be in accordance with Section IX "Welding Qualifications" of the ASME Boiler and Pressure Vessel Code. Contractor shall submit welding procedures to the engineer for review.

The qualification tests described in Section IX, Part A, include guided bend tests to demonstrate weld ductility. All penetrations shall be examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. Other shop-fabricated components, including the reinforcement about openings, shall be fully radiographed. All nonradiographable joint details shall be examined by the liquid penetrant method.

Full radiography shall be in accordance with the procedures and governed by the acceptability standards of Paragraph N-624 of the ASME Nuclear Vessels Code.

Methods for liquid penetrant examination shall be in accordance with Appendix VIII of the ASME Unfired Pressure Vessels Code.

In order to ensure that the joints in the liner plate and penetrations as well as all weld connections of test channels were leaktight, the specifications for the containment liner required that all welds "shall be examined by detecting leaks at 69 psig test pressure using a soap bubble test or a mixture of air and freon and 100% of detectable leaks arrested." These tests were preliminary to the performance of the initial integrated leak rate test which ensured that the containment leak rate was no greater than 0.1% of the contained volume in 24 hours at 60 psig.

3.8.1.6.5.4 Erection Tolerances

Erection tolerances of the containment liner were:

Overall out-of-roundness	±3 in.
Deviation from round in 10 ft	1-1/2 in. except at seams.
Overall deviation from the plumb line	±3 in.
Deviation from line between tangent points at cylinder to dome transition and base to cylinder transition	±3/4 in.

Shell plate edges to butt for a minimum of 75% of wall thickness

The locations of penetrations with regard to azimuth location to be within ±1/2 in. measured on the circulate section. The horizontal and vertical dimensions associated with the radial dimension shall be ±1/2 in.

During erection, internal wind stiffness temporary braces were added to the liner to maintain roundness tolerances. This bracing was removed after pouring of the wall concrete. The liner erector’s adherence to the tolerances specified for the liner were checked by means of a control survey.

3.8.1.6.5.5 Painting

The containment liner was painted as follows:

a. All interior surfaces of the cylinder and dome (i.e., all exposed surfaces including the wall behind the insulation panels) had a minimum of a 2.5-mil coat of Carbozinc #11 Gray, as manufactured by the Carboline Company.

b. All other surfaces except the underside of the base liner had a minimum of a 1.5-mil coat of paint conforming with Federal Specification TT-P-645A, Primer, Zinc Chromate Alkyd.

3.8.1.6.6 Elastomer Pads

The elastomer pads used for the containment number 320 and were manufactured to the following dimensions:

A. Plan area: 42 in. by 9 in.

B. Neoprene: two layers of neoprene each 11/16-in. thick.

C. Steel shims: an outer shim on each face with a minimum thickness of 16 gauge and one shim between the two neoprene layers of 10 gauge.

The neoprene has a nominal durameter hardness of 55. Physical requirements of the neoprene are shown in Table 3.8-8.

3.8.1.6.7 Tendons

3.8.1.6.7.1 Materials

The prestressing system used for the containment is the BBRV system utilizing ninety 0.25-in. diameter wires. The wires are high tensile steel, that is, bright, cold-drawn, and stress-relieved conforming to ASTM A421-59T, Type BA, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete, with a minimum guaranteed ultimate strength of 240,000 psi. The BBRV system uses parallel wires with cold formed buttonheads at the ends which bear upon a perforated steel anchor head, thus providing a mechanical means for transferring the prestress force. The buttonheads are formed by cold upsetting to a nominal diameter of 3/8 in. on the 1/4-in. diameter wire. The materials used for anchorage components were as follows:

Item	Size	Material
Movable anchor head	7-7/8 in. O.D. x 3-1/2 in.	C1141 heat treated
Fixed anchor head	5-1/8 in. O.D. x 3-3/4 in.	C1141 heat treated
Bushing (adaptor for couplers)	7-7/8 in. O.D. x 5-1/8 in. I.D.	C1045
Couplers	10-1/2 in. O.D. x 7-1/8 in. I.D.	C1018
Bearing plate	18-1/2 in. O.D. x 2-1/2 in.	A36
Split shims	8-1/2 in. O.D. x 1-1/2 in. wall	HFSM Tube C1026

The C1141 material is heat treated to Rockwell C30 to C33.

The material used for the exposed bearing plates at the upper end of the vertical tendons conformed to ASTM A36, Specification for Structural Steel, including the optional requirement of this specification of silicon killed fine grain practice for steel used at temperatures where improved notch toughness is important.

3.8.1.6.7.2 Tests and Inspection

All anchorage hardware was 100% visually inspected to ensure that no surface flaws, notches, and similar stress raisers existed. Hardness tests were performed on each anchor head to verify adequate heat treatment and strength. The tendon fabricator cut coupons from each end of each reel of wire, formed buttonheads, and tested the specimens. These tests were to ensure that the wire would rupture before failure of the buttonhead and that the wire would meet the physical requirements of ASTM A421. Coupons and the coils they represented not meeting the requirements were rejected. Records were maintained for each coupon test and for the tendons in which each coil of wire was used. Anchorage components were fabricated from materials specified on the manufacturer's parts drawings. Requirements for machining, tolerances, and heat treating were as specified on the parts drawings.

All buttonheads were visually inspected and a minimum of 10% of the buttonheads were randomly checked for size verification. Dimensions of the buttonheads were as follows:

a. Diameter equal to or greater than 0.372 in. and equal to or less than 0.388 in.

b. Length equal to or greater than 0.252 in. and equal to or less than 2.272 in.

c. A bearing surface on all sides.

Limitations on splits (cracks) in buttonheads were as follows:

aa. Splits are not to be inclined more than 45 degrees to the axis of the wire.

bb. Sum of the widths of all splits are less than 0.06 in. with inclinations less than 20 degrees to the axis of the wire.

cc. No more than two splits occur in buttonheads which have splits inclined more than 20 degrees but less than 45 degrees to the axis of the wire. In no event do the two cracks occur in the same place.

3.8.1.6.8 Liner Insulation

The inside surface of the liner may be inspected in the wall and dome area. However, the walls are covered by panels of thermal insulation to protect the liner in the event of an accident. Corrosion of the liner is not expected because the outside surface is in contact with concrete; the lower portion of the inside surface is protected from sweating by the insulation; and the entire liner is tied into the overall cathodic protection system. It is possible, however, to remove a section of insulation periodically to examine the liner if required.

The liner insulation is 1.25-in. thick Vinylcel, which is a rigid cross-linked polyvinyl chloride (PVC) foam plastic manufactured by Johns-Manville. Dimensions for full size sheets are 44 in. x 84 in. Sheet faces are finished with 0.019-in. thick sheets of type 304 stainless steel.

The sheets are attached to the steel liner with stainless steel studs (KSM #304 stainless #10-24). The full size sheets have six studs each. A 1.125-in. diameter neoprene backed stainless steel combination washer is placed outside the sheet over the stud and held in place by a self-locking stainless steel hexagonal head nut. Backs of the sheets are routed to fit over the test channels on the liner. Sheets are erected with the 44-in. dimension vertical and vertical joints are staggered. The joints at the base of the routed edges are taped with 3/8-in. wide tape and the routed area is filled with Dow Corning Sealant #780 silicone rubber base sealant or equivalent to make a flush finished joint.

At penetrations or other irregular surfaces, the sheets are cut to fit and the edges are beveled and caulked with the sealant. A similar caulked joint is provided at the extremities of the insulated area.

If for any reason a panel or section must be removed, it is possible to do so by cutting along the joints and removing the fastening nuts. Replacement would only involve reapplication of nuts and new sealant.

The PVC material is chemically compatible with steel and no degradation of either material because of contact and/or environment results. The sealant is an acid-free inorganic type; again, no chemical reaction results. The sealant is waterproof and remains pliant down to 80°F and does not soften up to 350°F.

The reports of tests performed to ensure meeting the functional requirements are included in Section 3.8.1.7 and Appendix 3E.

3.8.1.7 Testing and Inservice Inspection Requirements

<u>3.8.1.7.1 Construction Phase Testing</u>

Preoperational inspections and tests were performed in several stages which finally led to the structural proof and integrated leak rate tests. Inspections and tests of the structural elements of the containment vessel included the liner, tendons, concrete and concrete reinforcement, elastomer pads, and rock anchors.

3.8.1.7.1.1 Liner

Longitudinal and circumferential welded joints within the main shell, the welded joint connecting the dome to the cylinder, and all joints within the dome were inspected by the liquid penetrant method and spot radiography. All penetrations including the equipment access door and the personnel locks were examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. All other shop-fabricated components including the reinforcement about openings were fully radiographed. All other joint details were examined by the liquid penetrant method. Full radiography was performed in accordance with the procedures and governed by the acceptability standards of Paragraph N-624 of the ASME Nuclear Vessels Code. Spot radiography was performed in accordance with the procedures and governed by the standards of Paragraph UW-52 of the ASME Unfired Pressure Vessels Code. Methods of liquid penetrant examination were in accordance with Appendix VIII of the ASME Unfired Pressure Vessels Code. All piping penetrations and personnel locks were pressure tested in the fabricator’s shop to demonstrate leaktightness and structural integrity.

A prototype of the air-cooled penetrations was tested to verify thermal and hydraulic design calculations.

All accessible weld seams on the liner were spot radiographed, except for penetrations which were fully radiographed. Spot radiography was performed in accordance with Section UW-52 of the ASME Unfired Pressure Vessels Code, which required that:

> One spot shall be examined in the first 50 ft of welding in each vessel and one spot shall be examined for each additional 50 ft of welding or fraction thereof. Such additional spots as may be required shall be selected so that any examination is made of the welding of each welding operator or welder. The minimum length of spot radiograph shall be 6 in.

The liner weld seams were also examined by pressurizing the test channels to design pressure (60 psig) with a mixture of air and freon, and checking all seams with a halogen leak detector. All detectable leaks were corrected by repairing the weld and retesting.

3.8.1.7.1.2 Prestressing Tendons

The rock anchors and wall tendons for the containment were inspected by both the supplier, Joseph T. Ryerson and Son, Inc., and the prime contractor, Westinghouse Atomic Power Division.

Ryerson performed all tests enumerated in Section 3.8.1.6, and reports are retained in the Quality Control file.

Westinghouse did the following:

a. Submitted certified mill test reports to the designer, Gilbert Associates, Inc., for their review and comment.
b. Monitored the shop procedures and inspection by Ryerson.
c. Inspected each tendon at the Ryerson shop before shipment to ensure conformance to specifications and proper preparation for shipment.

In addition to the foregoing, a test was performed on each item of anchorage hardware to confirm that it was capable of developing the ultimate capacity of the tendon. Reports of these tests are included in Appendix 3D.

3.8.1.7.1.3 Concrete Reinforcement

Tension splices for bar sizes larger than No. 11 were made with the Cadweld splice designed to develop the ultimate strength of the bar, or with the use of deformed bars conforming to ASTM A408-64, Intermediate Grade (minimum tensile stress of 70,000 psi). A sampling of 20 splices was initially tested to destruction to develop an average ($\bar{X}$) and standard deviation (σ). Sufficient samples were tested to provide a 99% confidence level that 95% of the splices would meet the specification requirements. The average of all tests also was required to remain above the minimum tensile strength. As additional data became available, the average and standard deviations were updated. The actual frequency of testing carried out was one specimen for each 25 splices made for each crew for the first 250 splices made by that crew and one test for each 100 splices thereafter. In addition, where deformed bars were attached

to structural steel members, specimens were made and tested to ensure that the weld of the splice to the member did not fail before the rebar or the splice. The frequency of testing these specimens was the same as that for the normal splices. A plot of the results of all tests over a period of time is shown in Figure 3.8-35.

No arc welding was permitted on the Class I structures for splicing reinforcing bars during the original construction. All rebar splices of the major reinforcement in the containment structure (i.e., special large size bars) were made with the Cadweld process. There were no special requirements for chemical composition of reinforcing bars beyond the requirements of ASTM A15 and A408. Generally, no tack welding of reinforcing bars was permitted. The only exception involved those locations specifically shown on the drawings (refer to Figure 3.8-4) which were located where rebar strength was not required and bars were provided solely to provide electrical continuity below ground water level.

In sampling the Cadweld splices a test was concurrently performed on the rebar. Where the rebar failed prior to the splice, a check was provided on the ultimate strength of the rebar, thus providing a check on conformance with the manufacturer's certifications and the ASTM standards. In addition, certified mill test reports were received from the rebar supplier and checked for conformance with specification requirements. The splice and mill test reports are retained in the Quality Control file.

Replacement reinforcement for the dome openings constructed in the 1996 Steam Generator Replacement was #18S ASTM A615 Grade 60. The reinforcing bars were connected primarily with T-series Grade 60 Cadweld splices as manufactured by Erico Products. Prior to starting production splicing, a member of each splicing crew was qualified for performing cadwelds in each of three positions; horizontal, vertical and diagonal. During production, a specified number of sister splices were made in-place next to production splices, under the same conditions, and by the same crew. For each crew the following tensile tests on the sister splices were made:

A. Test one sister splice for the first 10 production splices.

B. Test four sister splices for the next 90 production splices.

C. Test three sister splices for the next and subsequent units of 100 splices.

The cadweld sister splices were tested to failure. All splices were determined to be capable of developing cadweld design criteria of 1.25 times the minimum yield strength of the replacement reinforcement which was 60,000 psi. The limited number of welded splices were performed using a prequalified arc welding procedure and visually inspected in accordance with AWS D.1.4.

3.8.1.7.1.4 Concrete

The prime contractor obtained the services of a testing agency which made preliminary determinations of controlled mixes, using the materials proposed and consistencies suitable for the work, in order to determine the mix proportions necessary to produce conformance to the type and strength requirements. During concrete operations, the testing agency maintained an inspector at the batch plant who certified the mixed proportions of each batch delivered to the

site and sampled and tested periodically all concrete ingredients and monitored aggregate surface moisture. One or more inspectors were retained at the construction site to take slump tests, make test cylinders, check air content, and record weather conditions. For the reactor containment, a set of no less than four cylinders was made for each 50 cubic yards or fraction thereof placed in any day. Two cylinders each were tested in 7 days and in 28 days. Slump tests were made at random, with a minimum of one test for each 10 cubic yards of concrete placed. Also, slump tests were made on the concrete batch used for test cylinders. A running average of test results through September 26, 1967, for 5000 psi concrete is shown in Figure 3.8-36.

Acceptance standards for compressive strength were based upon ACI 301, Section 1703 which stated that: "Strengths of ultimate strength type concrete and prestressed concrete shall be considered satisfactory if the average of any three consecutive strength tests of the laboratory cured specimens representing each specified strength of concrete is equal to or greater than the specified strength, and if not more than 10% of the strength tests have values less than the specified strength."

Acceptance standards for slump were based upon those limits stated in ACI 301, Table 304(a) which established a maximum slump of 3 in. for reinforced and plain footings, caissons, and substructure walls; 4 in. for slabs, beams, reinforced walls, and building columns; and also established a minimum slump of 1 in.

Figure 3.8-36 provides a moving average of compressive strength for 5000 psi concrete on five previous test groups. There were two periods of time when these averages fell below the specified 5000 psi, 28-day compressive strength. The occasions when this occurred involved the use of the first mix in areas requiring by design only 3000 psi concrete, namely the containment base slab and the turbine pedestal. The mix was then modified to produce the more satisfactory results thereafter reflected on the chart of the running average. At no time did in-place concrete fail to meet the specification requirements.

Type II cement, modified for low heat of hydration, was used to minimize shrinkage.

Grab samples were taken periodically at the batch plant, upon delivery of cement. Each sample was tested by the testing laboratory for conformance to ASTM C150, and the results were also compared with the certificate supplied with each delivery of cement.

3.8.1.7.1.5 Elastomer Bearing Pads

Tests were performed on elastomer specimens to ensure compliance with requirements for: (1) original physical properties including tear resistance, hardness, tensile strength, and ultimate elongation; (2) change in physical properties due to overaging; (3) extreme temperature characteristics; (4) ozone cracking resistance; (5) oil swell, and (6) shear modulus. In addition, two full size pads were tested, one for creep and one for ultimate load. Specimen No. 1 was initially placed under essentially a constant compressive load of 1000 psi (the design pressure) for 4 days to measure creep. This pad was then loaded up to 2000 kips (5.3 times design load) when the test was terminated without failure. Specimen No. 2 was similarly loaded up to 2000 kips without failure. The rebound of the pads after the 2000-kips load was

removed was essentially complete. A summary of the test results is shown in Figures 3.8-37 and 3.8-38.

3.8.1.7.1.6 Rock Anchor Tests

Three scaled-down test rock anchors were installed to demonstrate the holddown capacity of the rock and the capacity of the bond between rock and grout.

Two tests were made on rock anchor A, which was installed at the center of the proposed containment. The first test, called test A-1, was to determine rock hold-down capacity. The set-up for test A-1 is illustrated in Figure 3.8-39. The beam support piers were located beyond the assumed influence circle of rock having a diameter of 23 ft 6 in. An independent frame was erected to obtain deflection measurements on the concrete pier at the anchor. This placed all supports for lifting as well as measuring devices outside the influence circle of rock. Dial gauges were used to measure the movement of the concrete pier and the anchor head. The test load was applied with a 150-ton jack mounted on the beams spanning the test anchor.
Measurements of the jacking force were made with a dynamometer, calibrated immediately before the test. The second test on rock anchor A (test A-2) and the tests on rock anchors B and C, also installed near the center of the proposed containment, were made to demonstrate bond capacity. The set-up for test A-2 and for rock anchors B and C was an arrangement whereby the jack was supported directly by the concrete pier adjacent to the test anchor.

Rock anchor A consisted of twenty-eight 0.25-in. diameter wires grouted for a length of 4 ft 5.5 in. in a 3.5-in. diameter hole. All test rock anchors were oversized so that the test load of 100 kips would develop only about 30% of the ultimate capacity of tendon wires while developing a bond stress of 170 psi, which is the design stress for the containment rock anchors. This permitted testing bond stresses well in excess of design (170 psi) without exceeding ultimate wire stresses. The test procedure for test A-1 is described in the following paragraph.

The anchor was loaded in 20,000-lb increments to 100,000 lb. The load was maintained at each increment for 15 min prior to taking measurements for elongation of the tendon and elevations of the concrete pedestal and adjacent rock surface. Because the anchor head appeared from visual observation not to have lifted off at the 100,000-lb load, the load was increased to 110,000 lb, at which point lift-off was apparent. Subsequent review of measurements on the movement of the anchor head indicate that actual lift-off occurred between 80,000 lb and 100,000 lb, as would be expected.

In test A-2 and the tests on rock anchors B and C, the tendon was jacked from the concrete pier immediately adjacent to the tendon.

Table 3.8-9 lists measurements taken during test A-1. Figures 3.8-40 through 3.8-42 show plots of load versus elongation deflection for all tests.

The application of a test load of 110 kips to rock anchor A (as indicated by the results of test A-1 shown on Figure 3.8-40) is equivalent to 137.5% of the calculated hold-down capacity assumption used in the design. The plot of load versus elongation deflection for rock anchor A tests A-2 (see Figure 3.8-40) and B and C (see Figures 3.8-41 and 3.8-42) indicate a factor of safety against slippage by the grout and rock of at least 2.0 (200-kips load versus 100-kips

design load) for rock anchor B. If slippage occurred within the grout, the factor of safety against failure is even greater. The plot of load versus elongation for rock anchor A shows an apparent discontinuity which is indicated by a dashed line on Figure 3.8-40. This represents settlement of the concrete pier adjacent to the rock anchor when the load was transferred from the lifting frame used in test A-1 to the lock nut that bore on the concrete pier.

3.8.1.7.1.7 Large Opening Reinforcements

Testing of large opening reinforcements is discussed in Appendix 3B.

3.8.1.7.1.8 Liner Insulation

Tests were conducted on the Vinylcel for confirmation of the following material properties:

- Conductivity factor (Btu/hr ft^2/°F/in.), per ASTM C177-63, at 75°F, 100°F, 150°F.
- Compressive yield strength (psi), per ASTM C165.
- Moisture vapor permeability (per inch) by dry cup, per ASTM C355-64.
- Shear strength (psi).
- Shear modulus (psi), per ASTM C273-61.
- Compressive modulus (psi), per ASTM C165-54.
- Density (lb/ft^3), per ASTM D16 22-63.
- Average coefficient of linear expansion (in./in./°F) for temperature range.

Results of these tests are included in Appendix 3E. Also included are the results of a test to determine resistance to flame exposure, plus the results of an analog simulation of the insulation system due to the pressure and temperature transients associated with the 50% overpressure condition.

3.8.1.7.2 General Description of the Structural Integrity Test

3.8.1.7.2.1 Pressurization

After completion of the entire containment, a structural integrity air pressure test at 115% of design pressure was maintained for 1 hour.

The pressurization of the containment was done at 5 psi increments. Readings and measurements were taken at 35 psig, 50 psig, 60 psig, and the final test pressure of 69 psig. Except for the final pressure level, the vessel pressure was always increased 1 psi above the level at which measurements were made. The pressure was then reduced to the specified value and observations made after a delay of at least 10 min to permit an adjustment of strains within the structure.

Because the structure is so large, displacement measurements (absolute or relative) could be made with precision and could be used as confirmation of the previously calculated response. The test program further included a visual examination of the containment during pressurization to observe deformations and to demonstrate that no distortions occurred of a

significantly greater magnitude than those calculated in advance based upon the same analytical models used for the design of all structural elements for the loading combinations described in Section 3.8.1.2.

Prior to the test, a table of predicted strain, deflection, and rotation values was developed for an internal pressure of 69 psig, which was the pressure of the structural proof test, as well as those lower pressure levels used to take measurements. Strain, displacement, and rotation predicted from the analytical model for an internal pressure of 69 psig were used as a basis for verifying satisfactory structural response. Although strain gauges were installed on designated areas of the liner, concrete reinforcement, and tendon shims, the analytically derived strains were not used as acceptance figures for the actual values. The obtained values were analyzed and evaluated to determine magnitude and direction of principal strains. If the test data included any displacements which were in excess of the predicted extremes, such discrepancies required resolution including review of the design, evaluation of measurement errors and material variability and, conceivably, exploration of the structure. Prior to the test, maximum anticipated crack widths were predicted. If any crack widths occurring during the test were in excess of predicted values, such discrepancies were required to be satisfactorily resolved in a similar manner as for displacements. The anticipated values for crack widths and a complete report on other anticipated measurements were provided before the test.

3.8.1.7.2.2 Measurements

During the test at each specified pressure level, a series of measurements and observations were made as follows:

a. Radial displacements of the cylinder at three elevations and at three azimuths in order to ascertain if the response was symmetrical and to verify the estimated response due to average circumferential membrane stresses. On the same three azimuths, horizontal displacements were measured immediately above and below the dome to cylinder transition.

b. Vertical displacement of the cylinder at the top relative to the base ring girder at three azimuths to determine the vertical elongation of the side wall and average tendon strains.

c. Cylinder base rotation and displacement at three azimuths to verify hinge action and symmetrical response.

d. Horizontal and vertical displacements of the reinforcing ring around the equipment access hatch opening.

e. Strain of reinforcing bars near the concrete surface around the equipment access opening. Small access ports to selected reinforcing bars were left in the concrete to mount strain gauges just prior to the structural test. These gauges were provided only in those places where this limited exposure of the steel reinforcement would not be injurious to the behavior of the structure under test. Following completion of the structural test the access ports were sealed.

f. The liner was instrumented with electrical resistance strain gauges in the region of several typical penetrations as well as a region unaffected by geometric discontinuities. Redundancy in strain readings were accomplished by placing strain gauge rosettes at

several points about the penetration openings and by instrumenting four penetrations which were subjected to similar loadings and restraints.

g. To determine principal stresses, in magnitude and direction, the gauges employed were in the form of 120-degree rosettes. Associated with the gauges was the application of a strain-indicating brittle lacquer to qualitatively augment the local values indicated by the gauges and to show the existence of a symmetrical, or otherwise, overall stress pattern.

h. Horizontal displacements were measured immediately above and below the dome to the cylinder discontinuity. Strain gauges were installed on reinforcing bars near the exposed concrete surface above and below the discontinuity. Detailed concrete crack observations were made in the immediate vicinity of the discontinuity.

i. Load cells were used on four tendons at the top anchorages to verify the stress variation over the range of test pressures. Also, strain measurements were made on a limited number of bearing plates at the top anchorages.

In addition to displacement and strain data, observation for cracks in the concrete was made in the following manner:

aa. The containment was visually inspected for cracks and crack patterns.

bb. At selected locations, the surface was white-washed for detailed measurements of spacing and width of cracks to verify that local strains were not excessive. These selected locations included:

1. Quadrant of reinforcing ring for large opening.
2. Cylinder to dome transition.
3. The cylinder, where circumferential membrane stresses are maximum and where flexural stresses are maximum.

The movable (top) anchor heads of the sidewall tendons were inspected for wires which had failed. A ruptured wire would be readily evident because the energy release upon rupture causes the wire to noticeably rise and remain loose.

The maximum calculated radial displacement due to the test pressure of the cylinder was 0.62 in., and a minimum radial displacement calculated at the hinge (base of cylinder) was 0.06 in. Local variation in geometry of the structure made it extremely doubtful that uniform and predictable strain measurements would be achieved from the strain gauges installed on designated areas of the liner, concrete reinforcement, and tendon shims. Therefore, specific strain measurements could not be reasonably established as acceptance standards.

The program for instrumentation of the containment structure was established to permit installing the instruments immediately before the test, thereby precluding the necessity of providing unusual protection against construction abuse and weather. Shielding enclosures were provided on those external surfaces of the containment vessel where strain gauges were to be located.

Instrumentation for making displacement measurements included dial gauges, scales, and theodolites used to read prepositioned targets. All gauges and targets were installed immediately prior to the test.

All measuring devices, including theodolites and dial gauges, produced measurements of sufficient precision to ascertain satisfactory structural response. For a theodolite located approximately 150 ft from the targets, it was possible to measure within 0.01 in. For a maximum expected measurement of radial deflection of 0.62 in., a precision of 0.02 in. (twice the expected measuring accuracy) should be satisfactory. Dial gauges used at the hinge detail could measure to the nearest 0.001 in. which was sufficient to define the displacement and rotation of the hinge. Where it was practical to use dial gauges for greater accuracy, they were used to make displacement measurements.

3.8.1.7.2.3 Test Pressure Justification

The 115% design pressure used in the structural proof test was justified for the following reasons:

a. The principal tensile stress in the liner during a simultaneous loss-of-coolant accident (60 psig pressure) and 0.08g earthquake amounts to 19.9 ksi assuming the liner participates fully in taking earthquake shears.

   The tensile stress in the liner under the 69 psig test for structural integrity is 26.5 ksi. This means that before the leak rate test at 60 psig the liner has been subjected to tensile stresses in excess of those which would occur during a simultaneous loss-of-coolant accident and 0.08g earthquake. During the leak rate test the tensile stress in the liner is 23 ksi. During a loss-of-coolant accident, without earthquake, the tensile stress is 19.2 ksi.

b. The principal tensile stress in the outer circumferential reinforcement band during a loss-of-coolant accident and simultaneous 0.08g earthquake is 26.4 ksi. The principal tensile stress in this reinforcement during test for structural integrity is 26.5 ksi.

c. The average stress in a tendon during a loss-of-coolant accident is 145.2 ksi, the average stress in a tendon during tests for structural integrity is 145.5 ksi.

d. The test pressure conforms with the recommendations of Oak Ridge National Laboratory regarding testing of concrete vessels (Reference: ORNL - NSIC - 5, Volume II U.S. Reactor Containment Technology, page 10.8).

3.8.1.7.2.4 Test Results

See Section 14.6.1.6.10 for the results of the preoperational structural integrity test of the containment.

3.8.1.7.2.5 Containment Return to Service Testing Post 1996 Steam Generator Replacement

After placement, curing and acceptance of the 1996 Steam Generator Replacement dome opening repair concrete, the structure underwent a full pressure Integrated Leak Rate Test (ILRT) and a partial Structural Integrity Test (SIT). These tests were combined to satisfy the requirements of 10CFR50 Appendix J, “Primary Reactor Containment Leakage Testing for

Water-Cooled Power Reactors," and to demonstrate that the containment design and dome opening repairs are adequate to withstand postulated pressure loads. The containment interior and exterior were structurally inspected for cracks and anomalies prior to pressurization and after depressurization. Embedded strain gages were installed on the replacement rebar and monitored throughout the testing. The ILRT test pressure was 60 psig. This test was performed and accepted prior to increasing the pressure for the SIT. The original SIT pressure was 69 psig which represented 115% of the design pressure. A test pressure of 72 psig was used in 1996 which supports a potential increase in the design pressure to 62 psig.

The repaired dome openings and adjacent areas were monitored during the SIT. Crack mapping was performed in these areas prior to, at pressurization, and after depressurization. Vertical growth of the structure was monitored at the spring line and the dome apex. Radial growth measurements were taken at defined elevations at three azimuth locations. Predicted rebar strains, design vertical and radial displacements, and crack size and length criteria were used as the test acceptance criteria.

<u>3.8.1.7.3 Postoperational Surveillance</u>

3.8.1.7.3.1 Leakage Monitoring

Postoperational leakage rate testing is discussed in Section 6.2.6 and in Section 14.6.1.6.9.

3.8.1.7.3.2 Initial Tendon Surveillance Program

Means are provided to allow surveillance of all upper tendon terminations. The initial tendon surveillance program incorporated the following:

a. Visual inspection of all tendon terminations was made after the structural integrity test. A record was kept of all broken wires.

b. A number of tendons equally spaced around the containment were to be inspected 6 months, 1 year, 3 years, and 10 years after the structural test. If more than 1% of additional wires were found broken, additional equally spaced tendons were to be inspected until it was established that less than 1% of all wires inspected were broken.

c. A prestress confirmation lift-off test is made on the tendons referred to in item 2 above, to compare relaxation of tendons with a predicted curve. Tests were to be conducted 6 months, 1 year, 3 years, and 10 years after tensioning. This phase of the program provides for obtaining a lift-off reading by using a hydraulic jack to just lift the upper anchor head off the shim. This procedure provides a determination of the stress level in the tendons and also is used to confirm previously predicated stress losses including steel relaxation and concrete creep. Before reseating the tendon, the hydraulic jack is used to lift the termination sufficiently to apply an additional stress in the wires equal to that applied during pressurization of the shell (6%) to verify its ability to withstand additional stresses applied during accident conditions.

d. Each of 40 tendons includes an extra unstressed 0.25-in. diameter wire specimen, obtained from a reel represented in the tendon. The specimen extends from the top anchor head down to approximately elevation 240 ft. One wire is removed on an annual basis for examination. This provides a periodic check on tendon corrosion.

The initial structural integrity test of the containment was conducted at 69 psi. Displacement measurements were recorded during this test for pressures of 35, 50, 60, and 69 psig. The continuing structural integrity of the containment is verified by the tendon surveillance program and displacement measurements taken during subsequent leak rate tests. General agreement with initial measurements indicates a structural response similar to the initial tests. This, plus the tendon surveillance program, establishes a high degree of assurance that the integrity of containment has been maintained.

The initial 10-year tendon surveillance program has been completed as follows:

Prestressing of rock anchors	Fall 1966
Prestressing of tendons	March-April 1969
Structural integrity test	April 1969
6-month inservice inspection	October 1969
1-year inservice inspection	May 1970
3-year inservice inspection	May 1972
8-year inservice inspection	June 1977
10-year inservice inspection	October 1979
Retensioning of tendons - new time zero	June 1980

In June 1980, retensioning of 137 out of the total of 160 tendons was done. The 23 tendons that were not included in the retensioning program had been retensioned in May 1969, approximately 1000 hours after their original stressing.

3.8.1.7.3.3 Current Tendon Surveillance Program

The current tendon surveillance program includes the following:

a. Commencing with the new time zero, June 1980, an inspection for the presence of broken wires and prestress lift-off tests are to be conducted after 1 year, 3 years, and 5 years and every 5 years thereafter. The 1-year inspection was conducted in July 1981, the 3-year inspection was conducted during July and November 1983, and the 5-year inspection was conducted in August 1985. Inspections continue every 5 years (with a 25% extension allowed per Technical Specifications).

b. Fourteen tendons, equally spaced around the containment are to be inspected for the presence of broken wires. The acceptance criteria for the inspection are that no more than a total of 38 wires in 14 tendons are broken and that not more than five broken wires exist in any one tendon. If more than 38 broken wires are found, all tendons are to be inspected. However, if more than 20 wires in 14 tendons have been broken since the last inspection, all tendons are to be inspected. If inspection reveals more than 5% of the total wires broken, the containment must be declared inoperable.

If more than five broken wires are found in any one tendon, four immediately adjacent tendons (two on each side of the tendon containing more than five broken wires) are to be inspected. The acceptance criterion then will be no more than four broken wires in any of the additional four tendons. If this criterion is not satisfied, all of the tendons are to be inspected and if more than 5% of the total wires are broken, the containment must be declared inoperable.

c. Prestress confirmation lift-off tests are to be performed on the 14 tendons identified in item b. above.

   The lift-off readings are obtained in the same manner as described above for the initial tendon surveillance program. Before reseating a tendon, additional stress (6%) will be imposed to verify the ability of the tendon to sustain the added stress applied during accident conditions. If the average stress in the 14 tendons is less than 144,000 psi (60% of ultimate stress) equivalent to 636 kips, all tendons are to be tested for prestress and retensioned, if necessary, to a stress of 144,000 psi (636 kips). If a tendon fails its lift-off test lower limit of 636 kips, the two adjacent tendons are tested. If either adjacent tendon fails its lift-off test, a NRC report is required due to possible abnormal degradation of containment. If both adjacent tendons pass their test and no more tendons fail their test, the single tendon failure is considered unique and acceptable.

d. One unstressed wire specimen is removed during each surveillance for examination for corrosion as in the initial tendon surveillance program. The wire is also tensile tested. Failure of the wire below its ultimate strength identifies an unacceptable wire and requires NRC notification.

e. A visual inspection of the top anchorage assembly hardware for the 14 tendons identified in item b. above is also performed. The surrounding concrete is also inspected during integrated leak rate tests when the containment is at its maximum test pressure. Finally, the filler grease for the 14 tendons is inspected and tested. If significant deterioration of any tendon anchorage assembly, local concrete, or filler grease is observed, NRC notification is required.

f. If NRC notification is required, the report should include a description of the tendon condition, the condition of the concrete (especially at tendon anchorages), the inspection procedure, the tolerances on concrete cracking, and the measures being implemented if the tolerances are exceeded.

3.8.1.7.3.4 Current Tendon Surveillance Program Results

The 3-year surveillance of the containment vessel tendons performed after retensioning was during July and November 1983. A representative sample of 18 tendons was selected. The results following the surveillance are documented in the containment vessel tendon surveillance report submitted to the NRC by *Reference 24* and the conclusions are summarized as follows:

a. The results of the completed tendon surveillance, in which 18 sample tendons were lift-off tested, indicated that the forces in the tendons are maintained at the levels expected, and that no abnormal force losses have occurred. The agreement between the actual and

predicted tendon forces is better than that which is generally experienced on other containments.

b. Based on the forces measured in the sample tendons, the average force level of the tendons in the containment is 711 kips, which exceeds the minimum required value of 636 kips appearing in the Tendon Surveillance Program by 11.8%.

c. Based on the results of the 1983 surveillance, a recommendation was made for future surveillances that the predicted tendon force calculations be based on a 40-year wire relaxation of 16%, applicable to all tendons, and multiplied by factors to account for the retensioning effect.

d. From the results of the surveillance and a comparison of actual stress relaxation with that predicted, no future retensioning of tendons should be required for the remainder of the expected plant life.

In the safety evaluation report based on the results of the 1981 and 1983 lift-off tests, the NRC concluded that it appears that the tendon forces are stable and that there are no abnormal tendon force losses; and that the adequacy and integrity of the containment is ensured.(*Reference 52*).

The 5-year surveillances of the containment vessel tendons were performed in August 1985, August 1990, October 1995, and December 2000. The results of the August 1985 surveillance are documented in a report submitted to the NRC by *Reference 53*. It has been concluded that the surveillance program methodology provides an effective means of monitoring tendon forces and that the results of the surveillances confirm the structural adequacy of the containment vessel. Future surveillances will be conducted at 5-year intervals in accordance with the Tendon Surveillance Program. The 1990, 1995, and 2000 surveillance tests showed that the required tendon prestress continues to meet all design requirements. As part of the test program, a sacrificial tendon wire is extracted, examined, and tested during each surveillance. The wires extracted show no evidence of corrosion and test out to its specified yield and ultimate strengths. The grease that surrounds the tendon was analyzed using methods consistent with Regulatory Guide 1.35, Revision 2, and showed no evidence of water or unacceptable levels of chlorides, nitrates, or sulfides.

3.8.1.7.3.5 Test on Rock Anchors

In the June 1980 retensioning, 137 of the 160 tendons were stressed to at least 0.735 ultimate stress. This force had to be resisted by the rock anchors. Consequently, the tendon retensioning also constitutes a test of the rock anchor. The elongations of the wall tendon, measured at its upper anchor head, are a combination of (1) the wall tendon strains times the tendon length, plus (2) the movement, if any, of the upper anchor head of the rock anchor. The measured elongations agreed closely with those predicted based solely on the wall tendon strains. These results indicate that the rock anchors developed a force of 0.735 ultimate stress with no perceptible slippage or movement of their upper anchor head.

3.8.1.7.3.6 Inservice Inspection

The Nuclear Regulatory Commission issued an amendment to 10 CFR 50.55a, Codes and Standards, on August 8, 1996, that required the implementation of the 1992 Edition with the

1992 Addenda of ASME Section XI Code, Subsections IWE, IWL and applicable IWA requirements with limitations, modifications and supplemental requirements as described within the rulemaking. These requirements became effective on September 9, 1996 and are identified within the Containment Program and the Containment Repair and Replacement Program in the Inservice Inspection (ISI) Program document. Later Editions and Addenda of ASME Section XI Code may be used as specified within 10 CFR 50.55a that are identified within the Inservice Inspection (ISI) Program document. The second and later 10-year interval requirements are identified within the Containment Inservice Inspection (CISI) Plan.

3.8.2 STRUCTURAL REANALYSIS PROGRAM

3.8.2.1 Design Codes, Criteria, and Load Combinations - SEP Topic III-7.B

<u>3.8.2.1.1 Introduction</u>

The Franklin Research Center, under contract to the NRC, compared the structural design codes and loading criteria used in the R. E. Ginna Nuclear Power Plant design against the corresponding codes and criteria currently used for licensing of new plants at the time of the Systematic Evaluation Program (*Reference 25*). The objective of the code comparison review was to identify deviations in design criteria from current criteria and to assess the effect of these deviations on margins of safety.

3.8.2.1.1.1 Seismic Category I Structures

Franklin Research Center, for purposes of the review, considered the following to be Seismic Category I structures.

<u>Containment.</u>

- Cylindrical wall, dome, and slab.
- Liner (no credit for structural strength under mechanical loads).
- Equipment hatch.
- Personnel locks.

<u>Internal structures.</u>

- Steam generator/reactor coolant pump compartments (reviewed in Generic Task A-2).
- Biological shield (reviewed in Generic Task A-2).
- Fuel transfer canal.

<u>External structures.</u>

a. Auxiliary building.
    - Spent fuel storage pool.
    - New fuel storage area.
    - Portions of the fuel transfer tube.
    - Seismic Category I equipment.

i. Safety injection pumps and residual heat removal pumps (in pit beneath basement floor).

ii. Refueling water storage tank (RWST).

iii. Boric acid storage tanks.

iv. Containment spray pumps.

v. Waste holdup tanks.

vi. 480-V switchgear.

b. Control building.

- Control room.
- Battery room.
- Relay room.

c. Portions of the intermediate building (which house auxiliary feedwater pumps).

d. Cable tunnel.

e. Intake/discharge structure and screen house (service water (SW)) portion only.

f. Diesel-generator annex.

Major structures not classified as Seismic Category I are the turbine building and the service building.

3.8.2.1.1.2 Structural Codes

The structural codes governing design of the major Seismic Category I structures for the Ginna Nuclear Power Plant were as follows:

Structure	Design Criteria	Current Criteria
CONTAINMENT		
Concrete (including shell, dome, and slab)	ACI 318-63	ASME B&PV Code, Section III, Division 2, 1980 (subtitled ACI 359-80)
	ACI 301-63 (specifications for concrete)	ACI 301-72 (Revision 1975)
Liner	ASME B&PV Section III, 1965 (Provisions of Article 4[a])	ASME B&PV Code, Section III, Division 2, 1980 (Subtitled ACI 359-80)

Structure	Design Criteria	Current Criteria
	ASME B&PV Section VIII (undated), (Fabrication Practices for Welded Vessels Only) ASME B&PV Section IX (undated), (welding procedure and welders qualifications only)	
Personnel locks and equipment hatches	ACI 318-63 for concrete ASME B&PV Section III, 1965, for steel	ASME B&PV Code Section III, Division 2, 1980 (subtitled ACI 359-80)
AUXILIARY BUILDING	AISC-1963 ACI 318-63	AISC-1980 ACI 349-80
CONTROL ROOM BUILDING	AISC-1963 ACI 318-63	AISC-1980 ACI 349-80
PORTIONS OF THE INTERMEDIATE BUILD-ING	AISC-1963 ACI 318-63	AISC-1980 ACI 349-80
CABLE TUNNEL	ACI 318-63	ACI 349-80
INTAKE/DISCHARGE STRUCTURE AND SCREEN HOUSE	AISC-1963 ACI 318-63	AISC-1980 ACI 349-80
DIESEL-GENERATOR ANNEX	AISC-1963 ACI 318-63	AISC-1980 ACI 349-80

a. The two significant applications for this article are (1) determination of thermal stresses in the liner and (2) analysis of pipe penetration attached to liner.

3.8.2.1.1.3 Code Comparison

The current and older (Ginna design) codes were 6/501 compared paragraph by paragraph to determine what effects the code changes could have on the load carrying capacity of individual structural members. Appendix 3F is a summary of the code comparison findings. Those code changes judged by Franklin Research Center to have the potential to significantly degrade margins of safety are listed in Tables 3.8-10 through 3.8-14. Table 3.8-15 lists the structural elements for which a potential existed for margins of safety to be less than that originally computed because of load criteria changes since plant design and construction. Rochester Gas and Electric was requested by the NRC to review all Seismic Category I structures at Ginna Station to determine if the structural elements listed in Table 3.8-15 occur in the designs, and for those that occur, to assess the actual impact of the associated code changes on margins of safety.(*Reference 26*) The results of this assessment were reported in *References 27* and *28* and are summarized in Section 3.8.2.1.2.

<u>3.8.2.1.2 Assessment of Design Codes and Load Changes for Concrete Structures</u>

The concrete structural elements identified by Franklin Research Center as being potentially affected by concrete design code changes and by any associated load or load combination changes were evaluated and the results were as follows (*References 26* and *28*).

3.8.2.1.2.1 Columns With Spliced Reinforcing

ACI 349-76, Section 7.10.3, specifies requirements for columns with spliced reinforcing which did not exist in the ACI 318-63 Code. The ACI 349-76 Code requires that splices in each face of a column, where the design load stress in the longitudinal bars varies from f_y in compression to 1/2 f_y in tension, be developed to provide at least twice the calculated tension in that face of the column (splices in combination with unspliced bars can provide this if applicable). This code change requires that a minimum of 1/4 of the yield capacity of the bars in each face of the column be developed by both spliced and unspliced bars in that face of the column.

To assess the impact of this change on Ginna Station, concrete outline drawings, reinforcing fabrication drawings, and available original calculations were reviewed to determine to what extent columns with spliced reinforcing exist. As a result of these reviews, a total of 57 columns with spliced reinforcing was found. They occur in the auxiliary building (14), control building (1), diesel-generator building (6), intermediate building (20), and screen house (16). All of the columns found use lap splices which occur at the bottom of the columns.

To evaluate the columns in the auxiliary building, control building, diesel-generator building, and intermediate building, they were divided into groups according to their reinforcing details and size. This grouping resulted in the formation of nine groups of similar columns. The column within each group judged to have the most severe load from the applicable loads and load combinations was chosen for evaluation. Additionally, one column from the screen house was chosen for evaluation. These columns were evaluated for compliance with ACI 349-76 provisions. The capacity of the spliced reinforcing was calculated in accordance with

the code and this capacity was used with the worst-case load combination to determine if the code-required factor of safety was met. If the splices did not have the minimum required splice length to fully develop the bar in accordance with ACI 349-76, the splice capacities were reduced by a factor of L_p/L_d (where L_p is the splice length provided and L_d is the ACI 349-76 required splice length).

The results of the evaluation found that all concrete columns evaluated meet and/or exceed the code-required factor of safety.

3.8.2.1.2.2 Brackets and Corbels (Not on the Containment Shell)

ACI 318-63 did not have any specific requirements for brackets and corbels. Provisions for these components are included in ACI 349-76, Section 11.13. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

Concrete outline drawings and available original calculations were reviewed to determine if brackets and corbels were used at Ginna. A total of 12 corbels was found during these reviews. They occur in the auxiliary building (4), intermediate building (3), and containment interior structures (5). Seven of these corbels support primary structural elements (e.g., beams, slabs). The remaining five corbels support secondary elements (e.g., a corbel on the auxiliary building exterior walls which supports a 4-in. architectural brick facing) which generally cause no significant load on the corbel.

Corbels having similar geometry and reinforcing details were grouped together, and the corbel from each group judged to have the worst load was evaluated. If this corbel was acceptable, then the others in the group were judged acceptable. The selected corbels were first evaluated for compliance with ACI 349-76 requirements for minimum and maximum reinforcing, geometry constraints, and placement of reinforcing. If all of these requirements were met, the capacity of the corbel was calculated in accordance with ACI 349-76. This capacity was used, along with the load from the worst-case load combination, to determine if the code-required factor of safety was met. If a corbel did not conform to the above requirements, then the shear stresses in the concrete imparted by the loads on the corbel were compared to the code permissible shear stress for unreinforced concrete (even though there actually was some reinforcing in the corbel). If the actual stress was less than that permitted, the corbel was judged acceptable.

The results of the evaluation of the twelve corbels were:

a. Six of the seven corbels supporting primary structural elements meet the code requirements for reinforcing, geometry, and factor of safety. The remaining corbel does not conform to the code requirements for minimum reinforcing, but the stresses in this corbel are small and the corbel was judged to have an acceptable margin of safety.

b. The five corbels which support secondary elements do not comply with the code requirements for reinforcing. However, all of these corbels have loads which produce insignificant stresses in the corbels and are therefore judged to have an acceptable margin of safety.

3.8.2.1.2.3 Elements Loaded in Shear With No Diagonal Tension (Shear Friction)

The provisions for shear friction given in ACI 349-76 did not exist in ACI 318-63. These provisions specify reinforcing and stress requirements for situations where it is inappropriate to consider shear as a measure of diagonal tension.

Concrete outline drawings and available original calculations were reviewed to determine if conditions requiring evaluation for shear friction exist at Ginna. As a result of this review, a total of 203 shear-friction conditions was found. They occur in the auxiliary building (12), containment interior structures (133), and screen house (58). These conditions exist for embedded plates supporting steel beams, concrete ledges, removable concrete slabs, beam pockets, and several miscellaneous situations.

To evaluate these conditions found in the auxiliary building and containment interior structures, they were divided into a number of groups by similarity, considering their geometry and reinforcing details. This approach resulted in the formation of 15 groups. The condition in each group judged to have the most severe load from the applicable loads and load combinations was evaluated for compliance with the code provisions. Two conditions in the screen house were also evaluated for compliance with the code provisions.

The controlling conditions were first evaluated by determining their shear friction capacity utilizing only those details strictly conforming to the code. No credit was taken for other reinforcing installed which did not meet ACI 349-76 provisions. This capacity was then compared to the controlling factored load combination to see if the code-required factor of safety was met. If the factor of safety was not satisfied, several alternative evaluation approaches were used to assess safety, and these are described below along with a summary of all results.

The results of the evaluations for this code change indicate the following for the 15 groups in the auxiliary building and containment interior structures evaluated:

a. Six groups representing 26 conditions have safety factors that are equal to or greater than the code-required factor of safety, considering only code-satisfying reinforcing.

b. Five groups representing 108 conditions have safety factors that are equal to or greater than the code-required factor of safety, considering code-satisfying reinforcing plus taking credit for any additional well-anchored reinforcing installed.

c. Two groups representing three conditions have factors of safety that are equal to or greater than the code-required factor of safety for shear stresses in unreinforced concrete. These elements had small loads and the capacities were checked ignoring any reinforcing present in the design.

d. One group representing six conditions (beam pockets for beams supporting the intermediate building floor at column line N) have an actual factor of safety less than the code-required factor of safety (considering appropriate load factors) but greater than unity against ultimate failure (with all load factors reduced to 1.0).

e. One group representing two conditions (thrust blocks at the base of each reactor coolant pump) meets the code-required factor of safety assuming an in-situ concrete strength (f'c) of 3300 psi, as opposed to the 28-day strength of 3000 psi. This in-situ strength is judged to

be reasonable based upon typical concrete compressive strength increases over long time periods.

The results of the evaluation for this code change in the screen house show the safety factors are greater than those required by the code considering only code-satisfying reinforcing.

3.8.2.1.2.4 Structural Walls - Primary Load Carrying

Shear walls.

ACI 349-76, Sections 11.15.1 through 11.15.6, specifies requirements for reinforcing and permissible shear stresses for in-plane shear loads on walls.

The ACI 318-63 Code had no specific requirements for in-plane shear on shear walls.

Concrete outline drawings and available original calculations were reviewed to determine if shear walls exist at Ginna. All walls which connect a roof or floor to a lower floor were considered to act as shear walls. As a result of the drawing and calculation review, a total of 187 shear walls was identified. They were found in the auxiliary building (87), intermediate building (1), control building (3), diesel-generator building (16), containment interior structures (59), and screen house (21).

To evaluate the shear walls in the auxiliary building, control building, intermediate building, diesel-generator building, and containment interior structures, the walls in each building were considered as a separate group. Each group of walls was further broken down by classifying each wall as either an interior or exterior wall. One wall judged to be representative of each classification within the group was then evaluated. If these representative walls were found to be acceptable, then the other walls within their classification were judged acceptable. A wall was evaluated by first determining the controlling load combination for the wall, and then determining the in-plane vertical, in-plane horizontal, and lateral loads on the wall. Using these loads, the walls were evaluated using the code provisions. Vertical and lateral loads on the walls were evaluated in addition to in-plane horizontal loads because they directly influence the requirements for reinforcing in the walls. The shear walls in the screen house were qualitatively evaluated by comparison to the auxiliary building.

The results of this evaluation are as follows:

a. The shear walls in the auxiliary building, intermediate building, control building, containment interior structures, and screen house meet the code requirements.
b. The shear walls in the diesel-generator building do not meet the current code requirements for in-plane loads or flexural bending from lateral loads. (This was reevaluated and is being upgraded as part of the Ginna Station Structural Upgrade Program.)

Punching shear.

ACI 349-76, Section 11.15.7, specifies permissible punching shear stresses for walls. ACI 318-63 had no specific provisions for walls for these stresses. Punching loads are caused by relatively concentrated lateral loads on the walls. These loads may be from pipe supports,

equipment supports, duct supports, conduit supports, or any other component producing a lateral load on a wall.

Concrete outline drawings, available original calculations, and pipe support drawings and load sheets from the Ginna Piping Seismic Upgrade Program were reviewed to determine where punching loads occur and what the magnitude of these loads are. As a result of this review, both pipe and equipment support loads were judged to cause the most severe punching loads.

To evaluate the walls for equipment punching loads, the loads found from the above review were applied to the walls considering the specific details of each design. To evaluate the loads from pipe supports, since there are so many supports, the most severe loads found were applied to the thinnest wall found, conservatively using a 6-in.^{2} area of application. These loads were used, along with the capacity of the wall calculated in accordance with the ACI 349-76 provisions, to determine if the code-required factor of safety was met.

As a result of the above evaluations, it was found that the walls, in all cases, meet the code-required factor of safety for punching shear.

3.8.2.1.2.5 Elements Subject to Temperature Variations

ACI 349-76, Appendix A, specifies requirements for consideration of temperature variations in concrete which were not contained in ACI 318-63. These new provisions require that the effects of the gradient temperature distribution and the difference between mean temperature distribution and base temperature during MODES 1 and 2 or accident conditions be considered. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

Concrete outline drawings and pertinent calculations (in buildings where a possible thermal differential condition of any consequence could occur) were reviewed to determine the extent of possible thermal differential conditions in restrained concrete elements.

A total of six possible conditions/elements was found during this review. These conditions occurred in the containment interior structures (5) and in the cable tunnel (1). Based on restraint and degree of thermal differential, the cable tunnel condition was judged to be the worst case and was therefore evaluated to determine the effect on the factor of safety. The conditions for the containment interior structures are less severe because the temperature differential is less and the temperature would tend to dissipate and equalize.

The evaluation determined the moments in the cable tunnel, using the worst loading combination. The actual factor of safety was determined by dividing the theoretical moment capacity of the concrete section by the applied moments due to the loads imposed. This actual factor of safety was then compared to the ACI 349-76 required factor of safety.

The actual factor of safety for the cable tunnel was greater than the code-required factor of safety. Because the cable tunnel was considered the "worst-case" condition for the thermal differential requirement, the remaining five elements were judged to meet the current code requirements of ACI 349-76, Appendix A, for thermal loads.

3.8.2.1.2.6 Areas of Containment Shell Subject to Peripheral Shear

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code (B&PV Code), Section III, Division 2, 1980. The provisions for peripheral (punching) shear appear in code Section CC-3421.6. These provisions are similar to the ACI 318-63 Code provisions for slabs and footings, except that the allowable punching shear stress in CC-3421.6 includes the effect of shell membrane stresses. For membrane tension, the allowable concrete punching shear stress in the ASME code is less than that allowed by ACI 318-63.

Significant shell punching shear loads can occur at shell penetrations. To evaluate the impact of the code change, all penetrations found from a review of the containment shell concrete drawings were documented. As a result of this review, 126 penetrations, including two large access openings, were identified. Since the punching shear capacity of the shell at penetrations was expected to be closely related to penetration size, the penetrations were grouped by penetration sleeve diameter. The nominal penetration sleeve diameters range from 6 in. to 54 in. and the two large access openings are 9 ft 6 in. and 14 ft 0 in. A total of 10 groups of penetrations was defined in this manner.

All penetrations were found to be provided with a circumferential ring arrangement to allow transfer of the punching shear load directly to the concrete. The effect of the peripheral shear code change was evaluated by examining the shell capacity of the penetrations for current code adequacy. Where simple calculations or judgment showed that a penetration group is clearly adequate, the need for assessment was eliminated. For those groups that were assessed, a "worst-case" penetration from each group was chosen and the shell capacity for those penetrations was evaluated. Actual factors of safety were calculated and compared to the factor of safety required by the code. When the shell capacity for the "worst-case" penetration in a group was found adequate, the capacity of the other penetrations in the group was judged adequate.

The results of the evaluations are as follows:

a. For penetration groups with 6-in., 12.50-in., and 14.25-in. diameter sleeves, shell capacity was found adequate by calculations. For these penetrations, the code-specified punching shear capacity of the concrete exceeds the ultimate axial load of the pipe penetration. This axial load is the maximum that the process pipe is capable of developing based on its tensile strength.

b. For penetration groups with 24-in. and 54-in. diameter sleeves, the shell capacity was judged to be adequate. No significant punching shear loads were identified, and an evaluation was not considered necessary.

c. At the large access (equipment and personnel) openings (one group), significant punching shear loads occur due to containment internal pressure only. Adequacy against punching failure local to the penetration under the abnormal loading condition (90 psig internal pressure, which is 1.5 P_a) was demonstrated by calculations.

d. For the groups with 10-in. and 24.25-in. diameter sleeves, the shell capacity was shown adequate. The calculated punching shear loads for the "worst case" penetrations are well below the code-specified punching shear capacity of the concrete. Pipe break loads were

used for the evaluation and were obtained by conservatively using a factor of 2.0 times the pipe operating pressure times the pipe area. This method is consistent with current industry practice.

e. For the 29-in. and 45.25-in. diameter sleeve groups (feedwater and main steam penetrations), the shell was found not to meet the current code-required factor of safety when using pipe rupture loads from the original plant design calculations. However, the actual factor of safety is greater than 1.0, thereby providing a margin of safety against ultimate failure.

3.8.2.1.2.7 Areas of Containment Shell Subject to Torsion

Concrete containment design is currently governed by the ASME B&PV Code, Section III, Division 2, 1980. Section CC-3421.7 of the code contains provisions for the allowable torsional shear stress in the concrete. Such provisions were not contained in the ACI 318-63 Code. The present allowable torsional shear stress includes the effects of the membrane stresses in the containment shell and is based on a criterion that limits the principal membrane tension stress in the concrete.

Only two types of penetrations, the main steam and feedwater, are provided with torsion resisting elements which rely upon the concrete capacity. In both cases, redundant elements are provided. The penetration sleeves have lugs welded to them, which could resist torsional loads and impart torsional shear stresses to the concrete. However, the final design noted in the original calculations shows that the tie rods incorporated into the penetration details were adequately designed to resist torsion. These tie rods do not rely upon the torsional shear capacity of the concrete, and, therefore, a torsional shear stress check was not required.

3.8.2.1.2.8 Brackets and Corbels (On the Containment Shell)

The ACI 318-63 Code did not specify requirements for brackets and corbels. Provisions for these components are included in the ASME B&PV Code, Section III, Division 2, Section CC-3421.8. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

Concrete outline drawings and original calculations for the containment shell were reviewed to determine if brackets and corbels were used in its design. As a result of the review, no brackets or corbels were found on the containment shell. Therefore, no further evaluation was required.

3.8.2.1.2.9 Areas of Containment Shell Subject to Biaxial Tension

Increased tensile development lengths are required for reinforcing steel bars terminated in biaxial tensile areas of reinforced-concrete containment structures in accordance with Section CC-3532.1.2 of the ASME B&PV Code, Section III, Division 2, 1980. For biaxial tension loading, bar development lengths, including both straight embedment lengths and equivalent straight lengths for standard hooks, are required to be increased by 25% over the standard development lengths required for uniaxial loading. Nominal temperature reinforcement is

excluded from these special provisions. ACI 318-63 had no requirements related to this increase in development length.

Containment shell concrete outline drawings were examined to identify the areas where the main reinforcing bars are terminated with either straight development lengths or standard hooks. Special attention was paid to such areas as penetrations, where bars are likely to be terminated. The drawing review revealed nine areas where the main reinforcing bars in the wall and dome are terminated.

These cases involve vertical reinforcement in the wall and meridional bars in the dome above the ring girder. Main horizontal wall bars were found to be terminated using positive mechanical anchorage devices (such as Cadwelds and structural steel shapes) that are capable of transferring forces to other reinforcement. Typically, main horizontal and vertical bars terminated at penetrations are anchored using these positive mechanical anchorages. However, the drawing review revealed seven additional areas where supplementary bars are terminated at penetrations.

Thirteen of the 16 areas were evaluated individually by first determining the location of the critical section to be evaluated and then comparing the tensile development lengths required for the controlling load combination to the development lengths provided. The remaining three areas are similar to three of the areas evaluated, and individual evaluation was not considered warranted. In all of the 13 areas evaluated, the provided tensile development lengths exceeded ASME Code requirements. In several of the areas investigated, bars were actually terminated outside of the biaxial tensile stress area (i.e., in compressive areas which are excluded from these special requirements). As a result of this evaluation, it is concluded that the code change did not reduce the containment shell margin of safety.

3.8.2.1.2.10 Steel Embedments Transmitting Loads to Concrete

Appendix B to ACI 349-76 is a new appendix which specifies new requirements for stress analysis of steel embedments used to transmit loads from attachments into the reinforced-concrete structure. The only area of concern of this change was the integrity of the containment dome liner and studs under pressure and temperature loads that are caused by the loss-of-coolant accident and steam line break loading conditions. An evaluation of the integrity of the liner and studs was conducted by Gilbert Commonwealth for RG&E and submitted to the NRC by *Reference 29*. The conclusions were that although some failures of studs could possibly occur, these would be at the shank of the studs and thus, no tearing of the liner would occur. Details of this analysis are provided in Section 3.8.2.3.

3.8.2.1.3 Assessment of Design Codes and Load Changes for Steel Structures

Rochester Gas and Electric reported on the results of the evaluation of steel code and load changes by *Reference 28*. Seismic loadings for steel structures were not specifically analyzed because RG&E considered that the main structural steel elements were determined suitable by the Lawrence Livermore Laboratory Analysis documented in NUREG/CR-1821, Seismic Review of the Robert E. Ginna Nuclear Power Plant, which was approved by the NRC by *Reference 30*. The steel code changes concerning coped beams, moment connections, and

steel embedments were evaluated relative to the seismic loads and load combinations in conjunction with the Structural Upgrade Program.

The evaluation of code changes and new load changes was performed for all eight major findings of the AISC 1963 versus AISC 1980 Code comparison and the one major finding of ACI 318-63 versus ACI 349-80 Code comparison. The evaluations were for loads and load combinations involving normal and operating-basis earthquake loads. Safe shutdown earthquake loads were generally addressed in NUREG/CR-1821 with exceptions noted above. Tornado loads were addressed in the Ginna Structural Upgrade Program. The results were as follows:

3.8.2.1.3.1 Shear Connectors in Composite Beams

The code change that required this evaluation involved new requirements added in the AISC 1980 Code, Subsection 1.11.4, as compared with AISC 1963 Code, Subsection 1.11.4. The code change affects the distribution, diameter, and spacing of shear connectors in composite beams.

The approach used for this evaluation was to review the calculations and the construction drawings for the use of shear connectors for composite beams.

The results of the above review showed no use of shear connectors for composite design on the plant structures reviewed, and therefore, no change to the margin of safety.

3.8.2.1.3.2 Composite Beams With Steel Deck

This evaluation is required due to the addition of a new Subsection 1.11.5 to the AISC 1980 Code. The code addition defines requirements for composite beams where a formed steel deck is used for support of the concrete slab.

The approach used for this evaluation was to review the calculations and the construction drawings for composite beams with steel decking.

The results of the review determined that the main beams and girders on the turbine building operating floor elevation 289 ft 6 in. and located between all columns, had shear connectors attached to the top flange. The concrete slab was supported by steel decking.

Selected beams were analyzed for the loads shown on the drawings. The results of the analysis showed that composite design was not required for these beams and it is surmised that the shear connectors were added to provide lateral support for the top flange. Therefore, the code change has no effect on the margin of safety.

3.8.2.1.3.3 Hybrid Girders

This evaluation was required due to the addition of a new requirement by the AISC 1980 Code to Subsection 1.10.6 which did not appear in the AISC 1963 Code. This new requirement limits the maximum stress in the flange of a hybrid girder.

The approach used for this evaluation was to review the construction drawings and specifications for the existence of hybrid girders.

The results of the review showed no use of hybrid girders on the plant structures. Therefore, this code change does not affect the margin of safety.

3.8.2.1.3.4 Compression Elements

This evaluation is based on a revision to Subsection 1.9.1 of the AISC 1963 Code by new provisions in Subsection 1.9.1.2 and Appendix C of the AISC 1980 Code.

These new provisions revise the approach for designing certain unstiffened compression elements which exceed the width to thickness ratios prescribed in the codes.

From the results of case study 10 in the Franklin Research Center report (*Reference 25*), it was concluded that only T-sections in compression need to be reviewed as the AISC 1963 Code is more conservative for other members in compression.

The approach used for this evaluation was to review the members in the structural model of the plant to determine where T-sections were used and if they were subject to compression, under the normal operating load combinations evaluated in this report.

The results of the computer output review showed none of the T-sections failing the code check for normal load combinations with the member in compression.

It was therefore concluded that for normal load combinations the margin of safety for members affected by this code change is still acceptable.

3.8.2.1.3.5 Tension Members

This evaluation was necessary because of a new requirement in the AISC 1980 Code added in Subsection 1.14.2.2.

This code addition defines the requirements for the design of axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross section of the member.

A generic review of the two codes was performed to compare a design example using the formulas and allowables for each. The results showed that the AISC 1963 Code provided a more conservative design.

It was therefore concluded that this code change does not decrease the margin of safety.

3.8.2.1.3.6 Coped Beams

A new requirement was added in the AISC 1980 Code requiring that beam end connections, where the top flange is coped, be checked for a tearing failure, "block shear capacity", along a plane through the fasteners.

The method used to evaluate this code change was to completely review all steel fabrication drawings for major members with bolted connections and coped top flanges. Girts, platform steel, stair stringers, and miscellaneous steel were not included as these members are lightly loaded and shear is not a concern.

The drawing review turned up 452 coped beams with 335 different erection marks. From this total a random selection of 55 beams was statistically chosen for evaluation of the code change effects.

The evaluation consisted of calculating the block shear capacity of each of the beams selected and comparing this capacity against either the loads shown on the construction drawings, the shear capacity of the connection bolts, or the reaction based on the maximum allowable load for the beam span.

In all cases the block shear capacity was higher than these other controlling reactions.

It was therefore concluded that, using a statistical approach at a 95% confidence level, no more than 5% of the population of coped beams may have capacities controlled by this code change. (Safe shutdown earthquake checks were conducted as part of the Ginna Station Structural Upgrade Program.)

3.8.2.1.3.7 Moment Connections

A new requirement was added in the AISC 1980 Code in Subsections 1.15.5.2, 1.15.5.3, and 1.15.5.4. These subsections define the requirements for column web stiffeners where moment connected members frame into columns.

The construction and fabrication drawings were thoroughly reviewed for the use of moment type connections. This survey found that only some roof beams in the screen house were designed and detailed as moment connections.

These connections were then checked against the AISC 1980 Code and it was determined that, based on the member sizes, details, and original applied loads, no column web stiffeners are required.

It was therefore concluded that for the load combination reviewed the code change does not affect the margin of safety for the structures reviewed. (Safe shutdown earthquake checks were conducted as part of the Ginna Station Structural Upgrade Program.)

3.8.2.1.3.8 Lateral Bracing

The AISC 1963 Code, Section 2.8, has been revised by AISC 1980 Code, Section 2.9. This code change revises the formulas for determining the maximum spacing for lateral supports of members designed using plastic design methods.

This code change was evaluated by a review of the existing available calculations and the original FSAR. No evidence was found of plastic design methods being used.

It was therefore concluded that this code change does not affect the margin of safety for the structures reviewed.

3.8.2.1.3.9 Steel Embedments

This code change involves the use of the ACI 349-80 Code, Appendix B, for the design of steel embedments in concrete structures. The ACI 318-63 Code used in the original design

did not specifically address the design of steel embedments. It was up to the individual designer to provide an embedment which satisfied the allowable stresses in the code. Working stress design was the method used for determining loads and stresses.

The latest ACI Code requires the use of ultimate strength design which includes the use of factored loads and larger allowable stresses. This difference alone would make direct comparison of the margins of safety difficult.

There are many other differences in the methods and details that the designer would use for a given embedment and a given code, but the main difference is the requirement of ACI 349-80 Code, Appendix B, that the anchorage design be controlled by the ultimate strength of the embedment steel. Concrete strength of the anchorage must not control no matter what actual loads are applied to the anchorage. Unless the designers were fully cognizant of the requirements of ACI 349 during the actual design it is unlikely that all anchorages would satisfy this code requirement, since it allows only a ductile (steel) failure of the anchorage irrespective of the calculated or actual applied loads.

Due to these difficulties in direct comparison of the two codes it was decided to statistically select a random number of anchorages for evaluation against the ACI 349-80 Code.

From a total population of 194 columns, 51 columns were selected for evaluation. Of the 51 columns selected (*Reference 46*) had anchorage into concrete.

The approach taken for this evaluation was to analyze the column anchorage to determine if it met the ductile failure and other requirements, including minimum edge distances, embedment depth, anchor size, etc. of the ACI 349-80 Code. If the code requirements were met, it was concluded that the margin of safety for the anchorage is acceptable.

If the requirements were not met then the ultimate concrete capacity of the anchorage or the allowable steel capacity whichever was less, using the ACI 349-80 Code as the basis, was compared to the applied factored loads. Only normal design loads using current load combinations were used in the comparison. If the concrete or steel capacity, whichever controls, was still greater than the applied loads the anchorage was deemed to have an acceptable margin of safety.

The results of the evaluation for this code change are as follows:

a. Of the 46 column anchorages evaluated, a total of 22 did not meet the ACI 349-80 Code.
b. Of the 22 that did not meet the code, a total of five anchorages was unacceptable for the applied loads.

The result of this design code evaluation, using a statistical projection, is that at a 95% confidence level, no more than 21% of the population of 194 column anchorages would have unacceptable margins of safety for normal load combinations. (The issue of anchorages for normal, safe shutdown earthquake, and tornado loads was reviewed under the Ginna Station Structural Upgrade Program.)

3.8.2.1.4 Summary

RG&E defined all applicable loads and load combinations considered limiting for the concrete and steel safety-related structures at Ginna Station (*Reference 27*). The NRC staff concluded that these loads and load combinations were acceptable in *Reference 31*. The evaluation of Ginna structures for design code and load changes showed that for tornado-related loadings, all required safety-related structures were either able to meet currently required factors of safety, were shown to meet margin-to-failure criteria through detailed calculations or were provided with additional reinforcement as part of the Structural Upgrade Program. For seismic loadings, it was determined that all concrete code changes were acceptable, except for the shear walls in the diesel-generator buildings, coped beams, moment connections, and steel embedments. These were further evaluated and resolved as necessary in conjunction with the Structural Upgrade Program.

3.8.2.2 Structural Reevaluation of Containment

3.8.2.2.1 Introduction

The containment structure was reviewed as part of the SEP. The Lawrence Livermore National Laboratory performed a seismic review of Ginna Station for the NRC. This review included the containment and other structures and the results were reported in *Reference 30*. The Lawrence Livermore National Laboratory performed a further evaluation (structural review) of the capacity of the containment to withstand combined loss-of-coolant and safe-shutdown earthquake loads. The results of this evaluation were reported in *Reference 32*. For this latter evaluation, seismic loads were developed by scaling the loads developed previously in the SEP program for the 0.2g peak ground acceleration safe shutdown earthquake to 0.17g, which is consistent with the site specific ground response spectra developed by Lawrence Livermore National Laboratory (Section 2.5.2.2). Thermal and pressure loads were developed from pressure and temperature transients developed by Lawrence Livermore National Laboratory for the loss-of-coolant accident conditions.

An axisymmetric, multilayer shell of revolution analytical model was developed for the containment. The model included the concrete vertical wall and dome and the steel liner. Appropriate boundary conditions representing the shell-to-base-slab interface through neoprene pads were included. Since the base slab is founded on rock and the presence of the neoprene pads essentially isolates the base slab from the containment vessel, the base slab was not included in the model. No details, such as hatches or other penetrations, were evaluated.

New seismic, thermal, and pressure loads were developed for the Ginna containment structure as part of the SEP. New seismic loads and the adequacy of the structure to withstand the seismic loads alone were reported in *Reference 30*. New temperature and pressure time-histories were developed by Lawrence Livermore National Laboratory.(*Reference 33*) The normal operating loads, peak pressure loads, and the thermal loads corresponding to the peak pressure conditions, peak thermal loads and the pressure loads corresponding to peak thermal conditions, and seismic loads were combined. This implies that the safe shutdown earthquake occurs approximately 2 minutes after a loss-of-coolant accident. This is considered extremely unlikely and, therefore, the assumed load combination is considered very conservative.

3.8.2.2.2 Containment Temperature

The normal operating temperatures assumed for the Ginna evaluation correspond to a typical "cold day." Ambient temperature inside the containment is 110°F and the outside temperature is 2°F. This condition was selected as the operating condition in that thermal gradients and thermal stresses were expected to be most severe for a cold day. The assumed operating conditions were also the initial conditions for calculating the thermal gradients through the shell. Figure 3.8-43 shows the transient time-history of the containment temperature used in the analysis. This temperature transient has been shown to be much more severe than the predicted actual postaccident temperatures that may occur (see Section 6.2.1). A maximum temperature of approximately 421° F is indicated approximately 34 sec after the start of the transient. However, the internal temperature decreases to less than 300°F at approximately 91 sec which is the time the peak pressure occurs. The rate of change of temperature compared to the resonant frequencies of the containment was such that the temperature loads could be considered as equivalent static loads.

3.8.2.2.3 Containment Pressure

Containment pressure corresponding to the accident condition was developed by Lawrence Livermore National Laboratory (*Reference 33*). The time-history pressure variation within the containment is shown in Figure 3.8-44. This pressure transient is much more severe than the predicted worst-case conditions inside the containment following a loss-of-coolant accident or a steam line break (see Section 6.2.1). A maximum pressure of approximately 86 psia occurred at approximately 91 sec after the start of the transient. A 14.7 psia ambient pressure was assumed and this resulted in a pressure difference of 71.5 psig, compared with the 60 psig design pressure. The time of maximum pressure did not correspond with the time of maximum temperature. Therefore, a separate load case corresponding to the time of maximum thermal effects on the liner together with the internal pressure at that time was included.

Also, the evaluation was conducted for the conditions at 94 sec rather than 91 sec. The same peak pressure was used but a computer printout for the liner temperature which controlled the thermal stress results was available at 94 sec.

3.8.2.2.4 Seismic Loads

Dynamic seismic loads acting on the Ginna containment structure were replaced by a set of equivalent static loads. The equivalent static seismic loads were computed from a previous analysis of the containment structure conducted by Lawrence Livermore National Laboratory.(*Reference 30*) In the previous Lawrence Livermore National Laboratory analysis, the containment shell was modeled as a fixed base system of lumped masses connected by weightless springs (Figure 3.8-10). Table 3.8-16 lists the values of masses and characteristics of the connecting beams for the model. A response spectrum approach was used to determine the dynamic response of the model, i.e., the first 10 modal responses of the model were combined using the square root of the sum of the squares approach. The Regulatory Guide 1.60 spectrum at 0.2g and 7% critical damping was used for the analysis reported in *Reference 32*. For the structural review, the responses were scaled to a peak ground acceleration of 0.17g in the horizontal direction and 0.11g in the vertical direction. The 0.17g acceleration level is consistent with the site specific safe shutdown earthquake for Ginna. Table 3.8-17 lists modal

frequencies of the model, and Table 3.8-18 shows moment, shear, and axial loads induced in each connecting beam element scaled to 0.17g.

For the combined pressure, thermal, and seismic analysis, the containment shell was modeled as an axisymmetric shell of revolution and seismic loads acting on the shell were input in accordance with the first harmonic mode shape. The circumferential stiffness of the Ginna containment shell was much higher than its radial stiffness and, therefore, only a tangential load was applied to model the lateral seismic loads. Harmonic load amplitudes for the Ginna containment are listed in Table 3.8-19.

3.8.2.2.5 Design and Analysis Procedures

3.8.2.2.5.1 Containment Model

For the SEP reevaluation of Ginna Station, several new analyses were performed to evaluate the structural acceptability of the plant for the current loading conditions that were not considered in the original Ginna design (*Reference 30*).

Even though the containment building is surrounded by the auxiliary, intermediate, and turbine buildings (Figure 3.8-45) there are no structural connections between the containment building and the other buildings. The containment building was therefore modeled and analyzed independently.

The model for the containment shell was similar to the fixed-base cantilever beam model with 12 lumped masses shown in Figure 3.8-10. Mass and section properties are uniform up to elevation 232.66 ft. The remaining shell wall and the dome are modeled by four equivalent beam elements, each with a different uniform section. The following assumptions were made in modeling the containment building and its interior structures:

a. The containment has a rigid foundation at the basement floor (elevation 235.66 ft) and has no lateral support from the surrounding soil above that elevation.
b. Since the concrete containment shell is much stiffer than the steel crane structure, the constraints from the crane structure can be neglected in modeling the containment shell.

This model, shown in Figure 3.8-10, was analyzed by the response spectrum method in the horizontal and vertical directions. The spectral curves of Regulatory Guide 1.60 were scaled to 0.2g peak acceleration for the horizontal component and 0.13g for the vertical component and input as the base excitations. Modal responses and responses to horizontal and vertical excitations were both combined by the square root of the sum of the squares method.

3.8.2.2.5.2 Seismic and Loss-of-Coolant Accident Loads

The analysis for combined seismic and loss-of-coolant accident load combination was performed by Lawrence Livermore National Laboratory (*Reference 32*). For this analysis an axisymmetric, multilayer shell of revolution analytical model was developed for the Ginna containment. The model included the concrete vertical wall and dome and included the 3/8-in. steel liner. Appropriate boundary conditions representing the shell to base slab interface through neoprene pads were included. Since the base slab is founded on rock and the

presence of the neoprene pads essentially isolates the base slab from the containment, the base slab was not included in the model.

Since the scope of this evaluation was to concentrate only on the overall ability of the containment building to withstand the combined seismic and loss-of-coolant accident pressure and thermal loads, numerous details such as personnel and equipment hatches as well as piping and electrical penetrations were not included. The containment shell was assumed to be adequately reinforced around the equipment hatch and other openings so that the effects of these openings on the overall shell response were assumed to be small. Neither were any jet impingement or pipe whip forces considered during this phase of the SEP. The loss-of-coolant accident included both the primary loop loss-of-coolant accident as well as the secondary loop steam line break.

Two different computer codes were used to carry out the analysis. The computer program ANSYS (*Reference 34*) was used to determine the temperature gradient through the shell for steady-state (normal operating) temperature and the transient temperature conditions. Once the temperatures in the shell were determined, the computer program FASOR, Field Analysis of Shells of Revolution (*References 35* and *36*) was used to calculate displacements, stresses, and stress resultants under various loading conditions. FASOR employs a numerical integration method called the "field method" to solve the differential equations of a shell. A shell in FASOR may be modeled as a multilayer shell of revolution, where the thickness material properties, and temperatures for each layer are specified separately. The shape of a shell may be described as a general arc so that there is no need to divide the shell into small elements. The program defines integration points along the shell from an error tolerance specified by the user.

3.8.2.2.5.3 Pressure, Seismic, and Operating Temperature Loads

For pressure, seismic loads, and operating temperature loads, the shell was modeled as two-layers, i.e., a 0.375-in.-thick layer of steel connected to a layer of concrete. The concrete thickness changes from 3 ft. 6 in. in the cylinder to 2 ft. 6 in. in the dome. These thicknesses are nominal values. The true relevant engineering values are dependent on the specific location in the structure and the loading condition that is present. Concrete and steel material properties used in the analysis are listed in Table 3.8-20. For accident temperature loads, the shell was modeled as three layers, i.e., the steel liner and two layers of concrete. The temperature gradient through each layer was assumed to be linear. The boundary condition at the base was assumed to be fixed in the tangential direction. Radial stiffness at the base was computed to be 46.9 kips/in./in. as discussed above.

It was determined from a preliminary analysis that the insulation was effective in limiting the heat flow through the cylindrical portion of the structure and maintaining the insulated liner at a significantly lower temperature than that in the uninsulated liner in the dome. This was verified by a Lawrence Livermore National Laboratory analysis where the temperature of the inside surface of liner and effective film coefficients were computed throughout the containment for the transient thermal loads. This temperature included the temperature drop through the film coefficient at the liner inside surface. In order to develop the thermal gradients through the shell, a transient thermal analysis was performed using ANSYS (*Reference 34*)

with the inside liner surface temperature developed by Lawrence Livermore National Laboratory specified as a boundary condition.

It was found that the insulated part of the containment shell remained close to its steady-state condition throughout the transient time period. On the other hand, temperatures of the uninsulated liner as well as a very thin layer of the concrete containment next to the liner increased significantly as a result of internal transient air temperature. Figures 3.8-46 and 3.8-47 show the temperature gradient through the liner and adjacent concrete 94 sec and 380 sec after the start of the accident. Figure 3.8-46 corresponds to the time of peak pressure and Figure 3.8-47 corresponds to the peak liner temperature during the accident. Although this part of the concrete has only a small effect on the overall shell response, it was included as a separate layer in the analysis. The containment shell was therefore modeled as a three-layer shell consisting of the steel liner and two layers of concrete.

The temperature gradient was assumed to be linear in each of the layers. For the insulated liner, the liner temperature remained approximately at 69°F throughout the accident. The outer concrete surface temperature for both insulated and uninsulated parts of the containment was calculated to be approximately 10°F.

3.8.2.2.6 Structural Acceptance Criteria

For the SEP reevaluation, the seismic capability of critical structures was evaluated using loads developed in the reanalysis. A structure was generally judged to be adequate without the need for additional evaluation for the following two cases:

A. Where loads resulting from the reanalysis were less than those used in the original design.

B. Where loads resulting from the reanalysis exceeded the original loads (or where there was insufficient information about the original seismic analysis for a comparison) but the resulting stresses were low compared to the yield stress of steel or the compressive strength of concrete.

For cases in which the seismic loads from the reanalysis were not low and exceeded the steel yield stress or the concrete compressive strength, conclusions were reached on the basis of the estimated reserve capacity (or ductility) of the structures; that is, the capability of structures to deform inelastically without failure.

3.8.2.2.7 Structural Evaluation of Containment

The structural acceptability of the containment based on the SEP reevaluation is described in the following.

3.8.2.2.7.1 Seismic Analysis

There was sufficient information available for the containment building original seismic design and analysis to make a comparison to current criteria.

The original analysis was an equivalent static analysis, which was checked by a response spectrum analysis using Housner spectra. The seismic design loads were based on the equivalent static analysis. The reanalysis gave seismic loads higher than those of the original

Housner response spectrum analysis but lower than the seismic design loads from the equivalent static analysis (Figure 3.8-10). The containment building is therefore considered to be acceptable in light of current criteria if the structure meets the original design criteria.

3.8.2.2.7.2 Load Combinations

It was found that the effect of accident temperature was mainly in the uninsulated part of the dome. The meridional moment increased from 290 kips-ft/ft for the operating temperature to a peak value of 551 kips-ft/ft after 380 sec based on the very conservative accident curves used (see Sections 3.8.2.2.2 and 3.8.2.2.3). The moment in the cylinder remained at approximately 400 kips-ft/ft throughout the transient. Containment axial response to deadweight and prestress loads were computed to be 74 kips/ft and 299 kips/ft, respectively. Since it is unlikely that peak horizontal and peak vertical seismic loads happen at the same time, they were combined using the square root of the sum of the squares method. Since the pressure load and seismic loads were acting upwards, there was very little additional margin of safety available to resist containment uplift in the case of a combined seismic event and loss-of-coolant accident. However, even if the prestress and deadweight loads were overcome over a small segment of the shell, the vertical tendons would remain intact and the liner knuckle flexibility would provide for some uplift before liner failure could be expected. The seismic response of the structure for this case was based on the assumed 7% damping as discussed in *Reference 30*. To determine the required limiting capacity of the shell, two load combinations were considered. For the load combination D + P + E loads, radial shear, moment, and hoop tension were dominated by the peak pressure load (86 psia), while tangential shear was mainly due to the seismic lateral loads. For the $D + P + E + T_a$ load combination the displacement and meridional moment in the shell were very much affected by the transient accident temperature. The peak response parameters, especially hoop tension and meridional moment in the dome, were higher than their original design values.

It should be noted that the high meridional moment in the dome was mainly due to the thermal gradient through the shell which has a self-limiting effect due to shell cracking.

In order to check the stresses in concrete and reinforcing steel in the dome, a cracked section analysis based on simple elastic bending theory was carried out. The analysis was for the temperature load which corresponded to a pressure load of 69 psia. The results showed that the maximum stress in the main reinforcing steel in the dome was 12.8 ksi which was much lower than the ASME code allowable of $0.9\ \sigma_y = 36$ ksi. Also, the peak stress in the welded wire fabric which was placed towards the outer surface of the containment shell was below the steel yield stress. Maximum concrete compressive stresses were computed to be 3700 psi which was less than the code allowable of $0.85\ f'_c = 4250$ psi.

Radial restraint to withstand the temperature and pressure loads at the base slab-containment vessel interface was provided by radial bars. The maximum tensile stress in these bars under the combined loads was approximately 54 ksi. The 130 ksi minimum yield strength of these bars provided a substantial margin of safety.

The seismic overturning moment in combination with internal pressure was resisted by the dead weight of the vessel and the rock anchors. A factor of safety of approximately 1.0

existed for separation of the cylinder and base slab assuming 7% of critical damping in the seismic response of the structure. However, the liner knuckle was found to have adequate flexibility to resist some uplift without failure.

3.8.2.2.8 Structural Evaluation of Large Openings

Principal stress-resultants and stress-couples were computed and found to be co-linear or essentially so for all panels which were significant in the design check. Likewise the orientation of stress-resultants and stress-couples was found to essentially coincide with the mild steel reinforcement for all significant panels. Interaction diagrams were prepared based upon procedures for ultimate strength design of ACI 318-63.

The interaction diagrams showed that sufficient reinforcement was provided to carry all loads, including the full thermal stress-resultants and stress-couples.

3.8.2.2.9 Structural Evaluation of Tension Rods

The radial loads are resisted by the radial tension rods in the outward direction, while the radial loads in the inward direction are resisted by the concrete base slab in bearing. The thermal and pressure loss-of-coolant accident loads result in radial expansion and tension in the rods. The stiffness of the liner knuckle in the radial direction is very low compared to the rods and virtually no radial loads are transmitted through the liner. The maximum tensile stress computed in the rod for the combined load case was approximately 54,000 psi. No shear stress was developed in the rods due to the clearance between the rod and sleeve in the base slab. The minimum tensile yield strength in the rods is 130,000 psi so that a factor of safety of approximately 2.6 exists for this detail.

3.8.2.3 Dome Liner Reevaluation

Gilbert Associates performed an analysis to evaluate the behavior of the containment dome liner and studs under the pressure and temperature loads that are caused by the loss-of-coolant accident and steam line break loading conditions (*Reference 29*).

3.8.2.3.1 Dome Liner Studs

The stud scheme were used in supporting the dome liner is shown in Figure 3.8-48.

The scheme starts at the springline between the dome and cylinder, and extends to the apex. In this region the studs are 5/8-in. diameter Nelson S6L studs, and they are spaced at 2 ft-0 in. as shown in Figure 3.8-48. The S6L studs have internal threads to accept 1/2-in. diameter threaded fasteners. One-half in. diameter rods were threaded into the studs and the other end of the rod was bent around the three layers of #18 reinforcement in the dome. This was done to support the liner during concrete placement.

3.8.2.3.2 Loads

3.8.2.3.2.1 Loss-of-Coolant Accident

The dome liner and studs were evaluated based on the loss-of-coolant accident pressure and temperature transients in Figures 6.2-1 and 6.2-2.

3.8.2.3.2.2 Steam Line Break

The peak air temperatures for the steam line break exceed the loss-of-coolant accident peak temperature. However, for the liner evaluation it is the peak liner temperature rather than the peak air temperature which is important. The peak liner temperatures are not very different for the loss-of-coolant accident and steam line break because even though the peak loss-of-coolant accident air temperature is less than the steam line break air temperatures, the loss-of-coolant accident temperature remains near its maximum considerably longer than the temperatures for the steam line break, thus allowing more time for the liner temperature to increase. Based on this, the temperature of the liner is not expected to be significantly different from the loss-of-coolant value of 250°F.

A liner temperature of 250°F coincident with a pressure of 57.8 psig was used for the steam line break condition in the evaluation of the dome liner and studs.

3.8.2.3.3 Model Definition

3.8.2.3.3.1 General Dome Model

In the general dome area, Figure 3.8-49, the liner panels between studs are stressed equally under the pressure and temperature loads corresponding to the loss-of-coolant accident or steam line break conditions. For a liner without imperfections, all of the liner panels between the studs would reach their limiting stress capacities simultaneously. Under this condition, there would be no resultant shear force on the studs. However, if one panel is assumed to buckle prior to others, shear forces would be experienced by the adjacent studs. With the one panel buckled, the adjacent panels and studs displace towards the buckled panel. As a result of this displacement, the buckled panel displaces laterally further away from the concrete and exhibits a fall-off in its membrane stress as described in *Reference 37*. The extent of stress fall-off depends on the final displacement, Δ, of the studs on either side of the buckled panel. The difference between the fall-off stress in the buckled panel and the final stress in the adjacent panel produces a shear on the stud. The largest shear force and displacement occur for stud #1.

The liner plate material for the Ginna liner is ASTM A 442 grade 60 carbon steel, which has a minimum specified yield strength of 32 ksi. It is expected that the liner would have an actual mean yield strength of 48 ksi based on the information in *Reference 38*. In the general dome for the liner panels between the 3/4-in. diameter headed studs spaced at 4 ft-3 in., the calculated buckling stress is 5.8 ksi. For the liner panels between the 5/8-in. diameter S6L studs spaced at 2 ft-0 in., the calculated buckling stress is much less than the 32 ksi or 48 ksi yield strength, the calculated buckling stress is used as the value of limiting stress for all panels in the model adjacent to panel 1-1. The limiting compressive stresses in these panels of 26 ksi (or 5.8 ksi) combine and displace the critical stud (#1) in the direction of the buckled panel in the model for the general dome.

3.8.2.3.3.2 Insulation Termination Region Model

In the insulation termination region, the stresses in the liner behind the insulation are small relative to the large compressive stresses produced in the uninsulated portion of the liner. In the liner panel immediately outside the insulation, the largest compressive stress that is

capable of being developed will produce the largest displacement of the studs. This is the limiting stress corresponding to the calculated buckling stress of 26 ksi. With the panel stressed to this value, all of the studs behind the insulation will displace as indicated in Figure 3.8-49. The stud which experiences the greatest displacement is stud #1.

3.8.2.3.4 Analysis

3.8.2.3.4.1 Controlling Loads

The controlling loss-of-coolant accident loads on the dome liner are a liner temperature of 250°F coincident with an internal pressure of 42 psig. For the controlling steam line break condition the liner temperature is 250°F with an internal pressure of 57.8 psig. The 250° F temperature applies in the uninsulated portion of the dome liner. Behind the insulation the liner temperature decreases as indicated in Figure 3.8-50. The liner stresses were obtained using the elastic, shell analysis computer program KSHEL1 (*Reference 39*) for the controlling loss-of-coolant accident and steam line break loads. The results of these analyses indicated that the stresses in the uninsulated portion of the liner were generally in the neighborhood of 45 ksi compression. This value exceeds the limiting stresses of 26 ksi and 5.8 ksi discussed previously. Therefore, these limiting stresses control and were used in the liner-stud interaction analyses.

As additional cases, the liner-stud interaction analyses also reviewed somewhat higher values of limiting stresses in order to determine the sensitivity of the stud displacements to variations in the stress limits. This accounts for the real possibility that some liner panels may buckle at a stress greater than their theoretical value. For this purpose, the limiting stress of 26 ksi for the 2 ft 0 in. panels was increased only 10%, resulting in 29 ksi as an additional case for the analysis of the 5/8-in. diameter S6L studs in the general dome and in the insulation termination region. Considering the usual scatter in buckling test results, it is not unreasonable to expect that there would be liner panels which could develop membrane compressive stresses 10% above the theoretical buckling value of 26 ksi. For the liner panels in the general dome where the 3/4-in. headed studs at 4 ft 3 in. spacing exist (since the 5.8 ksi stress limit was relatively low) it was practically doubled to 12 ksi. This value was used as a conservatively high stress limit.

3.8.2.3.4.2 Liner-Stud Interaction

For the general dome, the analysis was based on the method developed in *Reference 37* using the model in Figure 3.8-49. The appropriate equations in this reference were modified to include the effect of the internal pressure on the stress fall-off curve for the buckled panel 1-1. For the insulation termination region, a somewhat different liner-stud interaction analysis was performed using the model in Figure 3.8-49. The main difference is that the stress in the buckled panel (26 ksi or 29 ksi) is given and the stress fall-off concept does not apply.

In these analyses, the force-displacement curves of the embedded studs are required for the 3/4-in. diameter headed studs and for the 5/8-in. diameter S6L studs. The determination of these curves is discussed below.

3/4-Inch Diameter Studs

The curve used for the 3/4-in. headed studs is shown in Figure 3.8-51. This curve is based both on the test results and recommendations from *Reference 40* and from test data reported in *Reference 41*. From *Reference 40* the shape of the force-displacement relationship is provided by Equation (4) in *Reference 40* as $Q = Qu\ (1\text{-}e^{-18\,\Delta})^{2/5}$. In the equation, Δ is the stud displacement, Q is the corresponding stud force, and Qu is the ultimate stud capacity. The ultimate stud capacity was obtained from Equation (3) of *Reference 40* as 31.1 kips. Test data from *Reference 41* for 3/4-in. diameter studs in shear (Table IX) support this value for Qu.
The ultimate shear force values reported here from four stud tests all exceed 31.1 kips. Also from *Reference 41*, a displacement of 0.341 in. at failure (Table X) is reported for the 3/4-in. studs, and this value is used as the ultimate displacement in Figure 3.8-51.

5/8-Inch Diameter Studs

Unlike the 3/4-in. headed studs, force-displacement property data for the 5/8-in. S6L studs was not found in the Nelson literature. Therefore, the curve for these studs was constructed indirectly from tests on other types of anchors. In *Reference 42*, direct shear tests on 3/8-in. diameter and 1/2-in. diameter Nelson D2L deformed reinforcing bar anchors are reported. The embedment lengths of these bars varied over 3 in., 6 in., 12 in., and 18 in. The test results for the 18-in. long bars indicate that these bars failed in shear at or slightly above the minimum specified tensile strength of the bar material, which was 80 ksi. The results from the tests on the 18-in. long bars are believed to be applicable to the 5/8-in. S6L studs installed on the dome liner since these studs were actually extended in length by the 1/2-in. diameter threaded rods that bend around the 3 layers of #18 dome reinforcement. The studs with the rods had straight embedment distances of 9-1/2 in., 14 in., and 18-1/2 in. This configuration will adequately develop these studs to allow them to achieve their minimum specified tensile capacity in shear based on the test results for the 18-in. long straight deformed bars. The capacity for the 5/8-in. diameter S6L stud then becomes

$$Fu = As\ fs = (\pi/4)(0.437)^2(60\text{ ksi}) = 90\text{ kips}$$

where the minimum diameter of the stud (0.437 in. at the base) was used. For use in the evaluation as a lower bound study capacity, 8.3 kips was used. This represents approximately a 10% reduction of the 9.0 kips value.

Actually the 9.0 kips value itself would appear to be a conservatively low value for the S6L studs due to their lower specified tensile strength of 60 ksi compared with the corresponding value of 80 ksi for the deformed bars tested in *Reference 42*. This would be the case because the actual tensile strengths of the S6L stud material are expected to consistently exceed their 60 ksi minimum specified value by greater margins than would occur for the 80 ksi strength material for the deformed bars. An example of the increase for 60 ksi grade studs is seen in the tests on the 3/4-in. diameter headed studs discussed previously. The steel for these studs (A108) has a minimum specified tensile strength of 60 ksi, which when multiplied by the stud area (0.442 in.^{2}) gives a capacity of 26.5 kips. However, these studs consistently failed above 30 kips in the tests reported in *References 40* and *41*. Therefore, use of the value Qu = 8.3 kips in the liner-stud interaction analyses is regarded as a conservative lower bound on the

expected actual capacity of the 5/8-in. diameter S6L studs. The determination of a more realistic value is discussed below.

The results and recommendations of *Reference 40* were used to establish what is regarded as an expected value for Qu for the 5/8-in. diameter S6L studs. *Reference 40* is applicable because the headed studs tested in this reference have a minimum specified tensile strength of 60 ksi which is the same as the specified tensile strength of 5/8-in. S6L stud material. Also, the embedment afforded the S6L studs on the dome by the bent 1/2-in. threaded rods is believed to be at least as effective as the head on the studs tested in *Reference 40*. Using Equation (3) from *Reference 40* gives:

$$Qu = \frac{1}{2} A_s \sqrt{f_c' E_c} = 0.5\frac{\pi}{4} \cdot 0.437^2 \sqrt{(5)(4000)} = 10.6 kips$$

(Equation 3.8-22)

Curves corresponding to Qu = 8.3 kips (lower bound) and Qu = 10.6 kips (best estimate) are shown in Figure 3.8-52. The ultimate displacement of 0.167 in. is the limit chosen for the 5/8-in. diameter S6L studs. In the absence of any specific data on these studs, the 0.167-in. value was obtained from the tests on 1/2-in. diameter headed studs reported in *Reference 41* (Table X). The value is in reasonable agreement with the deformed bar tests from *Reference 42*. In these tests on the 1/2-in. diameter by 18-in. long deformed bars, an ultimate displacement of approximately 0.160 in. was reported.

In summary, the liner-stud interaction analyses were based on the force displacement curve for the 3/4-in. diameter headed studs shown in Figure 3.8-51. This curve is based on actual test results as reported in *References 40* and *41*. The curves for the 5/8-in. diameter S6L studs are shown in Figure 3.8-52. In the absence of specific test data on the S6L studs, lower bound and best estimate curves were constructed based on tests reported in *References 40* and *42*.

3.8.2.3.4.3 Effect of Internal Pressure on Liner Buckling

The internal pressure potentially affects the liner buckling stress and stud evaluation in all three regions of the dome liner shown in Figure 3.8-48. Therefore, an evaluation of all studs was performed considering the internal pressure effect as a separate case in addition to the liner-stud interaction analyses described previously.

In order to specifically address the effect of internal pressure, it was necessary to solve the fundamental buckling problem of a straight strut, clamped at its ends, under the combined loads of a uniform temperature increase over the length of the strut plus a uniform lateral pressure. In addition the strut is continuously supported on the side opposite the pressure, which permits buckling to occur only in the direction opposed by the pressure. The resulting model is shown in Figure 3.8-53. The length of the strut, L, corresponds to the stud spacing of either 2 ft 0 in. (24 in.) or 4 ft 3 in. (51 in.). The temperature increase of the strut, ΔT, corresponds to the temperature increase (above a stress free state at 70°F) which the liner experiences under a loss-of-coolant accident or steam line break condition. Likewise, the pressure on the strut, P, corresponds to the internal pressure in the containment (above atmospheric) occurring simultaneously with the liner temperature.

The buckling problem was solved using an energy method. In this approach, expressions were derived for the strain energy in the strut both before (straight) and after (deflected) buckling. In the unbuckled position the strain energy is that due only to the membrane compressive stress in the strut produced by the full restraint to ΔT. In the deflected position, both bending and membrane strain energy are present. Also in the deflected position, only lateral displacements which satisfy the equilibrium conditions on the strut are admissible. The buckling problem is solved by determining the value of temperature increase, ΔT, in the presence of the pressure, P, required to make the strain energy of the straight strut equal to the sum of (1) the strain energy of the deflected strut and (2) the work done as P displaces from the straight to the deflected position of the strut. This value of temperature is the temperature increase required to buckle the strut (the liner panel) as it is concurrently acted upon by the specific pressure.

The resulting buckling curves for the liner panels corresponding to stud spacings of 24 in. and 51 in. are shown in Figure 3.8-54. The values at P = 0 are ΔT = 27.4°F for L = 51 in. and ΔT = 123.6°F for L = 24 in., both of which produce corresponding liner stresses equal to the Euler buckling values. From the curves in Figure 3.8-54, the increase in liner temperature required to cause buckling as the pressure increases is evident. For example, an internal pressure of 10 psig (24.7 psia) increases the buckling temperature (and stress) by factors of 6.0 (L= 51 in.) and 1.7 (L = 24 in.).

Superimposed on the buckling curves are values of liner temperature and internal pressure which are based on the loss-of-coolant accident curves of Figures 6.2-2 and 6.2-1. These are discussed in Section 3.8.2.3.2.

<u>3.8.2.3.5</u> <u>Results and Conclusions</u>

The results of the liner-stud interaction analyses are presented first for limiting stresses of 26 ksi and 29 ksi for the 5/8-in. diameter S6L studs spaced at 24 in. and for limiting stresses of 5.8 ksi and 12 ksi for the 3/4-in. diameter headed studs spaced at 51 in. Following this, the effect that the internal pressure has on the results are discussed.

3.8.2.3.5.1 Insulation Termination Region

The results from four separate liner-stud interaction analyses are presented in Table 3.8-21 for the studs in the insulation termination region of the dome liner. These studs are the 5/8-in. diameter S6L studs. Column (1) identifies the stud capacity, Qu, which is based on the force-displacement curve from Figure 3.8-52 used in the particular analysis. Column (2) identifies the stress in the liner just outside the insulation. The acceptance criteria for the studs is based on stud displacement, and the maximum displacement occurs for the #1 stud in Figure 3.8-49. These values are shown in column (3) and they are to be compared with the ultimate stud displacement of 0.167 in. in column (4). The percentage of the maximum displacement relative to the ultimate value is indicated in column (5). These values range from 84% to 99%. The results associated with the 10.6 kips stud capacity are more applicable than the values associated with the 8.3 kips lower bound stud capacity for the reasons discussed in Section 3.8.2.3.4.2. Therefore, the maximum stud displacement is estimated to be either 84% or 95% of its ultimate value, depending on the maximum stress which will be developed in the liner. These results are less than the 100% value indicating stud failure. However, considering the

magnitude of the displacements and their sensitivity to the 10% increase in the theoretical limiting liner stress of 26 ksi, some of the studs located just outside the insulation could possibly fail.

Any stud failures which might occur would not be expected to tear the liner, based on test results reported in *Reference 43*. This reference describes tests conducted on 1/2 in., 5/8 in., and 3/4 in. diameter headed studs attached to steel flanges of various thicknesses, ranging from 0.128-in. thick to 0.389-in. thick. A total of 41 specimens were tested in all. The primary objective of the tests was to determine the mode of failure of the studs and under what conditions failure would occur by tearing of the flanges rather than in the stud itself. The main conclusion reached from the tests is that if the ratio of stud diameter to flange thickness is less than 2.7 then the studs will fail in their shank and flange tearing or pull-out will not occur. For the 5/8-in. diameter S6L studs, the diameter-to-thickness ratio is 0.437/0.375 or
1.17. This value is much less than the 2.7 limiting value; therefore, any failure of the S6L dome liner studs would not result in a tearing of the liner.

3.8.2.3.5.2 General Dome

The results of the liner-stud interaction analysis for both regions of the general dome are presented in Table 3.8-22. The results in columns (1) through (5) were identified earlier. For the buckled panel in the general dome model (Figure 3.8-49), the displacements and strains are also of interest and these values are indicated in columns (6) through (9). The results for the 5/8-in. diameter S6L studs and 3/4-in. diameter headed studs are discussed separately below.

5/8-Inch Diameter S6L Studs

As indicated in the previous discussion of the insulation termination region, the results which are based on the stud capacity of 10.6 kips, rather than 8.3 kips, are considered to represent the best estimate for the S6L studs. The results in column (5) of Table 3.8-22 indicate a maximum stud displacement of either 68% or 102% of the ultimate value, depending on whether the limiting stress in all the unbuckled panels is 26 ksi or 29 ksi. Thus, the stud displacements are very sensitive to the stress limit developed in the adjacent panels. These results can be interpreted as follows referring to the model in Figure 3.8-49. For the studs adjacent to the buckled panel (1-1) to actually displace 102% of their ultimate value, the stress in all 19 adjacent panels would have to reach 29 ksi. This condition would occur only if there were no initial imperfections in these panels to cause them to buckle at a stress less than 29 ksi. If only one panel within the 19-panel group were to buckle at less than 29 ksi, the displacement of the
#1 stud in the model would probably be reduced to below 100%. Considering the results, it is possible that some of the S6L studs in the general dome region could fail. However, based on the test results in *Reference 43* discussed previously, any stud failures would not tear the liner in the process.

The relatively large lateral displacements in column (6) of Table 3.8-22 for the 24-in. buckled panel (1-1) deserve some attention because of the large associated strains. Due to these lateral displacements of the buckled panels, plastic hinging is calculated to occur. The strains which are produced across the liner section in the hinge region are given in columns (7), (8), and (9).

The largest membrane strain from column (7) of Table 3.8-22 for a Qu of 10.6 kips is 0.0096 in./in. compression. This value is six times the yield strain based on a 48 ksi liner yield stress. However, this strain, being compression, is not significant as far as liner integrity is concerned.

The extreme fiber strains (bending plus membrane) indicated in columns (8) and (9) of Table 3.8-22 are large by conventional measures as the results in column (10) indicate. Here, for the worst case, the extreme fiber strain is 39 times the yield strain of the liner material. To put this magnitude of strain in perspective, an extreme fiber strain equal to 39 times yield would be produced in a bend test if the liner were bent around a circular pin having a diameter of 5.6 in. The liner, being a low carbon steel, is ductile enough to be bent to this diameter without tearing. The version of the ASTM specification, A442, used for the Ginna containment liner material required that liner specimens be cold bent through 180 degrees around a pin diameter equal to the liner thickness of 0.375 in. without cracking the specimen. It is indicated in Section 3.8.1.6.5 that these tests were performed for each as-rolled liner plate supplied. This test produces an extreme fiber strain in the liner which is calculated to be 313 times the yield strain. These tests demonstrated that the liner is capable of undergoing bending strains which are much larger than those calculated for the buckled panels. Therefore, the structural integrity of the liner will not be impaired under the strain conditions calculated to exist.

<u>3/4-Inch Diameter Headed Studs</u>

The maximum stud displacements corresponding to limiting stresses in the unbuckled panels of 5.8 ksi and 12 ksi are shown in column (3) of Table 3.8-22. In both cases the maximum stud displacements are small, being only 11% of the ultimate value at worst. The corresponding strains in the buckled panel (1-1) due to the lateral displacement of the panel are also small; the largest value is only 1.5 times the yield strain. Thus, even though the liner is supported by a relatively large stud spacing of 51 in., which results in a low bucking capacity, the displacement of the liner does not produce strains which would impair its structural integrity.

Based on these results, it can be concluded that failure of the 3/4-in. diameter headed studs is extremely unlikely. Any stud failures that might unexpectedly occur would not tear the liner, even for studs as large as these. Recalling the conclusions from *Reference 43*, the stud diameter-to-liner thickness ratio is 0.75/0.375 or 2; and this is well within the 2.7 limit below which stud failure does not tear the liner in the process.

3.8.2.3.5.3 Effect of Internal Pressure on Liner Buckling and Stud Integrity

The buckling capacity of the liner under the combined effects of a temperature increase and coincident pressure is presented in Figure 3.8-54. The curves in the figure define the buckling capacity in the two regions of the liner where the stud spacings of 24 in. and 51 in. exist. For comparison, values of the liner temperature and internal pressure are indicated. These result from the loss-of-coolant accident conditions in Figures 6.2-1 and 6.2-2. The liner temperatures were obtained from a heat transfer analysis of the loss-of-coolant accident temperature transient in Figure 6.2-2. The time into the loss-of-coolant accident transient is indicated for several of the pressure and temperature values. For example, at 100 sec into the transient the liner temperature has increased 173°F (above 70°F) and the simultaneous pressure on the liner is 53 psig (67.7 psia).

The comparison in Figure 3.8-54 indicates that for the first 2.15 hours (7740 sec) into the transient, the internal pressure prevents the liner from buckling in all regions of the dome. During this time, the liner reaches a maximum temperature of approximately 260°F (190°F increase above 70°F), which is considerably above the temperature required to buckle it even in the region where the studs are spaced at 24 in. However, buckling does not occur because at this temperature the coincident containment pressure is 42.7 psia (28 psig). After 2.15 hours into the transient, when the internal pressure has decreased to 24.7 psia (10 psig), the results indicate that the region of the liner where the studs are spaced at 51 in. (3/4-in. headed studs) is susceptible to buckling. By that time, the liner temperature has reduced to approximately 250°F. The region of the liner where the studs are spaced at 24 in. (5/8-in. S6L studs) remains unbuckled. The effect of these results on the liner and stud evaluation is discussed below.

For the insulation termination region and the general dome region, the conclusions regarding the potential for stud failure were that failure of some of the 5/8-in. diameter S6L studs located in the insulation termination region and in the general dome region might occur, depending on whether or not the limiting stress of 26 ksi is actually exceeded. For the 5/8-in. diameter S6L studs in the general dome, this conclusion was based on an initial assumption that one panel has buckled. However, the comparison in Figure 3.8-54 indicates that the liner panels associated with these studs are not likely to buckle because of the effect of the internal pressure. The assumption that a buckled panel exists with the result that shear forces are produced in the studs is not considered to be realistic in light of these results. Therefore, stud failure is not expected to actually occur. For the remaining 5/8-in. diameter S6L studs in the region of the liner where the insulation terminates, the fact that the liner panel remains unbuckled increases the stress that is capable of developing well above the 26 ksi and 29 ksi limits used in the previous interaction analyses. The stress increases to a maximum value of approximately 47 ksi, which corresponds to the maximum liner temperature of 260 °F. The 47 ksi compressive stress exceeds the specified minimum yield strength of 32 ksi, but it is considered to be achievable since the actual average yield strength of the liner plates is expected to be in the neighborhood of 48 ksi. The effect of a 47 ksi stress occurring in the liner region outside the insulation would be to cause failure of the studs in the insulation termination region of the dome. However, based on the test results in *Reference 43* discussed previously, failure of these studs would not affect the integrity of the liner.

The remaining studs are the 3/4-in. diameter headed anchors in the region of the general dome which extends from the 55-degree meridian to the apex. The conclusions regarding the general dome area were that because of the relatively low buckling capacity of the liner in this region, the limiting stresses were small. The corresponding calculated stud displacements were considerably less than their ultimate values and stud failure was considered to be very unlikely. When the pressure effect is taken into account, it is also concluded that these studs will not fail during at least the first 2.15 hours of the loss-of-coolant accident transient because the liner panels would not buckle and, consequently, no unbalanced panel forces would exist to produce shear on the studs. Beyond this time, from Figure 3.8-54, the loss-of-coolant accident pressures and temperatures fall somewhat below the buckling curve for the 51-in. stud spacing and buckling of some liner panels could occur. If one panel buckles but adjacent panels do not, the 250°F liner temperature would produce a 45 ksi compressive stress in the unbuckled panels. This would result in an unbalanced shear force in the studs

that is large enough to cause their failure. However, this condition would not affect liner integrity because the ratio of stud diameter-to-liner thickness being 2.0 is significantly less than the limiting value of 2.7 required to tear the liner. After 2.15 hours into the loss-of-coolant accident transient, the internal pressure is down to approximately 10 psig which is far below the maximum value of 60 psig that the containment structure has been designed to resist and the stresses in the reinforced-concrete structure are relatively low.

<u>3.8.2.3.6 Overall Conclusions</u>

Of the results and conclusions presented above, those based on a consideration of the internal pressure are considered to be more realistic since pressure would actually be present in a loss-of-coolant accident transient loading condition on the liner.

In the region of the dome where the insulation terminates, the liner is expected to remain in an unbuckled condition. As a result, unbalanced compression stresses in the liner are produced which are large enough to result in failure of the 5/8-in. diameter S6L studs located in this region based on the results of the liner-stud interaction analyses described herein. However, failure of these studs would be limited to the shank of the studs and not in the liner. Therefore, the leaktight integrity of the liner will be maintained.

Above the insulation and extending to the 55-degree meridional coordinate axis on the dome, a distance of approximately 35 ft, the liner is expected to remain in an unbuckled condition, and no unbalanced compressive stresses exist in the liner. Because of this, no shear forces are produced in the 5/8-in. diameter S6L studs in this region and, consequently, stud failure would not be expected to occur.

Above the 55-degree meridional coordinate axis and extending to the apex of the dome, the liner panels are susceptible to buckling late in the loss-of-coolant accident transient after the containment pressure has reduced to approximately 17% of the design pressure of the containment structure. In the event that a panel buckles but adjacent panels remain unbuckled, unbalanced compressive stresses are produced which are large enough to fail some of the 3/4 in. diameter studs in this region. However, failure of these studs is predicted to occur in the shank of the studs and not in the liner. In addition, the liner plate material has demonstrated the capacity to accommodate strains which are much greater than the strains which the buckled liner panels are expected to undergo. Therefore, the leaktightness of the liner will be maintained.

The NRC Staff reviewed the analyses and concluded that it is unlikely that any stud failure will result in tearing of the containment liner and, therefore, the liner will retain its leaktight integrity during the postulated loading conditions (*Reference 44*).

3.8.3 CONTAINMENT INTERNAL STRUCTURES

3.8.3.1 Description of the Internal Structures

The containment interior structures include the concrete reactor vessel support, concrete floors (at elevations 245 ft, 253.25 ft, and 278.33 ft), concrete shield walls, the steel overhead crane support structures, the nuclear steam supply system, and other auxiliary equipment (see Figure 3.8-55).

The concrete internal structure is supported entirely on the base slab. No structural connections exist between the concrete internal structure and the containment shell and radial gaps permit unrestrained relative motion between the two structures. The only connection between the containment shell and its interior structures is at the top of the crane rail, where the rail top may bear on the concrete shell at four locations of neoprene pads. Figure 3.8-55 shows the overall configuration of the reactor building including the internals and major nuclear steam supply system equipment items.

3.8.3.2 Applicable Codes, Standards, and Specifications

The SEP reevaluation of the containment internal structures was performed using ACI 349-80.

3.8.3.3 Loads and Load Combinations

3.8.3.3.1 Load Combinations Considered

The loads (defined in Table 3.8-23) and load combinations to be considered on a generic basis according to current requirements (ACI 349-80) are as follows:

1. $1.4 D + 1.4 H + 1.7 L + 1.7 R_o$
2. $1.4 D + 1.4 H + 1.7 L + 1.7 E_o + 1.7 R_o$
3. $1.4 D + 1.4 H + 1.7 L + 1.7 W + 1.7 R_o$
4. $D + H + L + T_o + R_o + E_{ss}$
5. $D + H + L + T_o + R_o + W_t$
6. $D + H + L + T_a + Ra + 1.25 P_a$
7. $D + H + L + T_a + R_a + 1.15 Pa + 1.0 (Y_r + Y_j + Y_m) + 1.15 E_o$
8. $D + H + L + T_a + R_a + 1.0 P_a + 1.0 (Y_r + Y_j + Ym) + 1.0E_{ss}$
9. $1.05 D + 1.05 H + 1.3 L + 1.05 T_o + 1.3 R_o$
10. $1.05 D + 1.05 H + 1.3 L + 1.3 E_o + 1.05 T_o + 1.3 R_o$
11. $1.05 D + 1.05 H + 1.3 L + 1.3 W + 1.05 T_o + 1.3 R_o$

Any earth pressure loads are included in live load (L).

3.8.3.3.2 Applicable Load Combinations

Additional review of each of the code change elements was conducted to determine if the remaining loads, generically applicable to the structure, had any potential impact. As a result of this additional review, loads H, T_o, W, and W_t were considered not to have any significant effect. The H loads were not considered because there is no significant hydrostatic head on the containment interior structures. The T_o loads were not considered because they tend to equalize throughout the containment interior, thus resulting in no significant temperature differentials. The W and W_t loads were not considered because containment interior concrete is

enclosed by the containment shell, which withstands wind and tornado loads. Considering the results of both reviews, the generic load combinations are reduced to the following applicable combinations:

1. $1.4\ D + 1.7\ L + 1.7\ R_o$
2. $1.4\ D + 1.7\ L + 1.7\ E_o + 1.7\ R_o$
3. $1.4\ D + 1.7\ L + 1.7\ R_o$
4. $D + L + R_o + E_{ss}$
5. $D + L + R_o$
6. $D + L + R_a$
7. $D + L + R_a + 1.15\ E_o$
8. $D + L + R_a + E_{ss}$
9. $1.05\ D + 1.3\ L + 1.3\ R_o$
10. $1.05\ D + 1.3\ L + 1.3\ E_o + 1.3\ R_o$
11. $1.05\ D + 1.3\ L + 1.3\ R_o$

3.8.3.4 Design and Analysis Procedures

3.8.3.4.1 Original Design

In the original design of Ginna Station reinforced-concrete structures inside the containment were modeled as simple cantilever beams with all mass lumped at the center of gravity. Analysis was by the equivalent static method as follows:

A. The fundamental period was calculated based on the assumption that the structure is a simple harmonic oscillator.

B. The response acceleration was taken from the appropriate response spectrum (Figures 3.7-1 and 3.7-2).

C. This acceleration times the total mass acting at the center of gravity gave the shear force and overturning moment at the base.

D. The shears and moments were distributed throughout the model in proportion to structural stiffness, which was based on the flexural properties of the wall systems.

E. Structural element design capacity was evaluated.

Walls and floor slabs were designed for the concentrated seismic reactions of the attached major components.

Overhead crane support structures within the containment building were reportedly evaluated for natural periods of simple harmonic motion in the two horizontal directions. Equivalent horizontal seismic forces were then obtained by applying the corresponding acceleration from the seismic response spectra to the mass of the crane. Vertical response of the crane and crane

support structure was taken as the peak of the response spectra. Vertical forces were obtained by applying the peak acceleration to the mass of the crane, crane support structure, and lifted load.

3.8.3.4.2 Systematic Evaluation Program Reevaluation

During the Systematic Evaluation Program seismic reevaluation (*Reference 30*) Lawrence Livermore National Laboratory developed a mathematical model that included the interior structures, the nuclear steam supply system, and the crane structure and was based on a model developed for RG&E by Gilbert Associates, Inc., in 1979 (*Reference 45*). The following assumptions were made in modeling the interior structures:

A. The model for the interior structures and crane supports included the constraint effect from the containment shell at the crane top.

B. The interior structures were assumed to have rigid diaphragms at elevations 245, 253.25, 267.25, and 278.33 ft. Masses of all concrete floors and walls were lumped to the centers of gravity of the diaphragms. Major nuclear steam supply system equipment items, including steam generators, coolant pumps, and the reactor vessel, were modeled as lumped-mass systems.

C. The crane structure was assumed to have two lumped masses located at the center of the crane structure at elevations 329.66 ft and 311 ft.

D. Based on the recommendation in NUREG/CR-0098, damping was assumed to be 7% of critical damping for the steel-and-prestressed-concrete part of the structures and 10% for the concrete part.

The interior structures model, which was prepared for the computer program STARDYNE, included plate elements for the concrete shield walls and rigid beams for the rigid floors (Figure 3.8-56). The concrete-and-steel columns were represented by elastic beam elements. The nuclear steam supply system and the neoprene pads at the crane top were included as equivalent stiffness matrices. A cantilever beam model that had seven lumped masses represented the containment shell. The total mass of each floor was lumped to the center of gravity of the floor, and rotational inertia was accounted for. Equipment masses were represented by lumped masses at the corresponding nodes. There were 99 nonzero-mass degrees of freedom in the model. Use of the Guyan reduction technique reduced the 99 to the 45 associated with the interior structure floor centers of gravity and containment shell nodes.

3.8.3.5 Method of Analysis

The model was analyzed by the response spectrum method in the horizontal and vertical directions. The spectral curves of Regulatory Guide 1.60 were scaled to 0.2g peak acceleration for the horizontal component and 0.13g for the vertical component and input as the base excitations. Modal responses and responses to horizontal and vertical excitations were both combined by the square root of the sum of the squares method.

A time-history method was used to generate in-structure response spectra for the interior structures. Only horizontal excitations were included in the analysis. The input base excitation was a synthetic time-history acceleration record for which the corresponding response

spectra were compatible with the 0.2g Regulatory Guide 1.60 spectra. Response spectra associated with two orthogonal horizontal base excitations were generated independently at equipment locations and then combined by the square root of the sum of the squares method. Peaks of the spectra were broadened ±15% in accordance with current practice.

3.8.3.6 Structural Acceptance Criteria

All Seismic Category I components, systems, and structures in the original design of Ginna Station were designed to meet the following criteria:

A. Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.08g peak horizontal and vertical ground accelerations, are maintained within the allowable working stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards (ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings).

B. Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.2g peak horizontal and vertical ground accelerations, are limited in such a way that the safe-shutdown function of the component, system, or structure is unimpaired.

For the SEP reevaluation the structural acceptance criteria was as stated in Section 3.8.2.2.6.

3.8.3.7 Structural Evaluation

Results from the reevaluation showed that the estimated seismic stresses of interior structures, including concrete shield walls, steel and concrete columns, and crane support structures, are low. No further evaluation was necessary.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

3.8.4.1 Description of the Structures

Seismic Category I structures, other than the containment and internal structures, are the following:

- Auxiliary building.
- Control building.
- Diesel-generator building.
- Intermediate building.
- Standby auxiliary feedwater building.
- Screen house (service water (SW) portion).

A complex of interconnected buildings surrounds the containment building (Figure 3.8-57). Though contiguous, these buildings are structurally independent of the containment building (Figure 3.8-45). However, several Seismic Category I structures are connected to nonseismic structures. The Seismic Category I auxiliary building is contiguous with the nonseismic service building on the west side. The Seismic Category I intermediate building adjoins the non-

seismic turbine building to the north, and the auxiliary building to the south. The turbine building adjoins the Seismic Category I diesel-generator building to the north and the Seismic Category I control building to the south. The facade, a cosmetic rectangular structure that encloses the containment building, has all four sides partly or totally in common with the auxiliary and intermediate buildings.

3.8.4.1.1 Auxiliary Building

The auxiliary building is a three-story rectangular structure, 70 ft 9 in. by 214 ft 5 in. It is located south of the containment and intermediate buildings and adjacent to the service building. The structure has a concrete basement floor that rests on a sandstone foundation at elevation 235 ft 8 in., and two concrete floors--an intermediate floor at elevation 253 ft and an operating floor at elevation 271 ft. The floors have a minimum thickness of 1.5 ft, and are supported by 2.5-ft thick concrete walls at the south, east, and part of the north sides of the building. The northwest corner of the building is adjacent to the circular wall of the containment building. The west concrete wall, which separates the auxiliary building and the spent fuel storage pool, is 6 ft thick.

The spent fuel storage pool is a rectangular swimming-pool-type concrete structure. Its bottom is at elevation 236 ft 8 in. Walls are 6-ft thick at the north and west sides and 3-ft thick at the east and south sides, which are below the ground surface and also serve as retaining walls.

The auxiliary building has two roofs constructed of steel truss and bracing systems and supported by frame bracing systems. The high roof (elevation 328 ft) covers the west part of the operating floor and the spent fuel storage pool. The low roof (elevation 312 ft) covers the east part of the operating floor. Insulated siding is used for the wall above the operating floor.

A platform that supports a component cooling surge tank and a heat exchanger rises from the operating floor to elevation 281.5 ft. The platform is supported by columns and bracings. There are also a number of 2.5-ft to 3.5-ft thick concrete shield walls on the floors.

The bottom elevation of the foundation mat is 233 ft 8 in., with the deepest foundation for the decay heat removal area at elevation 217 ft 0 in. with a sump at elevation 214 ft 0 in. Rock elevation in this area is at approximately elevation 236 ft 0 in. The west end of the superstructure of the auxiliary building is connected with a portion of the service building and on the northwest with the intermediate building. However, the foundation of the auxiliary building is independent of these building foundations.

3.8.4.1.2 Control Building

The control building is located adjacent to the south side of the turbine building and is a 41-ft 11-3/4 in. by 54-ft 1-3/4-in. three-story structure with concrete foundation mat at elevation 253 ft. The foundation of the control building is supported on lean concrete or compacted backfill. The rock elevation in this area is at approximately elevation 240 ft 0 in. The foundation of the control building was excavated to the surface of the bedrock. The fill material was placed on the rock surface to a depth coincident with the control building foundation.

The bottom elevation of the deepest portion of the foundation mat is at elevation 245 ft 4 in., with a structural slab supported at elevation 250 ft 6 in. with a thickened slab for column footings. The common wall is reinforced with structural members, stiffeners, and siding to form a pressurization wall or "superwall." The portion of the common wall above elevation 289 ft 6 in. to the roof has 1/4 in. armor plate. The south and west sides have reinforced-concrete walls, and the roof is also reinforced concrete. The control room floor at elevation 289.75 ft and the relay room floor at elevation 271 ft are 6-in. thick reinforced-concrete slabs supported by steel girders that are tied to turbine building floors at the respective elevations. The basement is the battery room. The east wall of the control room, from elevation 289 ft 6 in. to the roof, has 1/4 in. armor plate covered by insulated siding. The relay room east wall is primarily insulated siding and some concrete block. The east wall has been modified during the Structural Upgrade Program to withstand the effects of tornado wind, tornado differential pressure, tornado missiles, and flooding of Deer Creek. The modification consists of a reinforced-concrete Seismic Category I structure adjoining the east wall of the relay room (see Section 3.3.3.3.6). The battery room is below grade.

3.8.4.1.3 Diesel Generator Building

The diesel generator building is a one-story reinforced-concrete structure that has two cable vaults underneath the floor. The south wall, which is common with the turbine building, is reinforced to be a pressurization wall like the one described above in Section 3.8.4.1.2. The building roof has a built-up roof supported by four shear walls that sit on concrete spread footings.

The diesel generator building was modified as part of the Structural Upgrade Program to withstand tornado winds and missiles, external flooding, seismic loads, and extreme snow loads. A new reinforced-concrete north wall was constructed 4 ft north of the existing north wall. Reinforced-concrete wing walls were constructed that extended the east and west walls to meet the new north wall, enclosing the space between the existing and new north wall. The new wall includes missile-resistant watertight equipment and personnel doors. A new reinforced-concrete slab roof with a reinforced-concrete parapet was constructed covering the entire diesel generator building. The existing north wall and portions of the existing roof were left in place. The building as modified was designed to remain undamaged during and after an operating basis earthquake and remain functional during and after a safe shutdown earthquake.

3.8.4.1.4 Intermediate Building

The intermediate building is located on the north and west sides of the containment building, and is founded on rock. The west end has a retaining wall where the floor at elevation 253 ft 6 in. is supported. The bottom of the retaining wall footing is at elevation 233 ft 6 in. Rock elevation in this area is at approximately elevation 239 ft 0 in. Foundations for interior columns are on individual column footings and embedded a minimum of 2 ft in solid rock. The building, which also encloses the cylindrical containment building, is north of the auxiliary building and is connected to the part of the auxiliary building that is under the high roof.

The building is a 136-ft 7-in. by 140-ft 11-in. steel frame structure with facade structures on each side. The facade structures are steel frame bracing systems covered with shadowall aluminum sidings. The concrete basement floor slab (elevation 253.5 ft) is supported by a set of 2-ft 10-in. square concrete columns and a concrete retaining wall on the west side. The

columns have individual concrete footings embedded in the rock foundation. The top elevations of the footings vary from 238 ft to 236.5 ft.

In the north part of the building, there are three floors at elevations 278.33 ft, 298.33 ft, and 315.33 ft, and a high roof at elevation 335.5 ft. In the south part of the building there are two floors at elevations 271 ft and 293 ft, and the low roof at elevation 318 ft. All floors are made of composite steel girders and 5-in. thick concrete slabs. Built around the circular containment building, the floors extend completely through the west side of the intermediate building, a major portion of the north side and a small portion of the south side. There are no floors on the east side. The roofs are supported by steel roof girders. The floors and roofs are also supported vertically on a set of interior steel columns which are continuous from the basement floor to the roof. Concrete block walls surround all the floor space between the basement floor and the roofs.

The top of the four facade structures is at elevation 387 ft. There is no roof at the top, only a horizontal truss connecting the four sides to provide out-of-plane strength. One special characteristic of the west facade is that the horizontal floor or roof girders are connected not to the bracing joints but somewhere between joints. In such a design, the columns must transform significant shears and moments when the structure is subject to lateral loads.

3.8.4.1.5 Standby Auxiliary Feedwater Building

The standby auxiliary feedwater building is a reinforced-concrete Seismic Category I structure with reinforced-concrete walls, roof, and base mat. The building is supported by 12 caissons which are socketed into competent rock.

The building was analyzed to obtain the seismic response to three simultaneous, independent, mutually perpendicular acceleration time-histories which enveloped the response spectrum of Regulatory Guide 1.60. The analysis considered soil/caisson interaction and soil liquefaction potentials. Equivalent seismic forces obtained from the analysis were distributed through the reinforced-concrete structure in proportion to the stiffness of the structural elements.

3.8.4.1.6 Screen House

The screen house-service water (SW) building is comprised of two superstructures, one for the service water (SW) system and one for the circulating water system (the screen house portion). The service water (SW) portion of the building (both below and above grade) is a Seismic Category I structure.

The service water (SW) portion houses four Seismic Category I service water (SW) pumps and Seismic Category I electric switchgear. The screen house portion houses the traveling water screens and circulating water pumps.

The entire screen house-service water (SW) building is founded in or on bedrock with the exception of the basement of the electric switchgear portion which is founded approximately 4 ft above bedrock. Since the building is founded in bedrock the basement will not realize any spectral acceleration and the seismic loading is equivalent to the ground motion of 0.08g and 0.20g.

The basement is designed to be dewatered. The full height of the wall is designed for an external hydrostatic pressure plus a seismic load equal to a percentage of the dead load of the wall and the hydrostatic pressure. For the portion of the wall below grade and above bedrock an active earth pressure based on a saturated soil weight is applied.

Internal walls, such as pump baffles and the wing walls between the traveling screens, were designed for a full height hydrostatic pressure on either side plus a seismic load due to the water movement during a seismic event.

The service water (SW) portion of the screen house consists of four rigid frame bents in the east-west direction with bracing for wind and seismic loads in the north-south direction. The roof system is designed as a horizontal truss to transmit horizontal seismic loads to the frame columns and through the bracing to the foundation.

3.8.4.1.7 Turbine Building

Even though the turbine building was not designed to be Seismic Category I, it is included in this section because of its connection to Seismic Category I structures.

The turbine building is a 257.5-ft by 124.5-ft rectangular building on the north side of the building complex. It has a concrete basement at elevation 253.5 ft, two concrete floors (a mezzanine floor at elevation 271 ft and an operating floor at elevation 289.5 ft). The roof includes a roof truss structure from elevation 342.66 ft to elevation 357 ft composed of top and bottom chords connected by vertical bracing. The roof and floors are supported by steel framing and bracing systems on all four sides of the building. The floors are also supported by additional interior framing at various locations under the floors.

Part of the south wall frame also serves as the north wall of the intermediate building. The north facade structure (from elevation 357 ft to elevation 387 ft) is actually on the top of the south frame of the turbine building. The west frame is the continuation of the west facade structure of the intermediate building. This west frame is also part of the service building. Except between buildings, the walls of the turbine building have insulated aluminum siding.

Inside the building and parallel to the south and north frames, there is an interior frame system supporting the crane from the basement elevation to elevation 330 ft. The crane frame is designed like the exterior frame system with vertical columns, horizontal beams, and cross bracing bolted to columns. Each interior column is welded to the corresponding exterior column at the joints and mid-points of columns by a series of girder connections.

The south frame of the turbine building is designed like the west facade structure of the intermediate building; that is, horizontal floor girders are connected to columns somewhere between joints.

3.8.4.1.8 Service Building

The service building is a nonseismic structure. It is included in this section because it is contiguous with Seismic Category I structures.

The service building is located on the west side of the building complex. It extends from the south end of the auxiliary building, through the intermediate building, and ends a little before the north end of the turbine building. The building is a two-story steel structure with spread footings, steel columns, and concrete-steel framing floors and roof. The basement is at elevation 253.66 ft, the floor is at elevation 271 ft, and the roof is at elevation 287.33 ft. The walls between the service building and the other buildings as well as the partitions in the building are made of concrete blocks.

3.8.4.1.9 Interconnected Building Complex

The auxiliary, intermediate, control, screen house, standby auxiliary feedwater, and diesel generator buildings are Seismic Category I structures, and the turbine and service buildings are nonseismic category structures (see Figure 3.8-57). In the original analysis, each Seismic Category I structure was treated independently. For the SEP reevaluation it was found that the interconnected nature of the buildings was an important feature, especially in view of the lack of detailed original seismic design information. Therefore, both Seismic Category I and nonseismic category buildings were included in the reanalysis model.

The auxiliary, intermediate, turbine, control, diesel-generator, and service buildings form an interconnected U-shaped building complex (Figure 3.8-58) that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which make the building complex a complicated three-dimensional structural system.

3.8.4.1.10 Canister Preparation Building (CPB)

The CPB superstructure is designed to meet the applicable requirements of 10 CFR 50, the Ginna UFSAR, and the Building Code of New York State. The CPB superstructure is a seismic II/I structure such that the building cannot adversely impact the transfer cask, Auxiliary Building, and DSC when fuel is present.

The CPB superstructure is designed to transmit full tornado loading without differential pressure to primary members. The secondary members, purlins and girts are allowed to deform and yield locally such that there is no loss of function. Under direct tornado wind loads there may be local building cladding failures by this design and therefore tornado differential pressure is not a design load condition due to venting made available as a result of these localized failures.

The CPB Building Overhead Crane shall be supported to Seismic II/I criteria without consideration for live loads.

The CPB finished floor elevation is 269' - 2" to permit movement of the transporter into the facility from existing grade elevations and allow the 125-Ton crane to lift the transfer cask clear of the transporter trunnion supports.

3.8.4.2 Applicable Codes, Standards, and Specifications

The structural codes governing the original design of major Seismic Category I structures for Ginna Station and the corresponding currently applicable codes are listed in Section 3.8.2.1.

The impact of the code changes was evaluated in *Reference 25* (see Section 3.8.2.1). Several elements and regions were identified in the Seismic Category I structures that needed reevaluation. Additional analyses were performed (*Reference 30*) to determine the acceptability of the structures. The summary of these results is presented in Section 3.8.2.1.2.

3.8.4.3 Loads and Load Combinations

The loads and load combinations used in the original design of Ginna Station, the currently applicable loads and load combinations, and a comparative evaluation of these two sets were studied by the Franklin Research Center (*Reference 25*). The loads and load combinations that were not considered in the original design but had a potential effect on the structural acceptability were identified and additional analyses were performed to evaluate these changes and the results were reported in *References 27* and *29* (see also Section 3.8.2.1.2).

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Original Design and Analysis Procedures

A brief description of the dynamic analysis performed for the original design of Ginna Station is in the following.

Auxiliary Building

The steel superstructure above elevation 271 ft of the auxiliary building was evaluated for equivalent horizontal seismic loads based upon either the maximum spectral response or the spectrum value corresponding to the first harmonic frequency of the structure. This superstructure was designed (*Reference 46*) originally to withstand a wind loading of 18 lb/ft^2.

Control Building

The original seismic design of the control building was based on the operating-basis earthquake as follows:

Structural steel columns were designed for flexural moments resulting from a horizontal load equivalent to 10% of the axial load applied at the mid-span of the column.

Concrete walls above grade were subjected to a horizontal reaction normal to the wall and applied at mid-span. The wall was treated as a fixed-base cantilevered beam. The equivalent seismic load was 10% of the wall weight.

Intermediate Building

The bracing system of the intermediate building is common to the turbine, service, and auxiliary buildings and the facade structure. The bracing was checked to demonstrate that it could resist equivalent seismic load components from the above structures.

Diesel-Generator Building

The diesel-generator building has concrete shear walls and steel-framed roof structures. The seismic design of the concrete shear walls considered both in-plane and normal equivalent static loads. Seismic accelerations were taken as the peak of the seismic response spectra for 5% of critical damping. The steel roof framing was designed for a horizontal equivalent safe shutdown earthquake seismic load, taken as the mass of the roof structure and superimposed loads times the peak seismic response for 2.5% damping. Column foundations were designed for an additional 20% of axial load to account for seismic effects.

Turbine Building and Service Building

The turbine and service buildings are nonseismic structures that are connected to Seismic Category I structures. For purposes of the original seismic design, coupling between the two classes of structures was not considered.

3.8.4.4.2 SEP Reevaluation Design and Analysis Procedures

The seismic design input for the SEP reevaluation of the Seismic Category I structures are described in Section 3.7. The seismic analyses of these structures performed by Lawrence Livermore National Laboratory for SEP reevaluation were as follows:

3.8.4.4.2.1 Mathematical Model

In the original analysis, each Seismic Category I structure was treated independently. Because of the interconnected nature of the buildings the SEP reevaluation included the entire building complex in the reanalysis model.

The auxiliary, intermediate, turbine, control, diesel-generator, and service buildings form an interconnected U-shaped building complex (Figure 3.8-58) that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which makes the building complex a complicated three-dimensional structural system. The compositions and interrelationships of the buildings in the complex are described in Appendix C to *Reference 30*.

The principal lateral force-resisting systems of the interconnected building complex are the braced frames. Several such systems tie all buildings together to act as one three-dimensional structural system. It was, therefore, necessary to model these buildings in a single three-dimensional model to properly simulate interaction effects. The model was developed based on the following assumptions.

Rigid foundation.

All buildings are founded on solid sandstone rock or on lean concrete or compacted backfill over rock and are assumed to have rigid foundations; thus, no soil-structure interaction effects are considered.

Uncoupled horizontal and vertical responses

There is no coupling between horizontal and vertical responses (i.e., only horizontal responses result from horizontal loadings and only vertical responses from vertical loadings). This is a reasonable assumption for this type of medium-height building that has regular frames and doors.

Only horizontal ground motion in the dynamic analysis.

For the dynamic analysis, the mathematical model was designed to have only horizontal responses because the major concern is the capacity of the lateral force-resisting system. Vertical response was calculated assuming no dynamic amplification. Because the structures were originally designed for vertical loads, such as dead and live loads, they are relatively stiff in the vertical direction and in most cases, are not considered to have significant dynamic amplification during vertical excitation. It is not necessary to simulate both vertical and horizontal behavior simultaneously.

Rigid floors and roofs.

All floors and roofs were assumed to be rigid in-plane because of the high stiffness for horizontal loads of the in-plane steel girders and concrete slabs. Each floor or roof has three degrees of freedom: two in horizontal translation and one in vertical (torsional) rotation. All points on a floor or roof were assumed to move as a rigid body. The center of gravity of each rigid floor or roof was selected as the representative node.

Lumped masses.

All structural and equipment masses were assumed to be lumped at the floor or roof elevations, then transformed to the centers of gravity of each rigid floor or roof.

Hinge connections.

Most bolted joints that connect bracing and beams to columns (and columns to base supports) were treated as pin or hinge connections based on reviews of pertinent drawings. The few exceptions are described in the discussion of the model for each building.

Buckled and unbuckled bracing systems.

Cross-bracing members, which are the primary elements of the lateral load-resisting system, are expected to buckle during compression cycles because of their large slenderness ratios. After a member buckles, it has zero or very small stiffness, but regains its capacity under tension. Such nonlinear behavior was approximately accounted for by considering two linear models: a half-area model that simulates buckled bracing and a full-area model that simulates unbuckled bracing.

In the half-area model, it was assumed that both cross-bracing members have only half the actual member cross-sectional area and can take both compression and tension during earthquake excitation. The full-area model was based on the assumption that bracings with the full cross-sectional area are effective in both compression and tension.

Stick model for concrete wall structures.

The control building, which has concrete walls and roof that are much stiffer than the other structures, was modeled as an equivalent beam. The two-story concrete substructure in the basement of the auxiliary building was treated similarly.

Stiffness and mass effects of the diesel-generator and service buildings.

The one-story diesel-generator building has four shear walls that have significant stiffness but minimal mass (only the roof mass needs to be considered; the other masses are on the rigid foundation). Therefore, the four shear walls were modeled as four elastic springs having the equivalent stiffness of the shear walls. The service building is a relatively flexible steel frame structure, and only its mass was included.

Damping.

A uniform damping of 10% of critical was assumed for the whole structural system based on the suggestion of NUREG/CR-0098 for bolt-connected steel structures under safe-shutdown earthquake loading.

The three-dimensional mathematical model for the building complex was prepared for the computer program SAP4 (*Reference 47*). All steel frames were modeled by beam elements. The model rigid diaphragms for all roofs and floors were represented by the rigid restraint option of SAP4. There are 17 such rigid diaphragms in the model that were treated this way.

The two-story concrete substructure of the auxiliary building and the control building were modeled by equivalent beams. The four shear walls of the diesel-generator building were represented by four elastic springs attached to the north frame of the turbine building at the diesel-generator building roof. The masses of the service building roof were lumped to the turbine and intermediate buildings. All other masses were lumped to the centers of gravity of floor or roofs.

The complete model had 686 nodal points, 44 dynamic degrees of freedom, 1213 beam elements, and 10 elastic springs.

3.8.4.4.2.2 Method of Analysis

Figure 3.8-59 is a flow chart of the analytical procedure. The frequencies and mode shapes of the structural system were obtained by the subspace iteration method provided in SAP IV.

After the frequencies and mode shapes were obtained, the structural responses were computed by the response spectrum method. The seismic input was defined by the horizontal spectral curve of the safe shutdown earthquake specified in Regulatory Guide 1.60 for 10% structural damping and 0.2g peak ground acceleration.

Two structural models were analyzed, one with half the bracing area (half-area model) and one with the full bracing area (full-area model). For each model, two analyses were performed, one with the input excitation in the north-south direction, the other in the east-west direction. In each analysis and for each direction the modal responses were combined by the square root of the sum of the squares method. Responses to the north-south and east-west

excitations were also combined by the square root of the sum of the squares method. Vertical responses were obtained by taking 13% (0.2g x 2/3) of the dead load responses.

3.8.4.4.2.3 Structural Evaluation

Auxiliary Building

Based on the stresses calculated in the reanalysis, the concrete structure has adequate load margins to withstand seismic loads. However, the braced steel frames of the superstructure are more critical. The bracings in the east-west direction have stresses below yield, but the north-south bracings are near or exceed yield. The bracing at the northeast corner of the low roof has a safety factor (defined as f_y/f) of about 0.8. Alone this may be considered marginal, but this bracing is one of only two lateral load-resisting systems for the auxiliary building superstructure in the north-south direction. The other one is the bracing between the high and low roofs, and its stress is close to yield. Consequently, RG&E upgraded this bracing on the auxiliary building east wall as part of the Structural Upgrade Program.

Intermediate Building and Facade Structures

The braced frames in the low portion of the east and west facades are the relatively weak areas of the intermediate building and facade structures. The stresses in the cross bracings are at or a little over yield (safety factor of 0.9). The lateral load-resisting systems have more reserve capacity than do the braced steel frames of the auxiliary building discussed above. The vertical columns of the floors and nonstructural members, such as stairway structures between floors and sidings, provide additional lateral support to the structure.

The reanalysis indicated that the columns supporting intermediate floors may yield locally at locations where floors at different elevations meet at mid-points between joints. However, those columns still have sufficient moment-resisting capacity, and the column systems can be considered acceptable.

Turbine Building

The Lawrence Livermore National Laboratory evaluation concluded that the lateral load-resisting system for turbine building floors had stresses below yield. The cross-bracings above the operating floor in the south, north, and west walls had stresses that exceed yield. The bracings right above the control building superwall had the lowest safety factor (0.7). These bracings sustain high loads because of the relatively high stiffness of the superwall and the control building compared to the turbine building frames. Consequently, RG&E upgraded this bracing on the turbine building south wall as part of the Structural Upgrade Program.

Control Building

Excluding stress concentration effects, the maximum shear stress in the reinforced-concrete walls of the control building is approximately 200 psi.

Because the walls have No. 5 reinforcing steel bars (5/8-in. diameter) at 12-in. spacing (in both horizontal and vertical directions), the structure is considered to be adequate for resisting shear.

3.8.4.5 Masonry Walls

As a result of IE Bulletin 80-11, Masonry Wall Design, RG&E identified the masonry walls at Ginna Station that were considered to be safety-related. Through a series of analyses a number of masonry walls were determined to be able to withstand all applicable loads and load combinations. Other masonry walls were qualified based on providing restraining modifications or safety-related equipment protection.

3.8.4.5.1 Applicable Walls

The masonry walls in the structures considered under this section were surveyed to determine if their failure could damage any safety-related systems, equipment, and attachments.

Figures 3.8-60 through 3.8-62 illustrate the location of the 37 masonry walls that are considered safety-related, i.e., whose potential failure must not endanger safe shutdown capability. The presence of a safety-related system or component within one wall height of these walls is sufficient to qualify the wall as safety-related. The 37 walls contain 56 panels, a panel being a wall division isolated for engineering analysis.

Twelve of the 37 safety-related walls are reinforced vertically. Of this total, seven are reinforced with one #3 bar on 32-in. centers. The remaining five are reinforced with two #3 bars on 16-in. centers. The joint reinforcement is DUR-O-WALL standard truss type on 8-in. centers or DUR-O-WALL "extra heavy" truss type on 16-in. centers. All except one safety-related masonry block walls are running bond masonry walls. One of the walls is composed of interlocking lead bricks.

3.8.4.5.2 Loads and Load Combinations

The walls were reevaluated for the following loads and load combinations.

Loads

- Wind load.
- Seismic accelerations.
- Dead loads.
- Ambient temperature differentials.

Load Combinations

- $D + (1.5\ P + 1.0\ T)^a$
- $D + (1.25\ P + 1.0\ \underline{T})^a + 1.25\ W$

---

a. Accident pressure and temperature loads will be considered only inside containment when wall configurations make differentials a possibility. No safety-related masonry walls satisfy this condition.

- $D + (1.0\ P + 1.0\ T)^a + 1.0\ E'$

Symbols used in the above equations are defined as follows:

D = Dead load of structure (a value of D ± 0.05 shall be used where it produces maximum stress)

P = Accident pressure load

T = Thermal loads based upon temperature transient associated with 1.5 times accident pressure

T′ = Thermal loads based upon temperature transient associated with 1.25 times accident pressure

$\underline{T}$ = Thermal loads based upon temperature transient associated with accident pressure

E′ = Safe shutdown earthquake load

W = Wind load

Wind Loads

Height Above Ground (ft)	**Pressure Load (psf)**
0-15	12
6-25	15
26-40	18
41-60	21
61-100	24
101-200	28

3.8.4.5.3 Stress Analysis

3.8.4.5.3.1 Computer Program

The computer program SAP 4 was used to calculate stresses. Wall geometry, boundary support conditions, material and physical properties, attachment loads, and response spectra information were input into the SAP 4 program. The program then performed static and dynamic analyses to determine stresses in the walls for the various load combinations.

The stresses determined by the SAP 4 program were then compared to allowable stresses using a special purpose post-processor program designed to combine stresses obtained from the static and dynamic analysis of the SAP 4 program and compare the resultant stresses against allowable values.

The analysis uses linear working stress principles. The uncracked moment of inertia is based on the unreinforced section. The cracked moment of inertia is calculated by equating the moment of the transformed tensile steel area about the centroid axis of the cross-section to the moment of the masonry compressive area. Section stiffness is calculated using Branson's equation.

Boundary conditions used in the analysis are applied to each wall so as to reasonably resemble the actual physical conditions.

3.8.4.5.3.2 Seismic Analysis

Seismic analysis of the Safety-related masonry walls was performed for the following three levels.

Level 1 Safe Shutdown Earthquake (0.2g SSE)

With Appendix A to Standard Review Plan 3.8.4 acceptance criteria.

Level 2 Safe Shutdown Earthquake (0.17g SSE)

(Site-specific SEP earthquake)

With Appendix A to SRP 3.8.4 acceptance criteria.

Level 3

Level 2 analysis with the exception that a 1.5 overstress factor for tension normal to the bed joint is used instead of the SRP value of 1.3 as acceptance criteria.

Seismic analysis was performed using the response spectrum method. Response spectra for the analyses were based on averaging the floor response spectra for the top and bottom elevations if the wall is supported at both locations. Otherwise, the floor response spectrum at the base of the wall is used. Response spectra were broadened by 15% to account for uncertainties in the analytical model compared with the physical structure. The assumed damping value of 7% is consistent with Appendix A to SRP 3.8.4.

The analysis takes into account the combined effects of all modes of vibration up to 33 Hz, which corresponds to the rigid range of the floor response spectra. For walls whose frequencies are greater than 33 Hz, the floor response accelerations at 33 Hz were used for the analysis.

Three directions of earthquake were considered in the analysis by evaluating walls for both vertical plus out-of-plane and vertical plus in-plane load combinations. The vertical plus out-of-plane load combination was found to be the limiting load case in the analysis.

3.8.4.5.4 Interstory Drift

In-plane strain criteria used to verify the adequacy of the walls is discussed in "Recommended Guidelines for the Reassessment of Safety-Related Concrete Masonry Walls," prepared by the Owners and Engineering Informal Group on Concrete Masonry Walls, October 6, 1980. The acceptance criteria are based upon an uncoupled system (separate treatment of in-plane and out-of-plane loads). Evaluations indicate that the in-plane strains induced on the walls due to interstory drift are less than the allowables permitted in the majority of instances, regardless of whether a mechanism exists to induce the drift into the walls. In the remaining instances, the implied strains would exceed the acceptance criteria if a positive transfer mechanism existed. For these later instances, a specific case-by-case review was conducted of the wall configuration with respect to the surrounding structure, displacements, and drift inducement mechanics. From this review, it was judged that a sufficient mechanism does not exist to induce significant interstory in-plane drift. Masonry walls at Ginna are not relied upon to provide horizontal shear load resistance (i.e., shear walls). Out-of-plane interstory drift has no significant effect on the walls since they can be considered simply supported between stories.

3.8.4.5.5 Multi-Wythe Walls

There are no safety-related multi-wythe or brick masonry walls.

3.8.4.5.6 Block Pullout

The attachments to the walls are typically made with 3/8-in. drilled anchors. Calculations of the forces on an 8-in. nominal block, which would result from two such anchors located symmetrically and nonsymmetrically, were made. Treating the block as a rigid body, forces necessary to provide equilibrium were calculated. The applied forces resulted in bearing and shear stresses at the perimeters of the loaded block, which were not sufficient to pull the block from the remainder of the wall.

3.8.4.5.7 Structural Acceptance Criteria - Allowable Stresses

3.8.4.5.7.1 Normal Operating Conditions

For normal operating conditions, allowable masonry working stress values are as specified in ACI 531-79. The allowable stresses are based on compressive strength of 700 psi on the gross area of the block. The value of m_o, the specified 28-day compressive strength of the mortar per ASTM C-270, is 750 psi.

3.8.4.5.7.2 Safe Shutdown Earthquake

The increase factors permitted by SRP 3.8.4 for load combinations containing SSE loads were used for evaluation with one exception. For tension normal to the bed joint, an increase factor of 1.5 versus 1.3 was used to qualify two walls. The 1.3 factor is exceeded by 10% for wall 3-17A-5 and 7% for wall 2-2I. This corresponds to increase factors of 1.43 and 1.38, based on the actual wall stresses, rather than 1.5. The allowable stresses identified in ACI 531 include a safety factor of 3. Therefore, the use of 1.43 and 1.38 as increase factors still provides margins of safety of 2.10 and 2.17 for the two walls and is judged to be acceptable for these limited cases.

<u>3.8.4.5.8 Evaluation Results</u>

3.8.4.5.8.1 General

All masonry block walls at Ginna Station were inspected and found to be built in accordance with the original specifications and with appropriate inspection and construction techniques applicable at the time of construction. See Section 3.8.4.5.9.

Of the 56 safety-related panels, the modifications installed after the original IE Bulletin 80-11 evaluation resulted in 29 panels meeting current stress criteria.

In the analysis no credit was taken for either horizontal or vertical reinforcing. Of the 27 panels that required modification or further analysis, twelve contain vertical reinforcing and horizontal DUR-O-WALL joint reinforcement.

As noted in Section 3.8.4.5.1, one safety-related wall, 971-2M, is composed of 4-in. interlocking lead bricks. The wall, 2 ft 3 in. wide at the base and 5 ft 4 in. high, was analyzed taking no credit for the interlocking effect of the brick. The steel framing network surrounding the wall can adequately restrain the wall in one direction during an earthquake, and wall failure in the other direction will not affect any safety-related equipment. Wall 971-2M is therefore seismically acceptable. Thus, 26 panels remained for further analysis or modification.

A cracked section analysis was performed on one wall panel. Due to the minimum reinforcing available in the evaluated panel, no significant benefit was gained from the cracked section analysis. No walls have been qualified using cracked section analysis.

A seismic analysis of the 12 safety-related reinforced masonry block wall panels in the control building was conducted as documented in *Reference 48*. The methodology used to evaluate the walls in the inelastic range was previously used on the masonry walls at the San Onofre Nuclear Generating Station Unit 1 (SONGS 1). Correlation of this methodology to Ginna Station was confirmed by *Reference 49*.

From the elastic analysis, the seven spanning walls had stresses in the vertical rebar exceeding the criteria limit of 36 ksi by ratios ranging from 1.25 to 2.18. Therefore, all walls required qualification by the inelastic analysis methodology as discussed below.

3.8.4.5.8.2 Inelastic Analysis

Spanning walls 971-1C and 971-6C and cantilever wall 973-4C were analyzed in detail. Spanning wall 971-1C is a 16 ft 10 in. high wall 38 ft 1 in. long between elevations 253 ft 8 in. and 271 ft 0 in. in the control building. It is reinforced with #3 bars at 32-in. centers and horizontally with DUR-O-WALL joint reinforcing. Spanning wall 971-6C is similar in construction and at the same elevation. Cantilever wall 973-4C has two layers of vertical rebars rather than being centrally reinforced as for the spanning walls.

The two walls were chosen because they represent the highest and lowest levels of overstress, thus enabling results for the other walls to be obtained by interpolation. The results of the two chosen walls indicated strains well within the criteria limits of masonry strain $E_m = 0.003$ and vertical steel strain ratio of $E_s/E_y = 45$.

With the interpolation of the result of the inelastic analysis of walls 971-1C and 971-6C, it was concluded that the remaining spanning walls will have similarly low material-strain ratios. Based on this it is considered that all spanning walls will perform satisfactorily under SSE loading with degrees of nonlinearity well within the capability of reinforced masonry.

The detailed model of the cantilever wall 973-4C was used for the nonlinear analysis. The results of the time histories showed that the masonry and steel strain ratios were well within the criteria limits.

Based on these analyses it is concluded that the reinforced masonry walls have ample ductility to resist the design SSE input motions.

3.8.4.5.8.3 Wall Modifications

For the remaining 14 wall panels, RG&E used the following methods to ensure wall qualifications:

a. A wall was considered safety-related if equipment was located within one full wall height of the base of the wall. Rochester Gas and Electric Corporation investigated the justification of using less than one full wall height, if applicable, on a wall-by-wall basis. If it were concluded that the collapse mechanism is such that the equipment is not hit, no further evaluation would be performed.

b. If a wall failure could impact safety-related equipment, additional analysis would be performed to determine if the equipment would actually be damaged and inoperable. If the equipment could withstand the wall impact and remain operable, no modification would be performed.

c. Modifications to protect safety-related equipment potentially impacted by wall failure would be designed and installed so that wall failure has no safety consequences.

d. Wall modifications would be designed and installed such that the wall would meet the evaluation criteria.

The NRC evaluated RG&E’s response to IE Bulletin 80-11, regarding masonry wall design adequacy and the commitments for the 14 wall panels requiring additional analysis or

modification, and determined that RG&E has adequately addressed the concerns of IE Bulletin 80-11 (*Reference 50*).

The 14 wall panels have been qualified either by structural modifications to the panel to meet the evaluation criteria or by protection of the safety-related equipment subject to impact. Protective structures have been installed to protect the A and B main steam isolation valve operators and solenoid valves and the auxiliary feedwater check valves that were subject to impact by wall failure. The main steam isolation valve control cables have been rerouted so as not to be susceptible to damage from failed walls.

3.8.4.5.9 Materials, Quality Control, and Special Construction Techniques

The original Ginna Station project specifications identified the materials to be used for the construction of masonry walls as follows.

A. Concrete: ACI 318-63.

B. Steel: ASME Section III, Article CC-2000.

C. Brick: Facing brick shall conform to the requirements of ASTM Specifications C 216-65, Grade SW and Type FBS.

D. Concrete masonry units: Hollow, load-bearing units shall conform to ASTM C 90-665, Grade G-11. Interior non-load-bearing partitions shall be Haydite block.

E. Concrete masonry bed reinforcing: Reinforcing shall be Dur-O-Wall standard, truss design, or Hohmann & Barnard, Inc., Turs-Mesh, with a width 2 in. less than the nominal thickness of the wall. Reinforcing in exterior walls shall be galvanized in accordance with ASTM A 116-65, Class 1, specifications. Installation shall be in strict accordance with the manufacturer's recommendations.

F. Partition ties: 1-1/4 in. x 1/4 in. x 8 in. with 2-in. right-angle bends at either end, prime painted with 13-Y-5 zinc chromate primer as made by Mobil Chemical Company, Metuchen, New Jersey, or approved equivalent.

G. Anchors at columns: Anchors will be provided by others at 24-in. centers.

H. Control joints: Dur-O-Wall, wide flange, Rapid Control Joint.

I. Mortar:

   a. Mortar and mortar materials shall conform to the requirements of the property specifications of ASTM Specifications for Mortar for Unit Masonry C 270-64T, Type N.

      1. **Portland cement**: ASTM C 150-66, Type I or II.
      2. **Hydrated lime**: ASTM C 207-49, Type S, or Miracle Lime as made by G. & W. H. Corson, Plymouth Meeting, Pennsylvania.
      3. **Sand**: ASTM C 144-66T.
      4. **Water**: Water shall be clean and free of deleterious amounts of acids, alkali, or organic materials.
      5. **Mixing**: Mixing shall be done in a mechanical batch mixer. No more mortar shall be mixed at one time than can be used within 1.5 hours.

6. **Admixtures**: Salts and antifreeze compounds to lower the freezing point of mortar will not be permitted.

b. At the subcontractor's option, a prepared mortar may be used conforming to ASTM Specification C 91-66, Type II.

3.8.5 FOUNDATIONS

The foundations of the interior containment structures, the auxiliary building, the screen house, and the intermediate building are founded on the bedrock of the Queenston formation, which is exhibited to be strong and fresh layers of shale, sandstone, and siltstone in the boring logs. The turbine building, control building, and the diesel generator building foundations were excavated and provided with lean concrete on compacted backfill to a depth whereby the elevation of the top of the fill material was coincident with the elevation of the bottom of the concrete foundation of that particular building.

The standby auxiliary feedwater building is on pilings to the bedrock. The technical support center is on the second floor of the all-volatile-treatment building, which is founded on a concrete mat.

The (CPB) foundation was designed as a Seismic Category I structure. In order to assure that suitable soil exists that is capable of supporting the CPB superstructure and the 125 ton crane an investigation revealed and determined that drilled caissons would be needed to prevent building displacement as the 125 ton crane tranverses into and out of the Aux building. The caissons also act to limit the horizontal and vertical seismic motions exerted during a postulated seismic event. The drilled caissons were installed into the bedrock beneath the CPB / crane foundations.

The major structures of Ginna Station have experienced no visible evidence of settlement since the construction of the station. During the SEP and evaluation of Topic II-4.F, the NRC concluded (*Reference 51*) that the settlement of foundations and buried equipment is not a safety concern for Ginna Station.

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Table 3.8-1a
COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS DIMENSIONS AND FORMULA

MEMBER DIMENSIONS

Each member is assumed to have uniform area with cross section as at the mid-point of the member

$$\text{Radius} = \sqrt{(R1^2 - h^2)}$$

(Equation Formula T-3.8-1)

$R_1^2 = 660^2 = 435{,}600$

$R_2^2 = 630^2 = 396{,}900$

Table 3.8-1b
COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS DIMENSION CALCULATIONS

Member[a]	h	h^2	$R1^2-h^2$	$R2^2-h^2$	External Radius	Internal Radius	Thickness
13-12	575	330,600	105,000	66,300	324.0	257.5	66.5
12-11	405	164,000	271,600	232,900	521.2	482.6	38.6
11-10	235	55,230	380,370	341,670	616.6	584.6	32.0
10-9	77	5,930	---	391,000	672	625.3	46.7
9-8					672		42.0
8-7					672		42.0
7-6					672		42.0
6-5					672		42.0
5-4					672		42.0
4-3					672		42.0
3-2					672		42.0
2-1					672		42.0

a See Figure 3.8-10

Table 3.8-1c
COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS NATURAL FREQUENCIES AND RESPONSE

				Response Accelerations (2% Damping)	
Mode	Frequency (Hz)	Period (Sec)	Modal Effective Mass (x 10^6)	0.08g	0.20g
1	6.95	0.144	18.46	0.14	0.36
2	19.19	0.052	4.78	0.09	0.22
3	34.44	0.029	0.30	0.08	0.20
4	38.01	0.026	0.92	0.08	0.20
5	54.63	0.018	0.51	0.08	0.20
6	64.82	0.015	0.05	0.08	0.20
		Total	25.02 x 10^6 lbs		

Table 3.8-2
MAJOR STRUCTURES FOR WHICH PRESTRESSED ROCK ANCHORS WERE USED

DAMS

Little Goose Lock & Dam

Snake River, Oregon, Washington and Idaho

Designed October 1963 by U. S. Army Engineers District

Walla Walla, Washington

Wanapum Hydro Station - Washington, 1962

Enestina Dam, Brazil - 1951-1954

Allt-Wa-Lairige Dam, Scotland - 1954-1956

Tourtemegne Dam, Switzerland - 1957-1958

Swallow Falls, South Africa - 1956-1958

Catagunya Dam, Tasmania - 1959-1961

Meadowbanks Dam, Tasmania - 1964

BRIDGES

Feather River Suspension Bridge

Oroville, California

Designed by California Department of Water Resources

TIE BACKS

Montreal Subway

Designed by Bealieu-Trudeau and Associates, Montreal

New York State's University Teaching Hospital in

Syracuse, New York

Designed by DiStasion and Van Buren

Washington Hilton Hotel

Designed by Wayman C. Wing

University of California

San Francisco Medical Center

Designed by Reid and Tarics

New York Life Insurance Company

New York City

Designed by Edwards and Hjorth

SPECIAL STRUCTURES

Test Facility for Saturn Rocket

Engines at Edwards Air Force Base

Designed by Corps of Engineers, Los Angeles

Research by Aero-Jet General Corporation

Table 3.8-3
PROPERTIES AND TESTS FOR CONTAINMENT ANCHOR AND TENDON CORROSION INHIBITOR

Physical Properties

Item	**Range**	**Method**
Specific gravity	0.88-0.90	ASTM D-287
Weight/gal	7.35-7.50 lb	---
Pour point	110°F-120 °F	ASTM D-97
Flash point (COC)	400 °F, Minimum	ASTM D-92
Viscosity at 130 °F	575-635 SSU	ASTM D-88
Viscosity at 150 °F	135-145 SSU	ASTM D-88
Viscosity at 210 °F	65-75 SSU	ASTM D-88
Penetration (cone) at 77 °F	328-367 Sec	ASTM D-937
Thermal conductivity	0.12 Btu/hr/ft^2/ °F/ft Thickness (approximate)	---
Specific heat (heat capacity)	0.51 Btu/lb/°F (approximate)	---
Shrinkage factor from 150 °F to 75 °F	3.5% - 4.5%	----
Accelerated Corrosion Test Results		
Humidity cabinet (JAN-H-792)	300 hr	ASTM D-1748-62T
Salt spray cabinet	75 hr	ASTM B-117-62 (Salt Fog Test)

Table 3.8-4
ALLOWABLE STRESSES

Load[a] Combination	Actual Maximum Tensile Stress (ksi)	Average Shear Stress Capability [b] (ksi)	Actual Average Shear Stress (ksi)	Ultimate Tensile Stress Capability [c] (ksi)
a	38.0	0	0	38.0
b	31.6	10.5	4.4	37.0
c	25.4	14.1	8.7	33.8

a. Load (a) C = 0.95 D + 1.5 P + 1.0 T
   Load (b) C = 0.95 D + 1.25 P + 1.0 T′ + 1.25 E
   Load (c) C = 0.95 D + 1.0 P + 1.0 $\underline{T}$ + 1.0 E′
b. For the given tensile stress.
c. For the given shear stress.

Table 3.8-5a
CONTAINMENT STRUCTURE STRESSES - LOADING #1 DEAD LOAD

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional $N\phi$	Hoop $N\theta$	Meridional $M\phi$	Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δ R
Base 0	-70.9	0	0	0	0	0
3	-69.4	0	0	0	0	0
6	-67.8	0	0	0	0	0
10	-65.2	0	0	0	0	0
15	-63.3	0	0	0	0	0
20	-60.5	0	0	0	0	0
30	-55.7	0	0	0	0	0
40	-50.5	0	0	0	0	0
60	-40.3	0	0	0	0	0
75	-32.7	0	0	0	0	0
85	-27.6	0	0	0	0	0
90	-25.0	0	0.0	0	0	0
95	-22.5	0	+20.0	0	0	0
Springline 99	-20.4	+20.4	+27.8	0	0	0
102	-19.4	+18.3	+31.0	0	0	0
105	-18.5	+16.2	+32.3	0	0	0
Dome Anchor 108	-17.5	+14.1	+31.0	0	0	0
-111	-16.8	+12.2	+26.5	0	0	0
111	-16.8	+12.2	+28.0	0	0	0
+111	-16.8	+12.2	-1.5	0	0	0
114	-16.1	+10.5	0.0	0	0	0
117	-15.4	+8.7	0.0	0	0	0
123	-14.3	+5.5	0.0	0	0	0
130	-13.2	+2.2	0.0	0	0	0
Apex	-10.2	-10.2	0.0	0.0	0.0	0.0

Table 3.8-5b
CONTAINMENT STRUCTURE STRESSES - LOADING #2 FINAL PRESTRESS - 636 K/ TENDON

$$N\phi = \frac{160x636}{108.5\pi} = 299k/in. -$$

Location in Feet Up From Base Element No.	Stress Resultants: Meridional Nϕ	Stress Resultants: Hoop Nθ	Stress Couples: Meridional Mϕ	Stress Couples: Hoop Mθ	Meridional Shear Vϕ	Radial Displacement δ R
Base 0	-299.0	0	0	0	0	0
3	-299.0	0	0	0	0	0
6	-299.0	0	0	0	0	0
10	-299.0	0	0	0	0	0
15	-299.0	0	0	0	0	0
20	-299.0	0	0	0	0	0
30	-299.0	0	0	0	0	0
40	-299.0	0	0	0	0	0
60	-299.0	0	0	0	0	0
75	-299.0	0	0	0	0	0
85	-299.0	0	0	0	0	0
90	-299.0	0	0	0	0	0
95	-299.0	0	0	0	0	0
Springline 99	-299.0	0	0	0	0	0
102	-299.0	0	0	0	0	0
105	-299.0	0	0	0	0	0
Dome Anchor 108	-299.0	0	0	0	0	0
-111	-299.0	0	0	0	0	0
111	-299.0	0	0	0	0	0
+111	0	0	0	0	0	0
114	0	0	0	0	0	0

$$N\phi = \frac{160x636}{108.5\pi} = 299k/in. -$$

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	**Meridional Nϕ**	**Hoop Nθ**	**Meridional Mϕ**	**Hoop Mθ**	**Meridional Shear Vϕ**	**Radial Displacement δ R**
117	0	0	0	0	0	0
123	0	0	0	0	0	0
130	0	0	0	0	0	0
Apex	0	0	0	0	0	0

Table 3.8-5c
CONTAINMENT STRUCTURE STRESSES - LOADING #3 OPERATING TEMPERATURE - WINTER

$N\theta$ = kry = 116.5 (651) 12 y/1000 = 912 y′c k/ft δr = -0.143

Location in Feet Up From Base Element No.	Stress Resultants: Meridional $N\phi$	Stress Resultants: Hoop $N\theta$	Stress Couples: Meridional $M\phi$	Stress Couples: Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δ R
Base 0	0.0	+130.2	0.0	99.5	-6.6	0.000
3	0.0	+95.6	-9.7	99.5	-0.3	.038
6	0.0	+65.0	-3.6	99.5	4.0	-.075
10	0.0	+27.3	+19.8	99.5	7.2	-.113
15	0.0	0.0	+59.7	99.5	8.4	-.143
20	0.0	-14.6	+100.2	99.5	7.6	-.159
30	0.0	-19.2	+160.4	99.5	4.3	-.164
40	0.0	+11.8	+188.0	99.5	1.5	-.156
60	0.0	0.0	+192.3	99.5	-0.4	-.144
75	0.0	0.0	+185.6	99.5	0.0	-.142
85	0.0	0.0	+186.0	99.5	0.0	-.143
90	0.0	+20.0	+149.1	99.5	+0.8	-.121
95	0.0	+34.6	+157.3	99.5	+3.1	-.105
Springline 99	0.0	+48.3	+173.8	+99.5	+5.9	-.090
102	0.0	-24.8	+28.1	28.2	-1.0	-.093
105	0.0	-12.1	+31.9	28.2	+1.0	-.072
Dome Anchor 108	0.0	-3.7	+31.8	28.2	+1.2	-.058
-111	0.0	0.0	+28.2	28.2	0.0	-.052
111	0.0	0.0	+28.2	28.2	0.0	-.052
+111	0.0	0.0	+28.2	28.2	0.0	-.052
114	0.0	0.0	+28.2	28.2	0.0	-.052
117	0.0	0.0	+28.2	28.2	0.0	-.052
123	0.0	0.0	+28.2	28.2	0.0	-.052
130	0.0	0.0	+28.2	28.2	0.0	-.052

$N\theta$ = kry = 116.5 (651) 12 y/1000 = 912 y′c k/ft δr = -0.143

Location in Feet Up From Base Element No.	Stress Resultants: Meridional $N\phi$	Stress Resultants: Hoop $N\theta$	Stress Couples: Meridional $M\phi$	Stress Couples: Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δ R
Apex	0.0	0.0	+28.2	28.2	0.0	-.052

Table 3.8-5d
CONTAINMENT STRUCTURE STRESSES - LOADING #4 OPERATING TEMPERATURE - SUMMER

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional $N\phi$	Hoop $N\theta$	Meridional $M\phi$	Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δR
Base 0	0.0	-130.2	0.0	0.0	+6.6	0.000
3	0.0	-38.3	+16.1	0.0	+4.2	+.101
6	0.0	-30.1	+25.9	0.0	+2.4	+.110
10	0.0	-19.1	+31.6	0.0	+0.6	+.122
15	0.0	-15.5	+30.9	0.0	-0.7	+.126
20	0.0	-2.7	+25.7	0.0	-1.3	+.140
30	0.0	+2.7	+12.5	0.0	-1.2	+.146
40	0.0	+2.7	+3.3	0.0	-0.6	+.146
60	0.0	0.0	-1.4	0.0	0.0	+.143
75	0.0	0.0	0.0	0.0	0.0	+.143
85	0.0	0.0	0.0	0.0	0.0	+.143
90	0.0	0.0	0.0	0.0	0.0	+.143
95	0.0	0.0	0.0	0.0	0.0	+.143
Springline 99	0.0	0.0	0.0	0.0	0.0	+.143
102	0.0	0.0	0.0	0.0	0.0	+.143
105	0.0	0.0	0.0	0.0	0.0	+.143
Dome Anchor 108	0.0	0.0	0.0	0.0	0.0	+.143
-111	0.0	0.0	0.0	0.0	0.0	+.143
111	0.0	0.0	0.0	0.0	0.0	+.143
+111	0.0	0.0	0.0	0.0	0.0	+.143
114	0.0	0.0	0.0	0.0	0.0	+.143
117	0.0	0.0	0.0	0.0	0.0	+.143
123	0.0	0.0	0.0	0.0	0.0	+.143
130	0.0	0.0	0.0	0.0	0.0	+.143
Apex	0.0	0.0	0.0	0.0	0.0	+.143

Table 3.8-5e
CONTAINMENT STRUCTURE STRESSES - LOADING #5 INTERNAL PRESSURE

p = 60psig δRD = 0.383 in. δR = 0.492 in.

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional Nϕ	Hoop Nθ	Meridional Mϕ	Hoop Mθ	Meridional Shear Vϕ	Radial Displacement δ R
Base 0	227.0	+79.6	-30.0	0.0	+55.3	.009
3	227.0	+127.4	+106.0	0.0	+36.2	.149
6	227.0	+199.4	+190.6	0.0	+20.9	.226
10	227.0	+282.2	+243.0	0.0	+6.2	.314
15	227.0	+363.1	+243.6	0.0	-4.8	.401
20	227.0	+418.8	+205.7	0.0	-9.7	.460
30	227.0	+469.0	+102.8	0.0	-9.5	.514
40	227.0	+473.2	+28.9	0.0	-5.2	.518
60	227.0	+454.2	+10.8	0.0	0.0	.498
75	227.0	+454.0	-7.1	0.0	0.0	.492
85	227.0	+438.0	-3.9	0.0	0.0	.480
90	227.0	+428.0	+34.7	0.0	-0.4	.470
95	227.0	+354.0	+7.7	0.0	-12.8	.388
Springline 99	227.0	+322.0	-60.5	0.0	-21.6	.353
102	227.0	+210.0	-126.7	0.0	-18.2	.346
105	0.0	+182.0	-199.1	0.0	-25.0	.301
Dome Anchor 108	0.0	+229.0	19.8	0.0	+3.1	.368
-111	0.0	+243.0	+10.3	0.0	+3.3	.402
111	227.0	+243.0	+10.3	0.0	+3.3	.402
+111	227.0	+243.0	+10.3	0.0	+3.3	.402
114	227.0	+243.0	+4.3	0.0	+2.0	402
117	227.0	+238.0	+0.2	0.0	0.8	.393
123	227.0	+230.0	0.0	0.0	0.0	388
130	227.0	227.0	0.0	0.0	0.0	.383
Apex	227.0	227.0	0.0	0.0	0.0	.383

Table 3.8-5f
CONTAINMENT STRUCTURE STRESSES - LOADING #6 ACCIDENT TEMPERATURE - P = 60 PSIG, T = 286°F

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional Nϕ	Hoop Nθ	Meridional Mϕ	Hoop Mθ	Meridional Shear Vϕ	Radial Displacement δ R
Base 0	8.0	-1.5	0.0	0.0	1.2	0.000
3	8.0	-0.6	2.5	0.0	-0.8	.001
6	8.0	+1.2	4.3	0.0	0.5	.003
10	8.0	+3.0	5.5	0.0	0.1	.005
15	8.0	+5.0	5.5	0.0	-0.1	.007
20	8.0	+6.0	4.6	0.0	-0.2	.008
30	8.0	+6.7	2.3	0.0	-0.2	.009
40	8.0	+6.7	0.6	0.0	-0.1	.009
60	8.0	+6.7	-0.2	0.0	0.0	.009
75	8.0	+6.7	0.0	0.0	0.0	.009
85	8.0	+25.8	-80.0	0.0	0.0	.030
90	8.0	+54.1	-85.7	0.0	+0.9	.061
95	8.0	+102.4	-66.8	0.0	+6.8	.114
Springline 99	8.0	+120.7	-28.4	0.0	+13.7	.134
102	8.0	+54.0	-0.3	0.0	-5.6	.179
105	8.0	+84.4	+8.7	0.0	-1.0	.229
Dome Anchor 108	8.0	+103.7	+8.2	0.0	+0.9	.261
-111	111.0	111.0	+5.0	0.0	+1.1	.273
111	111.0	111.0	0.0	0.0	0.0	.273
+111	111.0	111.0	0.0	0.0	0.0	.273
114	111.0	111.0	0.0	0.0	0.0	.273
117	111.0	111.0	0.0	0.0	0.0	.273
123	111.0	111.0	0.0	0.0	0.0	.273
130	111.0	111.0	0.0	0.0	0.0	.273
Apex	111.0	111.0	0.0	0.0	0.0	.273

Table 3.8-5g
CONTAINMENT STRUCTURE STRESSES - LOADING #7 ACCIDENT TEMPERATURE - P = 90 PSIG, T = 312°F

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional Nϕ	Hoop Nθ	Meridional Mϕ	Hoop Mθ	Meridional Shear Vϕ	Radial Displacement δ R
Base 0	8.0	-1.5	0.0	0.0	1.2	0.000
3	8.0	-0.6	2.5	0.0	0.8	.001
6	8.0	+1.2	4.3	0.0	0.5	.003
10	8.0	+3.0	5.5	0.0	0.1	.005
15	8.0	+5.0	5.5	0.0	-0.1	.007
20	8.0	+6.0	4.6	0.0	-0.2	.008
30	8.0	+6.7	2.3	0.0	-0.2	.009
40	8.0	+6.7	0.6	0.0	-0.1	.009
60	8.0	+6.7	-0.2	0.0	0.0	.009
75	8.0	+6.7	0.0	0.0	0.0	.009
85	8.0	+35.0	-90.0	0.0	0.0	.040
90	8.0	+61.4	-97.6	0.0	1.0	.069
95	8.0	+97.9	-76.1	0.0	7.7	.109
Springline 99	8.0	+134.5	-32.3	0.0	+15.6	.149
102	8.0	+59.8	-0.3	0.0	-6.4	.200
105	8.0	+95.6	+10.0	0.0	-1.1	.259
Dome Anchor 108	8.0	+119.9	+9.8	0.0	+1.0	.299
-111	+126.0	+126.0	+5.7	0.0	1.3	.309
111	+126.0	+126.0	0.0	0.0	0.0	.309
+111	+126.0	126.0	0.0	0.0	0.0	.309
114	+126.0	+126.0	0.0	0.0	0.0	.309
117	+126.0	+126.0	0.0	0.0	0.0	.309
123	+126.0	+126.0	0.0	0.0	0.0	.309
130	+126.0	+126.0	0.0	0.0	0.0	.309

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional $N\phi$	Hoop $N\theta$	Meridional $M\phi$	Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δR
Apex	+126.0	+126.0	0.0	0.0	0.0	.309

Table 3.8-5h
CONTAINMENT STRUCTURE STRESSES - LOADING #8 0.10G EARTHQUAKE - HORIZONTAL + VERTICAL COMPONENT

	Stress Resultants		Stress Couples			
Location in Feet Up From Base Element No.	Meridional $N\phi$	Hoop $N\theta$	Meridional $M\phi$	Hoop $M\theta$	Meridional Shear $V\phi$	Radial Displacement δ R
Base 0	70.3	0	0	0	0	0
3	+68.3	0	0	0	0	.002
6	+66.3	0	0	0	0	.003
10	+63.6	0	0	0	0	.005
15	+60.2	0	0	0	0	.007
20	+56.9	0	0	0	0	.010
30	+50.3	0	0	0	0	.016
40	+46.7	0	0	0	0	.021
60	+31.6	0	0	0	0	.034
75	+23.3	0	0	0	0	.044
85	+18.4	0	0	0	0	.050
90	+16.1	0	0	0	0	.053
95	+14.0	0	0	0	0	.055
Springline 99	+12.3	0	0	0	0	.058
102	+11.2	0	0	0	0	.059
105	+10.0	0	0	0	0	.062
Dome Anchor 108	+9.1	0	0	0	0	.062
-111	+8.2	0	0	0	0	.063
111	+8.2	0	0	0	0	.063
+111	+8.2	0	0	0	0	.063
114	+7.4	0	0	0	0	.064
117	+6.5	0	0	0	0	.064
123	+4.9	0	0	0	0	.063
130	+3.5	0	0	0	0	.059
Apex	0	0	0	0	0	0

Table 3.8-6a
CONTAINMENT STRUCTURE LOADING COMBINATIONS - LOAD NUMBERS 1 THROUGH 48

Load Combinations	Load No.	DL	VP	OT_W	OT_S	IP P=60	AT_{60}	AT_{90}	E a=0.1g
Normal Operation	1	1.0	1.0	1.0					
(MODES 1 and 2)	2	1.0	1.17	1.0					
	3	1.0	1.0		1.0				
	4	1.0	1.17		1.0				
	5	1.0	1.0	1.0					2.0
	6	1.0	1.17	1.0					2.0
	7	1.0	1.0		1.0				2.0
	8	1.0	1.17		1.0				2.0
	9	1.0	1.0	1.0					-2.0
	10	1.0	1.17	1.0					-2.0
	11	1.0	1.0		1.0				-2.0
	12	1.0	1.17		1.0				-2.0
Test	13	1.0	1.0	1.0		1.15			
	14	1.0	1.17	1.0		1.15			
	15	1.0	1.0		1.0	1.15			
	16	1.0	1.17		1.0	1.15			
Accident Pressure Condition "d"	17	1.0	1.0	1.0		1.0	1.0		
	18	1.0	1.17	1.0		1.0			
	19	1.0	1.0		1.0	1.0	1.0		
	20	1.0	1.17		1.0	1.0	1.0		
	21	1.0	1.0	1.0		1.0	1.0		0.8
	22	1.0	1.17	1.0		1.0	1.0		0.8
	23	1.0	1.0		1.0	1.0	1.0		0.8
	24	1.0	1.17		1.0	1.0	1.0		0.8
	25	1.0	1.0	1.0		1.0	1.0		-0.8

Load Combinations	Load No.	DL	VP	OT_W	OT_S	IP P=60	AT_{60}	AT_{90}	E a=0.1g
	26	1.0	1.17	1.0		1.0	1.0		-0.8
	27	1.0	1.0		1.0	1.0	1.0		-0.8
	28	1.0	1.17		1.0	1.0	1.0		-0.8
Condition "a"	29	1.0	1.0	1.0		1.5		1.0	
	30	1.0	1.17	1.0		1.5		1.0	
	31	1.0	1.0		1.0	1.5		1.0	
	32	1.0	1.17		1.0	1.5		1.0	
Condition "b"	33	1.0	1.0	1.0		1.25		1.0	1.0
	34	1.0	1.17	1.0		1.25		1.0	1.0
	35	1.0	1.0		1.0	1.25		1.0	1.0
	36	1.0	1.17		1.0	1.25		1.0	1.0
	37	1.0	1.0	1.0		1.25		1.0	-1.0
	38	1.0	1.17	1.0		1.25		1.0	-1.0
	39	1.0	1.0		1.0	1.25		1.0	-1.0
	40	1.0	1.17		1.0	1.25		1.0	-1.0
Condition "c"	41	1.0	1.0	1.0		1.0	1.0		2.0
	42	1.0	1.17	1.0		1.0	1.0		2.0
	43	1.0	1.0		1.0	1.0	1.0		2.0
	44	1.0	1.17		1.0	1.0	1.0		2.0
	45	1.0	1.0	1.0		1.0	1.0		-2.0
	46	1.0	1.17	1.0		1.0	1.0		-2.0
	47	1.0	1.0		1.0	1.0	1.0		-2.0
	48	1.0	1.17		1.0	1.0	1.0		-2.0

Table 3.8-6b
CONTAINMENT STRUCTURE LOADING COMBINATIONS - KEY TO SYMBOLS

KEY

Loading Number Fundamental Load	Symbol	Meaning
No. 1	DL	Dead Load
No. 2	VP	Vertical Prestress
No. 3	OT_W	Operating Temperature - Winter
No. 4	OT_S	Operating Temperature - Summer
No. 5	IP	Internal Pressure (P=60 psig)
No. 6	AT_{60}	Accident Pressure + Temperature (P=60 psig; T = 286 °F)
No. 7	AT_{90}	Accident Pressure + Temperature (P=90 psig; T =312 °F)
No. 8	E	Design Earthquake (horizontal acceleration 0.10g)

Table 3.8-7
CONCRETE COVER REQUIRED FOR REINFORCING STEEL

		Minimum Cover	
Location	Type of Steel	Actual	ACI 318
Dome	Principal (18S)	11-1/2 in	2-1/4 in.
	Crack control	2 in	1-1/2 in.
Cylinder	Hoop (18S)	2-3/8 in	2-1/4 in
	Vertical (14S & other)	4-5/8 in	1-3/4 & 1 1/2-in
Base ring	Bottom reinforcing	3 in.	3 in.
	Top reinforcing	1-1/2 in	1-1/2 in.
Base slab	Bottom reinforcing	3 in.	3 in.
	Top reinforcing	1-1/2 in.	1-1/2 in.

Table 3.8-8
ELASTOMER PADS PROPERTIES

Original Physical Properties	
Tear resistance, ASTM D625 D_6C C, psi of thickness, minimum 180	180
Hardness, ASTM D676, points	55 ± 3
Tensile strength, ASTM D412, minimum psi	2500
Ultimate elongation, minimum %	400
Change in Physical Properties (Oven Aging 70 hr at 212 °F in accordance with ASTM D573)	
Hardness, points change	0 to +15
Tensile strength, % change	±15
Ultimate elongation, maximum %	-40
Extreme Temperature Characteristics	
Compression set under constant deflection, (22 hr at 158 °F) ASTM D395 (Method B), maximum, %	25
Low temperature brittleness, ASTM D745, no breaks above	-20 °F
Ozone Cracking Resistance	
Exposure to 100 parts per 100 million of ozone in air by volume at a strain of 20% and a temperature of 100 °F ± 2° in a test otherwise conforming to ASTM D1149. (Samples shall be solvent-wiped before test to remove any traces of surface impurities). Time within which no cracks develop, minimum hours	100
Oil Sell, ASTM Oil No. 3	
70 Hours at 212 °F, ASTM D471, volume change, maximum, %	+80
Shear modulus, psi	138 ± 10%

Table 3.8-9
ROCK ANCHOR A - UPLIFT TEST WITH JACKING FRAME, MAY 19, 1966

		Pier Dials				Rock Surface Pegs		
Time	Load Kips	NE Corner (in.)	SW Corner (in.)	Head Dial (in.)	Average Deformation Top of Pier (in.)	North (in.)	Intermedite (in.)	South (in.)
0840	0	.300	0	.700	0	7-1/4	7-5/8	9-3/4
0955	20	.304	.005	.705	.0045			
1010	40	.308	.009	.709	.0085			
1025	60	.311	.012	.714	.0115			
1040	80	.318	.019	.723	.0185			
1055	100	.354	.031	.752	.0425	7-1/4	7-9/16	9-5/8
					LIFT OFF APPARENT			
1105	110	.380	.039	.767	.0595			
	80	.349	.025	739	.037			
	60	.334	.016	.724	.025			
	40	.326	010	.715	.018			
	20	.318	.003	.706	.0105			
	0	.312	-.002	.699	.005	7-1/4	7-9/16	9-5/8

Table 3.8-10
DESIGN CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

(AISC 1963 Versus AISC 1980)

Referenced Subsection		-	-
AISC 1980	**AISC 1963**	**Structural Elements Potentially Affected**	**Comments**
1.5.1.2.2	---	Beam end connection where the top flange is coped and subject to shear, or failure by shear along a plane through fasteners or by a combination of shear along a plane through fasteners plus tension along a perpendicular plane.	See case study 1 for details.
1.9.1.2 and Appendix C	1.9.1	Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2.	New provisions added in the 1980 Code, Appendix C. See case study 10 for details.
1.10.6	1.10.6	Hybrid girder - reduction in flange stress.	New requirements added in the 1980 Code Hybrid girders were not covered in the 1963 Code. See case study 9 for details.
1.11.4	1.11.4	Shear connectors in composite beams.	New requirements added in the 1980 Code regarding the distribution of shear connectors. (Equation 1.11-7). The diameter and spacing of the shear connectors are also subject to new controls.
1.11.5	---	Composite beams or girders with formed steel deck.	New requirement added in the 1980 Code.
1.14.2.2	---	Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members.	New requirement added in the 1980 Code.
1.15.5.2, 1.15.5.3, 1.15.5.4	---	Restrained members when flange or moment connection plates for end connections of beams and girders are welded to the flange of I or H shaped columns.	New requirement added in the 1980 Code.
2.9	2.8	Lateral bracing of members to resist lateral and torsional displacement.	Scale A 0.0 $M/M_p < 1.0$; C 0.0 $M/M_p > -1.0$

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

(AISC 1963 Versus AISC 1980)

Referenced Subsection		-	-
AISC 1980	**AISC 1963**	**Structural Elements Potentially Affected**	**Comments**
			See case study 7 for details.

Table 3.8-11
ACI 318-63 VERSUS ACI 349-76 CODE COMPARISONS

Reference Subsection			
ACI 349-76	**ACI 318-63**	**Structural Elements Potentially Affected**	**Comments**
7.10.3	805	Columns designed for stress reversals with variation of stress from f_y in compression to 1/2 f_y in tension	Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions.
11.13	---	Short brackets and corbels which are primary load-carrying members	As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be nonductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.15	---	Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear type cracks	Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.16	---	All structural walls - those which are primary load carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects	Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements.
Appendix A	---	All elements subject to time-dependent and position-dependent temperature variations and restrained so that thermal strains will result in thermal stresses	For structures subject to effects of pipe break, especially jet impingement, thermal stresses may be significant. Scale A for areas of jet impingement or where the conditions could develop causing concrete temperature to exceed limitation of A.4.2.
Appendix B	---	All steel embedments used to transmit loads from attachments into the reinforced-concrete structure	New appendix; therefore, considerable review of older designs is warranted. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

Reference Subsection			
ACI 349-76	**ACI 318-63**	**Structural Elements Potentially Affected**	**Comments**
Appendix C	---	All elements whose failure under impulsive and impactive loads must be precluded	New appendix; therefore, consideration and review of older designs is considered important. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

Table 3.8-12
ACI 301-63 VERSUS ACI 301-72 (REVISED 1975) COMPARISON

No significant changes were found in the ACI 301 Code comparison.

Table 3.8-13
ACI 318-63 VERSUS ASME B&PV CODE, SECTION III, DIVISION 2, 1980 CODE COMPARISON

Referenced Subsection			
Sec. III 1980	**ACI 318-63**	**Structural Elements Potentially Affected**	**Comments**
CC-3421.5	---	Containment and other elements transmitting in-plane shear.	New concept. There is no comparable section in ACI 318-63, i.e. no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain condition, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore, a check of the old designs could show some significant decrease in overall prediction of structural integrity.
CC-3421.6	1707	Regions subject to peripheral shear in the region of concentrated forces normal to the shell surface.	These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63 (Sections 1707 (c) and (d)) which address "punching" shear in slabs and footings with the ϕ factor taken care of in the basic shear equation (Section CC-3521.2.1, Equation 10).

Table 3.8-14
ASME B&PV CODE, SECTION III, DIVISION 2, 1980 (ACI 359-80) VERSUS ACI 318-63 CODE COMPARISION

Sec. III 1980	AIC 318-63	Structural Elements Potentially Affected	Comments
CC-3421.6			Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 13 for details.
CC-3421.7	921	Regions subject to torsion.	New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses.
CC-3421.8	---	Bracket and corbels.	New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria, and failure of such elements could be nonductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
CC-3440(b),(c)	---	All concrete elements which could possibly be exposed to short-term high thermal loading.	New limitations are imposed on short-term thermal loading. No comparable provisions existed in the ACI 318-63.
CC-3532.1.2	---	Where biaxial tension exists.	ACI 318-63 did not consider the problem of development length in biaxial tension fields.

Table 3.8-15
LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED

Structural Elements To Be Examined	Code Changes Affecting These Elements	
	New Code	Old Code
Beams	AISC 1980	AISC 1963
Composite Beams		
1. Shear connectors in composite beams	1.11.4	1.11.4
2. Composite beams or girders with formed steel deck	1.11.5	---[a]
Hybrid Girders		
Stress in flange	1.10.6	1.10.6
Compression Elements	AISC 1980	AISC 1963
With width-to-thickness ratio higher than specified in 1.9.1.2	1.9.1.2 and Appendix C	1.9.1
Tension Members	AISC 1980	AISC 1963
When load is transmitted by bolts or rivets	1.14.2.2	---
Connections	AISC 1980	AISC 1963
Beam ends with top flange coped, if subject to shear	1.5.1.2.2	---
Connections carrying moment or restrained member connection	1.15.5.2, 1.15.5.3, 1.15.5.4	---
Members designed to operate in an inelastic regime	AISC 1980	AISC 1963
Spacing of lateral bracing	2.9	2.8
Short brackets and corbels having a shear span-to-depth ratio of unity or less	ACI 349-76, 11.13	ACI 318-63
Shear walls used as a primary load-carrying member	ACI 349-76, 11.16	ACI 318-63
Precast concrete structural elements, where shear is not a measure of diagonal tension	ACI 349-76, 11.15	ACI 318-63
Concrete regions subject to high temperatures	ACI 349-76	ACI 318-63
Time-dependent and position-dependent temperature variations	Appendix A	---
Columns with spliced reinforcement subject to stress reversals; f_y in compression to 1/2 f_y in tension	ACI 349-76, 7.10.3	ACI 318-63, 805
Steel embedments used to transmit load to concrete	ACI 349-76, Appendix B	ACI 318-63

Structural Elements To Be Examined	Code Changes Affecting These Elements: New Code	Code Changes Affecting These Elements: Old Code
Elements subject to impulsive and impactive loads whose failure must be precluded	ACI 349-76, Appendix C	ACI 318-63
Containment and other elements, transmitting in-plane shear	B&PV Code Section III, Division 2, 1980, CC-3421.5	ACI 318-63
Region of shell carrying concentrated forces normal to the shell surface (See case study 13 for details)	B&PV Code, Section III, Division 2, 1980, CC-3421.6	ACI 318-63, 1707
Region of shell under torsion	B&PV Code Section III, Division 2, 1980, CC-3421.7	ACI 318-63, 921
Elements subject to short-term high temperature loading	B&PV Code Section III, Division 2, 1980, CC-3440(b), (c)	ACI 318-63
Elements subject to biaxial tension	B&PV Code, Section III Division 2, 1980, CC-3532.1.2	ACI 318-63
Brackets and corbels	B&PV Code, Section III, Division 2, 1980, CC-3421.8	ACI 318-63
Roof[b]	---	---
Extreme environmental snow loads are provided by SEP Topic II-2.A. Regulatory Guide 1.102 (Position 3) provides guidance to preclude adverse consequences from ponding on parapet roofs. Failure of roofs not designed for such circumstances could generate impulsive loadings and water damage, possibly extending to Seismic Category I components of all floor levels.		

a. Dash (---) indicates that no provisions were provided in the older code.

b. Not shown in tabular summary of code comparisons.

Table 3.8-16
MASSES, MOMENT OF INERTIA (I), FLEXURAL AREA (A), AND SHEAR AREA (A_s) FOR THE LLNL MODEL

Node	Element	Mass, lb-sec^2/in.	I, in. (x 10^9)	A, in. (x 10^4)	A_s, in. (x 10^4)
13		2480.4			
	12		5.202	12.15	6.074
12		4952.8			
	11		15.35	12.17	6.086
11		4952.8			
	10		21.80	12.08	6.038
10		7007.2			
	9		40.09	19.03	9.516
9		6491.06			
	8		36.44	17.18	8.590
8		5972.0			
	7		36.44	17.18	8.590
7		5972.0			
	6		36.44	17.18	8.590
6		5972.0			
	5		36.44	17.18	8.590
5		5972.0			
	4		36.44	17.18	8.590
4		5972.0			
	3		36.44	17.18	8.590
3		5972.0			
	2		36.44	17.18	8.590
2		5972.0			
	1		36.44	17.18	8.590
1		5972.0			

Table 3.8-17
MODAL FREQUENCIES FOR THE LAWRENCE LIVERMORE NATIONAL LABORATORY CONTAINMENT SHELL MODEL

Mode	Frequency
1	6.97
2	18.87
3	21.47
4	37.75
5	53.91
6	54.60
7	70.23
8	80.89
9	84.70
10	92.38

Table 3.8-18
RESPONSE VALUES FOR REGULATORY GUIDE 1.60 HORIZONTAL (0.17g) AND VERTICAL (0.11g) SPECTRA INPUT

	Horizontal		Vertical
Element	Moment (lb-in. x 10^9)	Shear (lb x 10^6)	Axial (lb x 10^6)
12	0.102	0.60	0.204
11	0.391	1.70	0.603
10	0.842	2.68	0.985
9	1.41	3.89	1.50
8	2.12	4.90	1.94
7	2.95	5.71	2.32
6	3.88	6.40	2.65
5	4.90	6.97	2.94
4	5.97	7.42	3.18
3	7.09	7.76	3.37
2	8.24	7.98	3.48
1	9.42	8.08	3.55

Table 3.8-19
PEAK HARMONIC AMPLITUDES OF THE SEISMIC LOAD ON CYLINDER AND DOME OF THE CONTAINMENT SHELL

Elevation[a] (in.)	Load Amplitude (psi)
0	0
73	0.334
219	0.736
365	1.138
511	1.508
657	1.908
803	2.310
949	2.712
1095	5.310
1188	
ϕ (rad)	**Load Amplitude (psi)**
1.57	3.944
1.80	2.074
2.20	2.907
2.62	4.602
3.14	

a. Elevation measured from mid-surface of base slab.

Table 3.8-20
MATERIAL PROPERTIES FOR STEEL, CONCRETE, AND FOAM INSULATION

	Steel Liner	Concrete	Insulation	Reinforcement Steel
Young's modulus (psi)	29×10^6	4.3×10^6	---	29×10^6
Poisson's ratio	0.3	0.25	---	---
Coefficient of thermal expansion of (in./in.-°F)	6.3×10^{-6}	5.5×10^{-6}	---	---
Density (lb/ft^3)	490	150	4	---
Coefficient of thermal conductivity, Btu/hr ft, °F	26	0.44	0.022	---
Specific heat Btu/lbm °F	0.11	0.160	0.30	---
Thickness (in.)	0.375	43.30	1.25	
σ_Y (psi) Steel and f'_c (psi) Concrete	32,000	5,000	---	40,000

Table 3.8-21
MAXIMUM DISPLACEMENTS OF 5/8-INCH S6L STUDS IN THE INSULATION TERMINATION REGION

Stud Capacity Qu (kips) (1)	Buckled Panel Stress (ksi) (2)	Maximum Stud Displacement Δ (in.) (3)	Ultimate Stud Displacement (in.) (4)	Max/Ultimate Displacement % (5)
10.6	26	0.141	0.167	84
8.3	26	0.148	0.167	89
10.6	29	0.159	0.167	95
8.3	29	0.166	0.167	99

Table 3.8-22
MAXIMUM DISPLACEMENT OF STUDS IN GENERAL DOME

						Membrane liner Strains (in./in.)			
Stud Capacity Qu (kips)	Stress Limit in Unbuckled Panels (ksi)	Maximum Stud Displacement Δ (in.)	Ultimate Stud Displacement (in.)	Max/Ultimate Displacement %	Liner Lateral Displacement (in.)	Membrane Compression	Membrane and Bending Compression	Membrane and Bending Tension	Column (8)/ $\in_y$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
5/8-In. Diameter S6L Studs at 24 In.									
10.6	26	0.113	0.167	68	1.67	0.0096	0.0558	0.0366	35
8.3	26	0.150	0.167	90	1.92	0.0097	0.0626	0.0433	39
10.6	29	0.170	0.167	102	2.03	0.0088	0.0597	0.0422	37
8.3	29	>0.300	0.167	>>100	NA	NA	NA	NA	NA
3/4-In. Diameter Headed Studs at 51 In.									
31.1	5.8	0.00343	0.341	1	0.42	0.000177	0.000767	0.000413	0.5
31.1	12	0.0388	0.341	11	1.41	0.00020	0.0024	0.0019	1.5
$\in_y$ = 48/30000 = 0.0016 in./in.									

Table 3.8-23
LOAD DEFINITIONS

D	Dead loads or their related internal moments and forces (such as permanent equipment loads).
E or E_O	Loads generated by the operating-basis earthquake.
E′ or E_{ss}	Loads generated by the safe shutdown earthquake.
F	Loads resulting from the application of prestress.
H	Hydrostatic loads under operating conditions.
H_a	Hydrostatic loads generated under accident conditions, such as post-accident internal flooding. (F_L is sometimes used to designate the post-LOCA internal flooding).
L	Live loads or their related internal moments and forces (such as movable equipment loads).
P_a	Pressure load generated by accident conditions (such as those generated by the postulated pipe break accident).
P_o or P_v	Loads resulting from pressure due to normal operating conditions.
P_s	All pressure loads which are caused by the actuation of safety relief valve discharge including pool swell and subsequent hydrodynamic loads.
Rs or R_r	Pipe reactions under accident conditions (such as those generated by thermal transients associated with an accident).
R_o	Pipe reactions during startup, normal operating, or shutdown conditions, based on the critical transient or steady-state condition.
R_a	All pipe reaction loads which are generated by the discharge of safety relief valves.
T_a	Thermal loads under accident conditions (such as those generated by a postulated pipe break accident).
T_o	Thermal effects and loads during startup, normal operating, or shutdown conditions, based on the most critical transient or steady-state condition.
T_s	All thermal loads which are generated by the discharge of safety relief valves.
W	Loads generated by the design wind specified for the plant.
W′ or W_t	Loads generated by the design tornado specified for the plant. Tornado loads include loads due to tornado wind pressure, tornado-created differential pressure, and tornado-generated missiles.
Y_j	Equivalent static load on the structure generated by the impingement of the fluid jet from the broken pipe during the design-basis accident.
Y_m	Missile impact equivalent static load on the structure generated by or during the design-basis accident, such as pipe whipping.

Y_r	Equivalent static load on the structure generated by the reaction on the broken pipe during the design-basis accident.

3.9 MECHANICAL SYSTEMS AND COMPONENTS

3.9.1 SPECIAL TOPICS FOR MECHANICAL COMPONENTS

3.9.1.1 Design Transients

3.9.1.1.1 Load Combinations

The load combinations considered in the original design of Ginna Station were (1) normal + design earthquake, (2) normal + maximum potential earthquake, and (3) normal + pipe rupture loads. "Normal," "Upset," "Emergency," and "Faulted" terminology was not used in the original safety evaluation of Ginna Station.

3.9.1.1.2 Cyclic Loads

3.9.1.1.2.1 Thermal and Pressure Cyclic Loads

The various components in the reactor coolant system were designed to withstand the effects of cyclic loads due to reactor system temperature and pressure changes. These cyclic loads are introduced by normal unit load transients, reactor trip, and startup and shutdown operation (see Section 5.1.5). The number of thermal and loading cycles used for design purposes is shown in Table 5.1-4.

3.9.1.1.2.2 Pressurizer Surge Line

NRC Bulletin 88-11 requested licensees to take certain actions to monitor thermal stratification in the pressurizer surge line because recent measurements indicate that top-to-bottom temperature in the surge line can reach 250°F to 300°F in certain modes of operation, particularly during heatup and cooldown. Surge line temperature stratification causes bending of the pipe and possible reduction of fatigue life. RG&E joined the Westinghouse Owners Group in a program to perform a generic evaluation of surge line stratification in Westinghouse PWRs. Temporary thermocouples were installed on the pressurizer surge line and four temporary displacement transducers were installed on the surge line to monitor movement during heatup, cooldown, and other temperature stratification conditions. The data was continuously monitored by a data logging computer installed in the Multiplexer (MUX) room for the duration of the test, which commenced in June 1989 and was completed during the 1990 MODE 6 (Refueling) outage when the instrumentation was removed.

The generic evaluation of surge line stratification in Westinghouse PWRs was reported in Westinghouse Owners Group report, WCAP 12639, submitted to the NRC in June 1990. Westinghouse performed a plantspecific analysis of the Ginna pressurizer surge line to demonstrate compliance with NRC Bulletin 88-11, and the results were reported in WCAP 12928 (*Reference 1*). The results indicated that the surge line meets the stress limits and usage factor requirements, and the pressurizer surge nozzle meets the code stress allowables under thermal stratification loading and fatigue usage requirements of ASME Section III, 1986 edition. By *Reference 20*, the NRC found the RG&E response to Bulletin 88-11 to be acceptable.

3.9.1.1.2.3 Unisolable Connections to the Reactor Coolant System

NRC Bulletin 88-08 requested licensees to review systems connected to the reactor coolant system piping to determine whether unisolable sections of piping connected to the reactor coolant system can be subjected to stresses from temperature stratification or temperature oscillations that could be induced by leaking valves and that were not evaluated in the design analysis of the piping. The Bulletin requested that

a. For any unisolable sections of piping connected to the reactor coolant system that may have been subjected to excessive thermal stresses, licensees nondestructively examine the welds, heat-affected zones, and high stress locations, including geometric discontinuities in that piping, to provide assurance that there are no existing flaws.
b. Licensees plan and implement a program to provide continuing assurance that unisolable sections of all piping connected to the reactor coolant system will not be subjected to combined cyclic and static thermal and other stresses that could cause fatigue during the remaining life of the unit. This assurance may be provided by
    1. Redesigning and modifying these sections of piping to withstand combined stresses caused by various loads including temporal and spatial distributions of temperature resulting from leakage across valve seats.
    2. Instrumenting this piping to detect adverse temperature distributions and establishing appropriate limits on temperature distributions.
    3. Means for ensuring that pressure upstream from block valves that might leak is monitored and does not exceed reactor coolant system pressure.

RG&E determined that there were three unisolable sections of piping connected to the reactor coolant system that had the potential for thermal cycling. These sections are as follows:

aa. Charging system to loop B hot leg between check valve 393 and the reactor coolant system nozzle.
bb. Alternate charging system to loop A cold leg between check valve 383A and the reactor coolant system nozzle.
cc. Auxiliary pressurizer spray system between check valve 297 and the 3-in. tee, which connects the auxiliary pressurizer spray to the main pressurizer spray line.

Examinations were performed at the most susceptible locations, as recommended by Westinghouse, on each of the three unisolable pipe sections. All examination results were acceptable.

A program to provide assurance that the identified unisolable sections of piping attached to the reactor coolant system do not fail, due to thermally initiated or advanced fatigue, was initiated. This assurance was provided, in part, by instrumenting the affected piping to detect adverse temperature conditions and by nondestructive examinations during MODE 6 (Refueling) outages. Temporary thermocouples were installed on the affected piping during the 1989 MODE 6 (Refueling) outage. The data was monitored by a data logging computer installed in the MUX room for that purpose.

The temperature monitoring was continued until the 1991 refueling outage when the instrumentation was removed. The data was analyzed and it was determined that adverse temperature conditions did not exist. Based on the results of the temperature monitoring, nondestructive examinations, and engineering analysis, the program was restructured to provide continued assurance based on periodic nondestructive examinations during MODE 6 (Refueling) outages. By *Reference 21*, the NRC reported that the staff had determined that the RG&E response to Bulletin 88-08 met the requirements.

3.9.1.1.3 Transient Hydraulic Loads

Transient hydraulic loads were considered in the dynamic analysis of the pressurizer safety and relief valve discharge lines (*References 2* and *22*) (see Section 3.9.2.1.4).

3.9.1.1.4 Operating-Basis Earthquake

The mechanical systems and components in the original design of Ginna Station were designed for the operating-basis earthquake using the response spectra developed by Housner and characterized by a peak ground acceleration of 0.08g at 0.5% damping. The operating-basis earthquake was not considered during the Systematic Evaluation Program (SEP) reevaluation (see Section 3.7).

3.9.1.1.5 Safe Shutdown Earthquake

The mechanical systems and components in the original Ginna design were reviewed for a safe shutdown earthquake of 0.2g peak ground acceleration. The response spectra developed by Housner were used for this purpose. For the SEP review, the seismic input motion was typically defined by means of floor response spectra generated by direct method or by means of a time-history analysis. See Section 3.7 for details of how the floor response spectra were developed.

3.9.1.1.6 Secondary System Fluid Flow Instability (Water Hammer)

Secondary system flow instability (water hammer) was considered in the dynamic analysis of the main and auxiliary feedwater piping (*Reference 3*) presented in Section 3.9.2.1.6. It was determined that the primary cause for water hammer was the recovery of the feed ring while feedwater flows were above a threshold flow. This threshold flow was determined to be approximately 200 gpm. Design of the feed ring piping, installation of J-tubes in the feed ring and operating procedures minimize the possibility of water hammer.

3.9.1.1.7 Loss-of-Coolant Accident

The forces exerted on reactor internals and core, following a loss-of-coolant accident, were originally computed by employing the BLODWN-1 digital computer program developed for the space-time-dependent analysis of multiloop PWR plants (see Section 3.9.2.3). Additional analysis of the blowdown effects was performed during the resolution of the unresolved safety issue A-2, Asymmetric Blowdown Loads, discussed in Section 3.9.2.4.

3.9.1.2 Computer Programs Used in Analysis

The following computer programs were used in the dynamic and static analyses of the Seismic Category I systems and components:

ITCHVALVE	Used to perform the transient hydraulic analysis of the pressurizer safety and relief line analysis.
FORFUN	Used to calculate unbalanced forces for each straight segment of pipe from the pressurizer to the relief tank.
WESTDYN	A special purpose program designed for the static and dynamic solution of redundant piping systems with arbitrary loads and boundary conditions.
FIXFM and FIXFM3	Computer programs which determine the time-history response of three-dimensional structures excited by an internal forcing function.
WESTDYN-2 and WESTDYN2	A slightly modified version of WESTDYN program, this program accepts the time-history displacements from FIXFM (or FIXFM3) and calculates the time-history internal forces in the pipe elements.
ADLPIPE	Was used in the original pipe stress analysis of Ginna Station. The verification of this piping analysis program developed by Arthur D. Little, Inc., was provided to the NRC in a memorandum dated April 19, 1979.
M003	A Gilbert/Commonwealth computer program for piping stress analysis. It consists of the Southern Service Company thermal stress program and the IBM scientific subroutine for eigenvalue problems. M003 has been verified against PIPDYN II.
PIPDYN II	(Gilbert/Commonwealth version) - A piping analysis computer program developed by Franklin Institute Research Laboratory. It has been verified against ASME Sample Problem No. 1 in the ASME publication, Pressure Vessel and Piping: 1972 Computer Programs Verification, and ANSYS and PIPESD.
DYNAFLEX	A piping analysis computer program developed by Auton Computing Corporation. It has been verified against ADLPIPE and PIPESD.
PIPESD	A piping analysis computer program developed by URS/John A. Bloom and Associates. It has been verified against ANSYS, ADLPIPE, PIPDYN, and SAP IV.
NUPIPE	A piping analysis computer program developed by Nuclear Services Corporation. It has been verified against ADLPIPE and ASME Benchmark Problem No. 5 in the ASME publication, Pressure Vessel and Piping: 1972 Computer Programs Verification.
PIPSAN	A Westinghouse piping support analysis code.
PS+CAEPIPE	Ginna in house piping anaylsis code.
PD STRUDL	Structural finite element code used @ Ginna.

3.9.1.3 **Experimental Stress Analysis**

3.9.1.3.1 Plastic Model Analysis

During the original design of Ginna Station the mode shapes and frequencies of the primary coolant loop piping system were determined experimentally using model analysis (*Reference 4*).

A plastic model was employed to perform this analysis. Since the reactor pressure vessel, the steam generator, the reactor coolant pump, and their supports are integral to the analysis of the primary loop, they were included in both the plastic model and the mathematical model. The plastic model output of mode shapes and frequencies was coupled with the Housner 0.2g response spectra and used as input to a three-dimensional mathematical model of the primary coolant loop. A computer solution to yield stresses, deflections, support reactions, and equipment nozzle reactions was obtained.

3.9.1.3.2 Plastic Model Details

The model, shown in Figure 3.9-2, was built with a geometric ratio of 0.25 in. equals 1 ft. The plastic model material used was ABS plastic extrusion grade for piping and plexiglas for support structures and equipment. The reactor pressure vessel, steam generator, and reactor coolant pump were represented by hollow circular plastic cylinders filled with lead shot positioned with cotton spacers to properly represent the mass and center of gravity locations of these three pieces of equipment. They were supported by modeled plastic supports.

For a steel beam of identical geometry the natural frequency of the cantilever is 114 Hz. Therefore,

f(steel)/ f(plastic) = 2.78

The ratio of the natural frequency of the model to the prototype was determined by

$$\frac{\omega_{\text{model}}}{\omega_{\text{prototype}}} = \frac{L_p}{L_m}\sqrt{\frac{E_M}{E_p}\cdot\frac{p}{m}}$$

(Equation 3.9-1)

where L_p/L_m = geometric factor and

$$\sqrt{\frac{E_M}{E_p}\cdot\frac{p}{m}} = \text{material factor.}$$

(Equation 3.9-2)

Therefore

ω(model)/ ω (prototype) = 48 / 2.78 = 17.2

3.9.1.3.3 Plastic Model Test Arrangement

Three separate tests were conducted in order to examine the response of the model to a sinusoidal input at various levels. A vertical test and horizontal tests in two perpendicular directions were conducted.

In the horizontal tests, the model was flexibly suspended from a framed supporting structure. One end of the base plate of the model was then secured to the MB vibrator. The arrangement was such that the rigid body rocking modes frequencies were much lower than the frequencies of interest in the piping system. The sizable moment introduced by not driving through the dynamic center of gravity of the system was therefore not a problem. It was possible to conduct the tests in the intended linear direction without very much cross talk or rocking motion.

There was a slight distortion in the geometric scaling of the connecting piping because of available model materials. This geometric relationship is as follows:

Location	Actual Pipe Size		Assumed Pipe Size		Model Pipe Size	
	I.D.	O.D.	I.D.	O.D.	I.D.	O.D.
Cold leg	27.5	32.3	30	36	5/8	3/4
Crossover	31.0	36.8	30	36	5/8	3/4
Hot leg	29.0	34.0	30	36	5/8	3/4
All dimensions are in inches.						

To determine the properties of the plastic, a rectangular sample was separately measured and dynamically tested. The sample was clamped as a cantilever beam to the vibrator and the frequency noted.

The dynamic modulus of elasticity was then calculated. Physical characteristics are as follows:

Sample size = 0.25 x 10 x 1 in.

Volume = 2.5 in.^{3}

Weight = 0.1 lb

Density = 0.04 lb/in.^{3}

For a cantilever beam 8.5 in. long, the test natural frequency was 41 Hz.

Using the equation

$$f_n = \frac{0.56}{l^2}\sqrt{\frac{gEI}{\rho A}} = 0.56\frac{h}{l^2}\sqrt{\frac{EI}{\rho A}}$$

(Equation 3.9-3)

Then the dynamic modulus is

E (plastic) = 547,000 psi

The vertical test was conducted with the model mounted directly to the exciter plate of the vibrator. Since the geometry of the model permitted driving through the center of gravity of the system, rocking excitation was again minimized.

Resonant frequencies and mode shapes were noted by sweeping the model frequency span of 17 to 172 Hz and noting the modal response of the model by use of a strobotac light.

3.9.2 DYNAMIC TESTING AND ANALYSIS

3.9.2.1 Piping Systems

3.9.2.1.1 General

All safety-related and non-safety-related piping systems were originally designed and fabricated to the requirements of USAS B31.1, Power Piping Code. Since the original construction, repairs and/or modifications have been made that have been designed and fabricated to later codes, including ASME Section III. Reanalysis of critical safety-related piping 2-1/2 in. and larger was performed under the Seismic Upgrade Program, which was reviewed by the NRC under SEP Topic III-6 (see Section 3.9.2.1.8). This program updated the piping analysis basis to criteria consistent with the ANSI B31.1 Code, including Summer 1973 Addenda, with some amendments. This code edition remains as the current analysis basis for modifications performed on safety-related piping. Non-safety-related piping is designed and fabricated in accordance with the appropriate current edition of ANSI B31.1.

The loads and load combinations considered in the original design of Ginna Station are given in Table 3.9-1.

The original Ginna Station design did not utilize dynamic computer analyses for seismic qualification of Seismic Category I piping. Seismic Category I piping was divided into three groups, reactor coolant system piping, piping 2-1/2 in. nominal size and larger and piping 2-in. nominal size and smaller. The reactor coolant system piping was seismically qualified using a combination of model testing and analysis. Seismic Category I piping, 2-1/2 in. nominal pipe size and larger, was seismically qualified using equivalent static analyses. Seismic Category I piping, 2-in. nominal pipe size and smaller, was seismically qualified using support spacing tables. Dynamic analysis of sections of the A residual heat removal and B main steam piping were performed solely to verify the equivalent static analysis method. In addition, an onsite inspection of Seismic Category I piping was performed which resulted in the installation of additional supports.

In general, modifications or additions to piping systems at Ginna Station since initial operation have been seismically qualified using dynamic analyses. Some small piping has been seismically qualified utilizing equivalent static analysis or spacing table techniques.

3.9.2.1.2 Seismic Category I Piping, 2-1/2 Inch Nominal Size and Larger

3.9.2.1.2.1 Static Analysis

This group of Seismic Category I pipes was originally analyzed (*Reference 4*) by dividing each pipe run into lumped masses. The number of masses lumped between any two supports was based upon the spacing interval and increased with the length of the spacing interval. Every mass was given an acceleration equal to the maximum response from the response curve with 0.5% of critical damping, i.e., 0.8g for 0.2g ground acceleration. Each piping system, with its supports, was modeled as a three-dimensional frame and the loads given by the mass times the acceleration were applied at each lumped mass along three directions, two horizontal and one vertical, separately. The moments and torque for each of the three loading directions were then obtained by stiffness analysis. The stresses were calculated at critical points in the piping and its supports for each loading direction. The stresses in the piping were found by using the USAS B31.1 formula

$$S = \left[\frac{Mx^2 + My^2 + Mz^2}{z^2}\right]^{1/2}$$

(Equation 3.9-4)

where

S =	stress
Mx, My, Mz =	moments about the two horizontal directions and the vertical direction
Z =	section modulus

At each point the stresses obtained for the two horizontal earthquakes were compared and the one giving the larger value was then combined with the stress obtained for the vertical loading by direct addition. The maximum stresses imposed by the normal loads plus the loads associated with the larger of the two earthquakes (0.8g) were below 1.2S, where S is taken from the power piping code, USAS B31.1.1.0-1967, Paragraph 119.6.4. If the combination of normal loads and no-loss-of-function earthquake loads is considered as a faulted condition, the allowable membrane and bending stresses could be chosen to be the stresses corresponding to 20% and 40% of the material uniform strain at temperature, respectively. This would give more than a factor of 2 margin between the allowable and the maximum actual stresses.

3.9.2.1.2.2 Dynamic Analysis

In order to increase the confidence in the adequacy of the seismic design of this group of Seismic Category I piping, two pipe runs were selected and analyzed employing modal and response spectra methods. These pipe runs were (1) the residual heat removal system line

from the reactor coolant system loop A to the containment penetration, and (2) the main steam line from steam generator B to the containment penetration.

Dynamic analyses were also performed for sections of the above pipe runs and the charging line as a result of IE Bulletin 79-07. These analyses were based on the as-built piping system isometrics and support information.

The defined piping/support systems which were analyzed were evaluated incorporating three-dimensional static and dynamic models which included the effects of the supports, valves, and equipment. The static and dynamic analysis employed the displacement method, lumped parameters, and stiffness matrix formulation and assumed that all components and piping behaved in a linear elastic manner. The response spectra modal analysis technique was used to analyze the piping. The 0.5% Housner ground response spectrum was employed with zero period acceleration values of 0.08g and 0.2g for the operating-basis earthquake and safe shutdown earthquake, respectively. The stress intensification factors due to welds were included in the reanalysis.

3.9.2.1.2.3 Residual Heat Removal System Line From Reactor Coolant System Loop A to Containment

Original dynamic analysis

In the original dynamic analysis the residual heat removal system line was "mathematically" located at the elevation of the steam line on the containment. The reason for this was to investigate the effect of response spectrum distortion, as a function of location and elevation, on the pipe loading and associated stresses.

This pipe run with a 10-in. nominal diameter was selected because it was judged typical of a large portion of Seismic Category I piping with a diameter ranging from 6 in. to 14 in.

Idealized lumped mass models were developed and analyzed dynamically. The analysis was made by assigning three translational and three rotational degrees of freedom to each lumped mass point with each mass point representing a geometrically proportional amount of the total system mass. Elastic characteristics of the system included the translational and rotational stiffnesses. The rotational elastic characteristics were carried into the reduced stiffness matrix that was inverted and formed with the mass matrix, the dynamic matrix.

Following normal mode theory, the natural frequencies, mode shapes, and participation factors were computed to yield the dynamic system characteristics. These characteristics were then combined with the appropriate shock spectra to yield the D'Alembert reverse effective forces on the system for each mode. The modal forces were then used to compute the stresses per mode. The stresses were summed on a root mean square basis for final comparison to code allowable stresses. More than 70 modes were analyzed for their response to earthquake excitation. The Housner 0.5% critical damping ground response spectrum normalized to 0.2g was used. This spectrum was considered adequate because of the location of this pipe run low in the containment.

For the location of maximum stress, the stress values were calculated at three points on the pipe cross-section: the bottom, one side 90 degrees away, and half way between these two.

First the stresses due to the two bending moments and one torsional moment on the pipe were calculated. Then for each of the three points, the root mean square of the stresses acting at the point for the significant modes (first three) was calculated. To this was added the dead weight stress, and then the result was multiplied by the stress intensification factor, as the location of maximum stress was the end of an elbow. The pressure stress was added to this result in order to obtain the total additive longitudinal stress. The total maximum stress was calculated, considering the torsional shear stress and using the formula for maximum principal stresses.

The maximum principal stresses were close to the 1.2S values. They were well below the values corresponding to 20% or 40% of uniform strain. It was concluded that the residual heat removal system line located in the containment at the steam line elevation is not overstressed.

IE Bulletin 79-07 Reanalysis

For the IE Bulletin 79-07 reanalysis, the line analyzed was the residual heat removal system line from the anchor near reactor coolant loop A to the containment penetration.

Table 3.9-2 is a comparison of stress results for the original model, and the model reflecting as-built conditions. The reanalysis considered both as-built conditions and support stiffness. The stress results reported were obtained using B31.1-1973 Summer Addenda, Formula 12. Stress allowables given are based on the stress limits given in Table 3.9-1. The line was found to be seismically qualified.

3.9.2.1.2.4 Steam Line From Steam Generator B to Containment

Original Dynamic Analysis

A dynamic modal analysis was originally run on the steam line of loop B. The ground response spectrum was modified to factor in building effects. It was found that the previous static analysis of the steam line that used the peak of the response curve for 0.5% critical damping gave a very conservative estimate of inertially induced stresses. In order to account for the relative support movements, a separate stress analysis was run on the piping system. This analysis indicated a stress of 8500 psi, which was combined with the maximum thermal stress in the steam line of 11,000 psi. These combined secondary stresses are below the allowable stress of 20,600 psi.

IE Bulletin 79-07 Reanalysis

For the IE Bulletin 79-07 reanalysis, the line analyzed extended from steam generator 1B to the containment penetration. Seismic results were originally reported in *Reference 4*. A seismic reanalysis of this line was performed using the Westinghouse proprietary computer code WESTDYN.

The WESTDYN dynamic model reflected the as-built conditions as well as the actual support stiffness. The main steam line analyzed was coupled to a reactor coolant loop B model. In Table 3.9-3 is a comparison of stress results from the reanalysis reflecting as-built conditions, support stiffness, and the allowable stresses. The stress results reported were obtained using

B31.1-1973 Summer Addenda, Formula 12. Stress allowables given are based on the stress limits given in Table 3.9-1. The line was found to be qualified seismically.

3.9.2.1.2.5 Charging Line

IE Bulletin 79-07 Reanalysis

For the IE Bulletin 79-07 reanalysis, the lines analyzed extended from charging pumps 1, 2, and 3 to the charging pump discharge filter; and included the 2 and 3-in. discharge lines from the filter and the 3-in. bypass. A seismic analysis was originally performed of this line by the M. W. Kellogg Company. A seismic reanalysis of this line was performed using the Westinghouse proprietary computer code WESTDYN.

The WESTDYN dynamic model reflected the as-built conditions as well as the actual support stiffness. Table 3.9-4 is a comparison of stress results from the reanalysis reflecting as-built conditions, support stiffness, and the allowable stresses. The stress results reported were obtained using B31.1-1973 Summer Addenda, Formula 12. Stress allowables given were based on the stress limits given in Table 3.9-1. The line was found to be seismically qualified.

3.9.2.1.3 Seismic Category I Piping, 2-Inch Nominal Size and Under, Original Design

The pipes falling in this category were field erected (*Reference 4*). The large majority of these pipes has lateral and vertical support spacing selected in accordance with that suggested by USAS B31.1 for vertical supports. The piping so supported can be considered rigid with respect to the buildings in which they are housed. The pipes are subjected to the building acceleration only at the points of support without any further appreciable amplification. Conservative calculations show that the largest building amplification of ground acceleration is about 4. This gives inertial loads of 0.8g.

Simple beam calculations performed for the three pipe sizes falling in this category (i.e., 2 in., 1 in., and 3/4 in.) and for the typical schedules adopted for these pipes (i.e., Schedules 10, 40, 80, and 160 for stainless steel pipes and Schedules 40, 80, and 160 for carbon steel pipes) indicated that the stress levels were significantly lower than the allowable values.

3.9.2.1.4 Pressurizer Safety and Relief Valve Discharge Piping

3.9.2.1.4.1 1972 Analysis

In response to a request from the NRC for additional information in 1972 (*Reference 5*), dynamic analyses were performed for the pressurizer safety valve discharge piping.

The pressurizer safety valve piping system is a closed system and no sustained reaction force from a free discharging jet of fluid exists. Transient hydraulic loads can be imposed at various points of the piping system from the time a safety relief line begins to open until steady flow is completely developed. Calculations were performed (*Reference 22*) to provide a time-history of such loads acting on each straight leg of pipe from the safety valve downstream to the relief tank header. The FLASH IV digital computer program was employed in performing these calculations. Frictional losses were included for the piping and the associated elbows. The time-history hydraulic forces were determined based on several loop seal temperatures.

The natural frequencies and mode shapes of the system were solved using program WESTDYN. The calculated loop seal temperature for Ginna Station with a 3-in.-thick insulated water loop was 330°F. The hydraulic forces assuming a 300°F water temperature were applied to the structural dynamic model at each change in flow direction throughout the system. This constituted a truly impulsive dynamic analysis with simultaneous contributions from all the dynamic modes of the system.

The piping systems for PCV 434 and PCV 435, were represented by lumped mass models as shown in Figures 3.9-3 and 3.9-4. The time-history analysis was performed by the mode superposition method using computer programs WESTDYN, FIXFM, and WESTDYN-2. The stresses from the deadweight, pressure, seismic, and transient hydraulic load analyses were calculated separately. It was conservatively assumed that the maximum stress around the pipe circumference occurs at the same point for all load cases considered. These stresses were added absolutely and compared with the code allowable stress limit of 1.2 x S_a, where S_a = stress allowable. A review of the analysis showed that the stress levels in the pressurizer safety valve Class 1 and Class 2 piping systems were within the allowable design requirements of USAS B31.1.

3.9.2.1.4.2 NUREG 0737, Item II.D.1 Analysis

Under NUREG 0737, Item II.D.1, it was requested that the functionability and structural integrity of the as-built pressurizer safety and relief valve discharge piping system be demonstrated on a plant-specific basis. In response to the NRC request Westinghouse performed (*Reference 2*) an analysis of the pressurizer safety and relief valve discharge piping system.
Additional information was supplied in *References 23*, *24*, and *25*.

A water seal is maintained upstream of the pressurizer safety valves. The water slug, driven by high pressure steam upon actuation of the valves, generates severe hydraulic shock loads on the piping and supports. The pressurizer safety valves and Pressurizer Power Operated Relief Valves (PORV) are provided with a reflective insulation system that adds pressurizer radiant heat to the loop seal piping. This maintains the safety valve water seals at elevated temperatures such that the loop seal contents exiting the valve nozzles are converted to steam, which reduces the loads on the piping and supports.

NUREG 0737, Item II.D.1, required testing to qualify the reactor coolant system and safety valves under effected operating conditions and transients. When the pressurizer pressure reaches the safety valve set pressure of 2500 psia and the valve opens, the high-pressure steam in the pressurizer forces the water in the water loop seal through the valve and down the piping system to the pressurizer relief tank. Additionally, when the relief valve set pressure of 2350 psia is reached and the valve opens, high-pressure steam is discharged to the downstream piping.

The computer code ITCHVALVE was used to perform the transient hydraulic analysis for the system (*Reference 2*). One-dimensional fluid flow calculations applying both the implicit and explicit characteristic methods were performed. The piping network was input as a series of single pipes, generally joined together at one or more places by two or three-way junctions.

Each of the single pipes included associated friction factors, angles of elevation, and flow areas.

Unbalanced forces were calculated for each straight segment of pipe from the pressurizer to the relief tank using program FORFUN. The time-histories of these forces were used for the subsequent structural analysis of the pressurizer safety and relief lines.

The safety and relief lines were modeled statically and dynamically. The mathematical model used for dynamic analyses was modified for the valve thrust analysis to represent the safety and relief valve discharge. The time-history hydraulic forces determined by FORFUN were applied to the piping system lump mass points. The dynamic solution for the valve thrust was obtained by using a modified predictor-corrector-integration technique and normal mode theory.

The piping between the pressurizer nozzles and the pressurizer relief tank was analyzed according to the requirements of the appropriate equations of the ANSI B31.1-1973 Code through the 1973 addenda. The allowable stresses for use with the equations were determined in accordance with the requirements of the ANSI Code. The load combinations and acceptance criteria defined in Tables 3.9-5, 3.9-6, and 3.9-7 were used in the analysis.

The piping stress analysis considered all pertinent loadings that result from thermal expansion, pressure, weight, earthquake, and transient hydraulic effects.

The transfer matrix method and stiffness matrix method were used to obtain a piping deflection solution. All static and dynamic analyses were performed using the WESTDYN computer program. It was determined that the operability and structural integrity of the system were ensured for all applicable loadings and load combinations including all pertinent safety and relief valve discharge cases.

3.9.2.1.5 Main Steam Header Dynamic Load Factor Analysis

In response to a request from the NRC for additional information in 1972 (*Reference 5*), dynamic analysis was performed for the main steam header.

In the original design of Ginna Station, the main steam header (case 2) was analyzed for the internal loads generated by the safety valve during the relieving process by modeling the system as a single degree of freedom system and using a conservative dynamic load factor of 2.0 to account for the impact effects of the safety relief valve reaction. The magnitude of the thrust was based on the combined effects of static pressure at the safety valve discharge system and the momentum of the flowing steam. This analysis indicated that, for the Ginna Station main steam header, the maximum upper bound load factors were 1.15 and 1.50 for a single and multiple valve discharge, respectively. In calculating the dynamic load factor, the analysis accounted for the contri-butions to the piping response given by all the significant vibrational modes of the structure for a single valve and multiple valve discharge. The report concluded that the valve/header design was conservative based on a calculation of the actual dynamic upper bound values of the dynamic load factor. The effects of multiple safety valve discharges should be considered since the analysis showed a possible 30% increase in load factor due to actuation of a second valve. The actual load factor achieved in the system was

expected to be significantly lower than the upper bound values predicted since damping reduced the maximum contribution from each mode; and for multiple valve discharge the time between valve discharges had to be exactly equal to a period of one of the primary modes for the maximum response to occur.

3.9.2.1.5.1 Extended Power Uprate Considerations

Additional analysis was developed in support of *Reference 31* to consider potential hydraulic transients that may be developed as a result of the Ginna Extended Power Uprate.

3.9.2.1.6 Secondary System Water Hammer

3.9.2.1.6.1 Analysis

In response to an NRC request regarding secondary system fluid flow instabilities (water hammer), RG&E performed an analysis of the potential for occurrence and potential consequences of water hammer at Ginna Station (*Reference 3*). Analyses of the main feedwater piping were performed for postulated water hammer utilizing a dynamic forcing function. These analyses assumed that a steam-water slugging process was initiated at the steam generators, that the steam generator level was being recovered utilizing auxiliary feedwater, and that the main feedwater check valves were closed. The analyses were based on the piping configuration and supports installed at Ginna Station at the time of analyses.

An examination was made of the normal, abnormal, and accident transients which could result in a steam generator water level below the feed ring long enough for it to drain; and which would result in feedwater flow being initiated in order to recover level. It was determined that the following operating occurrences could cause these conditions:

a. Load changes when the steam generator level was under manual control.
b. Intermittent manual operation of auxiliary feedwater pumps to maintain steam generator level during MODE 3 (Hot Shutdown).
c. Loss of main feedwater.

The main feedwater piping at Ginna Station consists of two lines, A and B, which run from the control valve station in the turbine building to the steam generators.

The auxiliary feedwater piping at Ginna Station consists of six lines: two from the motor-driven auxiliary feedwater pumps (MDAFW) 1A and 1B, two from the turbine-driven auxiliary feedwater pump (TDAFW), and two from the standby auxiliary feedwater pumps (SAFW).

The forcing function used for the analyses is shown in Figure 3.9-1. The forcing function is a time-dependent mathematical quantity representative of the energy released by water hammer in the feedwater piping connected to PWR steam generators. The forcing function provides a time-history of the pressure in the piping system which results from the acoustic shock wave generated by a steam-water slug. The forcing function shown in Figure 3.9-1 was modified for the specific piping configuration at Ginna.

This forcing function was derived by Westinghouse from measurements of pressure and displacement observed during a water hammer test at the Tihange site in Belgium. Calculations performed by Westinghouse employing this forcing function for the Tihange feedwater piping resulted in displacements in fair agreement with those observed. Westinghouse considered the forcing function as preliminary and it was still under development at the time the analyses were performed.

The loading combinations and stress criteria used in evaluating the results of the analyses were based on the original construction code, ANSI B31.1, Power Piping. These criteria were that the sum of the longitudinal stresses due to pressure, weight, and water hammer would not exceed 1.2 times the allowable stress in the hot condition, S_h.

3.9.2.1.6.2 Evaluation Results

Evaluation of the stresses obtained in the analyses showed that inside the containment there were several locations on the A main feedwater piping and several locations on the B main feedwater piping which exceeded the stress criteria. Outside the containment there were no locations on the A main feedwater piping and several locations on the B main feedwater piping which exceeded the stress criteria. Analyses were not performed for the auxiliary feedwater piping systems for a postulated water hammer from the steam generators.

3.9.2.1.6.3 Corrective Actions

Various administrative controls, steam generator mechanical modifications, and piping support modifications were evaluated to determine their effectiveness in either preventing the occurrence of water hammer, or reducing its consequences should it occur. In evaluating these changes, the effect of other changes that were being made to the plant and the overall reliability and integrity of the steam generators were also considered.

It was determined that the best alternative available for precluding water hammer was installation of J-shaped discharge tubes on top of the feed rings and plugging of the bottom holes in the rings to provide for top discharge of water rather than bottom discharge. See Section 10.3.2.2.

In 1996, Ginna Station replaced the steam generators. The replacement steam generators incorporated many of the guidelines from NRC Branch Technical Position ASB-10-2, “Design Guidelines for Avoiding Water Hammers in Steam Generators,” to minimize the potential and consequence of waterhammer in the feedwater system. Specifically, the BWI replacement steam generators are designed to minimize the potential for a steam pocket forming in the feed header using top discharge J-tubes in the feed ring, internals which maximize secondary water inventory above the feed ring, and an all-welded thermal sleeve/internal feed header assembly that eliminates the possibility of steam leakage into the feed ring through sleeve/header mechanical joints. The BWI design is also less prone to serious consequences from a steam pocket forming because of the feed header gooseneck which tends to retard rapid condensation and water-slug acceleration better than a horizontal header run would.

3.9.2.1.6.4 Extended Power Uprate Considerations

Additional analysis was developed in support of Reference 31 to consider potential hydraulic transients that may be developed as a result of the Ginna Extended Power Uprate.

3.9.2.1.7 Velan Swing Check Valves

In response to IE Bulletin 79-04, RG&E analyzed the effect of changes in weights previously assumed for swing check valves manufactured by Velan Engineering Corporation. There is one 6-in. Velan swing check valve installed in both low head safety injection system lines and four 3-in. valves installed in the high head safety injection system lines. The initial installation assumed a weight of 225 lb for the 6-in. valves and 60 lb for the 3-in. valves. The correct weights were 450 and 95 lb, respectively.

In order to investigate the effect of valve weight differences, Westinghouse performed seismic analyses on some representative configurations of the safety injection system and studied the effect of increasing valve weight by 100% on the pipe stresses and support loads of the line.

An operating-basis earthquake seismic analysis was performed for each case. It was a two-dimensional response spectrum analysis considering each horizontal direction separately, combined with the vertical direction. It was determined from the analysis that the increase in valve weight did not result in unacceptable pipe stress for the lines investigated.

3.9.2.1.8 Seismic Piping Upgrade Program

As a result of SEP preliminary seismic review of Ginna (SEP Topic III-6), the NRC IE Bulletin 79-14, and other NRC seismic requirements, RG&E initiated a seismic piping upgrade program described in Section 3.7.3.7. In order to conservatively respond to the SEP seismic review and possible future NRC seismic requirements, a set of analysis procedures and criteria that conform with current NRC review criteria were used for the piping analysis. These are discussed in Section 3.7.3.7. The loading combinations and associated stress limits used for the piping systems that are part of the seismic upgrading program are given in Table 3.9-8. Pipe rupture loads were not considered; as such, the stress limits used for the safe shutdown earthquake condition did not correspond to the faulted condition, as they could be for the safe shutdown earthquake evaluation, but to the emergency condition stress limits. The piping stresses were calculated using the formulas given in ANSI B31.1-1973, 1973 Summer Addenda. Thermal stresses were evaluated per ANSI B31.1-1973, Summer 1973 Addenda requirements.

The maximum loads that the main feedwater piping and steam line piping were permitted to transmit to the steam generator nozzles are given in Table 3.9-9.

The allowable loads for the seal injection and component cooling system nozzles on the reactor coolant pump and motor are listed in Table 3.9-10.

Two pipe lines from the upgraded piping systems were selected and analyzed independently by the NRC to verify the adequacy of the as-built design and confirm the upgrade analysis results. The pipe lines selected were portions of residual heat removal and safety injection

system piping. Audit analyses, which incorporated current ASME Code and Regulatory Guide criteria and used the floor response spectra as input motion, were performed for each portion of the piping system selected. The results from these analyses were compared to ASME Code requirements for Class 2 piping systems at the appropriate service conditions. This comparison provided the bases for assessing the structural adequacy of the piping under the postulated seismic loading condition. Assumptions made for the analysis, methodology employed and analysis results are found in *Reference 6*. The results from the confirmatory analysis showed that the sampled piping systems are capable of withstanding the postulated safe shutdown earthquake seismic input.

Structural members within the various buildings at Ginna Station were analyzed and were modified as required to accept new or recalculated pipe support loads from the seismic piping upgrade program and to transfer these loads into the main structural framing.

Pipe supports were analyzed as discussed in Section 3.9.3.3.

3.9.2.2 Safety-Related Mechanical Equipment

Mechanical equipment was originally seismically qualified by a combination of test and analysis. The methods of analysis used in the original analyses and during the SEP reevaluation are described briefly in Section 3.7.3. The results of the analysis are presented in this section.

3.9.2.2.1 Original Seismic Input and Behavior Criteria

For Seismic Category I mechanical equipment, all components and systems originally classified as Class I were designed in accordance with the criteria described in Section 3.7.1.1. All components of the reactor coolant system and associated systems were designed to the standards of the applicable ASME or USAS Codes. The loading combinations and behavior criteria not otherwise defined by the USAS and ASME Codes in use at the time of the original design, which were employed by Westinghouse in the design of the components of these systems, i.e., vessels, piping, supports, vessel internals and other applicable components, are given in Table 3.9-1. Table 3.9-1 also indicates the stress limits which were used in the design of the equipment for the various loading combinations. In addition, the supports for the reactor coolant system were designed to limit the stresses in the pipes and vessels to the stress limits given in Table 3.9-1.

Heat exchangers were designed in accordance with the criteria set forth in Section 3.7.1.1. The peak of the 0.5% critical damping response spectra corresponding to the 0.2g maximum potential earthquake was selected as the seismic design load. Stress limits were set equivalent to those of the pressure vessel codes and the structural steel standards of AISC.

The design of pumps (casing and shafting) was based not on stress criteria, but on deflection limits. For the case where efficiency was of minimum importance, deflection at the stuffing box controlled the design. For the case where efficiency was of importance, deflection of the shaft at the impeller wear rings controlled the design. In either case, the natural frequency (identical to critical speed) was approximately 20 Hz and 30 Hz for 1800 rpm and 3600 rpm machines, respectively, for flexible shafting. In reality, the stuffing boxes served as an additional bearing and the natural frequency was above that corresponding to the operating speed.

For stiff shafting, the fundamental frequency was above that corresponding to the operating speed (30 Hz and 60 Hz). Both the pump casings and the motor casings were extremely stiff when evaluated as simply supported beams with uniform load distribution. A typical natural frequency for a casing with a length-to-diameter ratio of 3 and a diameter of 36 in. was 100 Hz.

The combined pump-motor unit is mounted on a common bedplate which is grouted into the foundation. The stiffness of the foundation mass and the rigid bolting eliminated possible relative movement between the pump and motor under operating loads as the couping between the motor and pump was designed only to accommodate geometric misalignment.

The analysis of tanks was performed in the manner set forth in TID 7024, taking into account the possible dynamic effects resulting from the sloshing of the water. The techniques are set forth in Chapters 5 and 6 of TID 7024.

Shell stresses and support stresses are limited to those permitted in the pressure vessel codes and the structural steel standards of AISC.

Electric motor-operated valves were verified to be capable of sustaining a 1g shock load without interruption of circuitry or loss of function. This was verified up to 20 Hz.

3.9.2.2.2 Current Seismic Input

Current seismic input requirements for determining the seismic design adequacy of mechanical equipment are normally based on in-structure (floor) response spectra for the elevations at which the equipment is supported. The floor spectra used in the SEP reassessment, which are based on Regulatory Guide 1.60 spectra, are shown in Figures 3.7-12 through 3.7-28.

For mechanical equipment, a composite 7% equipment damping was used in the evaluation for the 0.2g safe shutdown earthquake.

3.9.2.2.3 Systematic Evaluation Program

Seismic Category I components that are designed to remain leaktight or retain structural integrity in the event of a safe shutdown earthquake are typically designed to the ASME Section III Code (ASME III), Class 1, 2, or 3 stress limits for Service Condition D. The stress limits for supports for ASME leaktight components are limited as shown in Appendix F or Appendix XVII to ASME III (1977).

When qualified by analysis, active ASME III components that must perform a mechanical motion to accomplish their safety functions typically must meet ASME III Class 1, 2, or 3 stress limits for Service Condition B. Supports for these components are also typically restricted to Service Condition B limits to ensure elastic low deformation behavior.

For other passive and active equipment, which are not designed to ASME III requirements, and for which the design, material, fabrication, and examination requirements are typically less rigorous than ASME III requirements, the allowable stresses for passive components are limited to yield values and to normal working stress (typically 0.5 to 0.67 yield) for active components.

The current behavior criteria used in various equipment and distribution systems for Ginna passive components are given in Table 3.9-11.

Experience in the design of such pressure retaining components as vessels, pumps, and valves to the ASME III requirements, at 0.2g zero period ground acceleration, indicates that stresses induced by earthquakes seldom exceed 10% of the dead weight and pressure-induced stresses in the component body (*Reference 7*). Therefore, design adequacy of such equipment is seldom dictated by seismic design considerations.

Seismically induced stresses in nonpressurized mechanical equipment and component supports may be significant in determining design adequacy.

3.9.2.2.4 Systematic Evaluation Program Reevaluation of Selected Mechanical Components for Design Adequacy

The Systematic Evaluation Program (SEP) Seismic Review Team selected mechanical and electrical components representative of items installed in the reactor coolant system and safe shutdown systems for review in order to develop conclusions as to the overall seismic design adequacy of Seismic Category I equipment installed at Ginna Station. The electrical equipment is listed in Table 3.10-2 and discussed in Section 3.10.2.1. The mechanical equipment is listed in Table 3.9-12 and the seismic analysis of these components is described in the following sections.

3.9.2.2.4.1 Essential Service Water (SW) Pumps

The essential service water (SW) pump and motor units are oriented vertically in the screen house and supported at elevation 253.5 ft. The intake portion of the pumps extend down from the discharge head and pump base a distance of approximately 36.5 ft, including the clip-on type basket strainer installed on the suction end bell.

The previous seismic analysis was performed for equivalent static loads of 0.32g acting simultaneously in one horizontal and the vertical direction.

The pump-motor units are located at grade; therefore, the seismic input used in SEP reevaluation was essentially the Regulatory Guide 1.60 ground response spectrum for 7% of critical damping. The pumps were evaluated for an inertial acceleration value considering peak response of 0.52g horizontal acceleration and 0.35g vertical acceleration. Overturning tensile and shear stresses in the pump base anchor bolts were determined as were stresses at the attachment of the intake column pipe to the discharge head.

Because the intake portion of the pumps are oriented vertically as cantilever beams, the dynamic characteristic of the intake suction pipes were determined. The intake suction pipes were found to have a fundamental frequency of 1.6 Hz based on a weight distribution that includes water in the shaft. Because of this natural frequency, the spectral acceleration used was the peak of Figure 3.7-4, 0.52g.

It was determined that a brace needed to be installed on the intake column pipes. With the brace, the stresses at the bolts would be 15,700 psi in tension and 7000 psi in shear, which would yield a minimum factor of safety in shear of 2.29 for ASME Condition D stress limits

for an assumed A307 bolt material. Also, the stresses calculated at the flange connecting the discharge head to the intake column pipes were well within allowable stresses. This modification was performed in 1984.

3.9.2.2.4.2 Component Cooling Heat Exchanger

The component cooling heat exchanger is a horizontal heat exchanger located in the auxiliary building and supported by two saddles at elevation 281.5 ft. One saddle is slotted in the longitudinal direction to permit thermal expansion. During the SEP reevaluation the previous analysis was reviewed and independent evaluation of the dynamic response characteristics of the heat exchanger and its saddle support system using the response spectra for 7% damping shown in Figure 3.7-21 was performed. The review indicated that the system was relatively rigid and had no response frequencies below 33 Hz. Thus, safe shutdown earthquake input horizontal seismic accelerations in the orthogonal directions used were 0.36g and 0.60g. The seismic stresses induced in the tubes and shell were determined, combined with other applicable loads, and compared to code allowables. The safety factor determined for the heat exchanger tube is 33.9 and that for the shell is 11.0.

Both the component cooling heat exchanger and the component cooling surge tank are supported by a complex structural steel framework. Evaluation of the fundamental frequencies of both the heat exchanger and the surge tank did not consider any flexibility of the structural steel support framing. It was assumed that the dynamic characteristics of this structural steel framing were included in the response spectra.

The anchor bolt stresses were also determined. The analysis established a factor of safety with respect to ASME Code-allowable stress limits of 1.41 for the anchor bolts. Therefore, it was concluded that the component cooling heat exchanger will withstand a 0.2g safe shut-down earthquake without loss of structural integrity.

3.9.2.2.4.3 Component Cooling Surge Tank

The component cooling surge tank is a horizontal component located in the auxiliary building and supported by two saddles at elevation 281.5 ft. For the SEP reevaluation the previous analysis was reviewed. In addition, independent evaluation of the structural characteristics of the surge tank and its support system using the response spectra for 7% damping shown in Figure 3.7-23 was performed. In the transverse (east-west) direction, the tank-support system was found to be rigid. However, it was determined that it was not completely anchored against sliding. As a result, the tank saddle supports were modified to provide restraint in the longitudinal direction.

The seismic forces in the transverse (east-west) direction developed from a 0.75g in-structural spectral acceleration were applied to the surge tank and the resulting tank, saddle, and anchor bolt stresses were determined. Factors of safety for the tank, saddle, and anchor bolts--loaded seismically in the transverse and vertical directions--were 125.5, 57.7, and 5.08, respectively.

3.9.2.2.4.4 Diesel-Generator Air Tanks

The diesel-generator air tanks are oriented vertically in the diesel-generator building and supported at grade elevation in a rock-supported structure.

The seismic input used for the SEP reevaluation was the Regulatory Guide 1.60 ground response spectrum for 7% of critical damping (Figure 3.7-4). The previous analysis to seismically qualify the tanks used a 0.2g safe shutdown earthquake ground response spectrum. The tanks are supported by a skirt structure and the combined tank-support system was found to have a fundamental frequency of 33 Hz. Therefore, the input acceleration used was 0.2g. The maximum calculated stress in the anchor bolts was approximately 0.28 ksi in shear, which yields a safety factor of 61.3 for A307 bolt material. The minimum safety factors in the tank body and skirt support were 4.43 and 3968, respectively.

3.9.2.2.4.5 Boric Acid Storage Tank

The boric acid storage tank is a column-supported tank. The tank, its support legs, and its anchors were reviewed to determine seismic design adequacy. The tank, which is supported at elevation 271 ft, was evaluated using the in-structure response spectra shown in Figure 3.7-24. The dynamic analysis considered the effective impulsive and convective response of the contained fluid. The fundamental response frequencies for the tank were calculated to be 17.2 Hz for tank-support system bending and shear deformation under impulsive loading (7% damping) and 0.56 Hz under convective loading (0.5% damping). The analysis established minimum factors of safety of approximately 41.7 for membrane stress in the tank, 6.20 for compressive stresses in the tank legs, and 4.65 for compressive stresses in the anchor bolts.

3.9.2.2.4.6 Refueling Water Storage Tank (RWST)

The refueling water storage tank (RWST) is a vertical vessel that is 81 ft high to the top of the cylindrical portion and 26.5 ft in diameter. The anchorage consists of thirty, 2.5-in. diameter A36 bolts. The tank was originally qualified according to TID 7024 assuming a safe shut-down earthquake ground acceleration of 0.2g (without vertical amplification) and assuming that it was supported at the ground floor (elevation 236 ft) of the auxiliary building.

In 1983, RG&E investigated the ability of the refueling water storage tank (RWST) to withstand dead weight and seismic forces (*Reference 8*). Analysis loads consisted of the dead weight of the tank and contents, and seismic loads in two horizontal and the vertical directions. The seismic loads were defined by the site specific ground response spectrum for R. E. Ginna as specified by Regulatory Guide 1.60. The full spectrum was used for the horizontal analysis. Two thirds of the full spectrum was used for the vertical analysis.

The dynamic response analysis followed the requirements of NUREG/CR-1161. Analysis of the convective (sloshing) horizontal response was performed using the conventional "rigid tank" assumptions. Tank flexibility and fluid-structure interaction was incorporated in the analysis of the impulsive (non-sloshing) horizontal response. Tank flexibility was incorporated in the vertical response analysis. A damping level of 0.5% was used for the convective horizontal response analysis. A 7% damping was used for the impulsive horizontal and vertical response analysis.

The acceptance criteria considered the following principal points:

a. Anchorage Stresses: These include the stresses in the bolts, brackets, and bracket welds. Allowables were calculated per ASME Section III, Subarticle NF 3300.

b. Tank Wall Material Stress: The axial, hoop, and shear stresses developed in the tank wall were compared to material allowables per ASME Section III, Subarticle NC 3800.

c. Tank Wall Buckling: The axial, hoop, and shear stresses developed in the tank wall were compared to experimentally derived buckling criteria.

The results of the analysis indicated that no modifications to the refueling water storage tank (RWST) were required and that the tank was capable of withstanding dead weight loads in combination with the (SEP) site specific postulated seismic event.

In 1992, RG&E responded to Generic Letter 87-02, Supplement 1 and Generic Letter 88-20, Supplement 4 (SQUG and seismic events issues). As part of this response, RG&E stated that a review of the RWST would be performed for response spectra based on a peak ground acceleration of 0.2g and a Regulatory Guide 1.60 shape.

As a result of subsequent seismic analysis, modifications were determined to be required. The modifications consisted of 16 equally spaced vertical stiffeners, a welded steel support skirt extending 360° around the tank at the operating floor of the auxiliary building, and a large number of 3" diameter pins set through the skirt and into the concrete floor. As a result of these modifications which were completed in 1996, the RWST is capable of resisting the higher seismic input loads associated with 0.2g peak ground acceleration.

3.9.2.2.4.7 Motor-Operated Valves

During the SEP reevaluation, calculations performed on randomly selected motor-operated valves (2-in., 3-in., and 4-in. diameter) in the Ginna plant demonstrated that stress levels were in excess of the guideline value of 10% stress levels of ASME III, Class 2, Condition B for active valves and Condition D when pressure boundary integrity was required.

It was recommended that RG&E evaluate the seismic stresses induced by motoroperated valves in supporting pipe that is 4 in. in diameter and smaller and show that stresses resulting from motor operator eccentricity are less than 10% of the service Condition B code-allowable stresses. Rochester Gas and Electric explicitly modeled motor-operated valves in the as-built installation as part of the Seismic Piping Upgrade Program and either found the stresses to be acceptable or modified the supports. The Seismic Piping Upgrade Program is discussed in Sections 3.7.3.7 and 3.9.2.1.8.

Additionally, in accordance with the motor-operated valve program, as described in the Ginna Station Motor-Operated Valve Qualification Program Plan, the impact of design basis seismic events is evaluated and identified for susceptible components of each motor-operated valve under the requirements of NRC Generic Letter 89-10. (See Section 5.4.9.3.)

3.9.2.2.4.8 Steam Generators

In 1975, a generic stress report was written which contained updated analyses of most areas of the steam generator that are subject to external loads, i.e., primary nozzles, feedwater nozzle, steam nozzle, and lower support pads. The updated stress report also contained an analysis of the tubes, swirl vanes, and feedwater ring. Calculated stress intensities were

compared with the ASME III design condition allowable levels for an operating-basis earthquake and the emergency condition allowable levels for a safe shutdown earthquake.

A detailed seismic analysis was not performed during the SEP reevaluation, but a comparison of the seismic input used in the original design of Ginna Station with that determined from the in-structure response spectra was used as a criterion for qualification.

Since the fundamental frequency of the steam generator was found to be below 10 Hz, the peak acceleration in both the north-south and east-west directions is 0.60g (see Figures 3.7-15 through 3.7-18) and the square root of the sum of the squares value for two horizontal components is 0.85g. Since the original horizontal response spectra used for the design of the steam generator had a minimum spectral acceleration of 2.0g for the safe shutdown earthquake condition, the seismic stresses resulting from use of the Ginna reassessment response spectra would be less than the stress values from the original analysis. The steam generator components were determined adequate by the 1975 analysis.

In 1996, the steam generators were replaced. Seismic evaluation of the primary and secondary side pressure boundaries demonstrate that these components satisfy ASME III Class 1 design requirements for Service Levels A, B, C and D.

3.9.2.2.4.9 Reactor Coolant Pumps

In the original design of Ginna Station, a static seismic load stress analysis was performed for the pumps. The safe shutdown earthquake analysis used 0.8g horizontally and 0.54g vertically. The stresses and deformations resulting from these loads were then combined with the dead weight and other normal operating loads to determine the total stresses in the motor, support stand cylinder, flange welds, support stand bolts, and main flange bolts. This analysis also contained evaluations of the pump support feet, primary nozzles, and casing for seismic plus normal operating loads. The stresses calculated in these analyses were compared with ASME III allowables.

A detailed seismic analysis was not performed for the SEP reevaluation. Instead, a comparison of the input acceleration with that used in the earlier analysis was used to check the adequacy of the reactor coolant pump.

For the SEP reevaluation, in-structure response spectra for the reactor coolant pump given in Figures 3.7-19 and 3.7-20 were used. For the peak spectral acceleration of 0.55g for both the north-south and east-west directions, the square root of the sum of the squares value was 0.78g, and the ratio of this value to the original design value of 0.8g was 0.97. The pump input acceleration was less than that considered in the 1968 analysis and therefore the pumps were considered adequate based on the original generic analysis.

3.9.2.2.4.10 Pressurizer

The pressurizer is a vertical cylindrical vessel with a skirt type support attached to the lower head. The lower part of the skirt terminates in a bolting flange where 24 1.5-in. bolts secure the vessel to its foundation. In 1969, a generic seismic analysis of the pressurizer shell, support skirt, support skirt flange, and pressurizer support bolts was performed. The weight of

the largest pressurizer (1800 ft ^{3}) was used instead of the actual operating weight of the Ginna pressurizer (800 ft ^{3}). In the safe shutdown earthquake evaluation, accelerations were applied statically at the center of gravity of the 1800 ft ^{3} model: 0.48g in the horizontal direction and 0.32g in the vertical direction. ASME III upset condition allowable levels were used for safe shutdown earthquake load cases.

In 1973, a more detailed evaluation was performed of the pressurizer skirt and shell (*Reference 9*). For that evaluation the loads applied to the skirt were equivalent to 10 times the operating-basis earthquake loads and 14 times the safe shutdown earthquake loads used in the 1969 evaluation. The results contained the primary membrane and bending stresses.

The pressurizer heaters were qualified generically for the 51 Series Pressurizer (*Reference 9*). The heaters in the 800-ft ^{3}pressurizer are shorter than those qualified but are otherwise identical. The qualification procedure used an equivalent static load of 37.5g for the safe shutdown earthquake condition. The fundamental frequency of the heater rods was found to be greater than 33 Hz.

The in-structure response spectra were used in the SEP reevaluation of the pressurizer as shown in Figure 3.7-12. Since the fundamental frequency of the pressurizer may be as low as 3 Hz, peak spectral accelerations were used: 0.55g for the north-south direction and 0.60g for the east-west direction. The square root of the sum of the squares value is 0.81g, and the ratio of this value to the original design value of 0.48g is 1.7. Based on the primary stress resultants of the 1973 analysis, the seismic input of 0.81g is well within the design limits presented in *Reference 9*.

3.9.2.2.4.11 Control Rod Drive Mechanism

The response spectra for the SEP reevaluation of the control rod drive mechanisms are given in Figures 3.7-13 and 3.7-14. Assuming the fundamental frequency of the drive mechanism as less than 12.5 Hz, the peak spectral acceleration in both the north-south and east-west directions was 0.60g and the square root of the sum of the squares value was 0.85g and this square root of the sum of the squares value is greater than the design value of 0.8g used in the original analysis. As noted in the NRC safety evaluation report on SEP Topic III-6 (*Reference 10*) the Westinghouse analysis was found to have utilized correct loadings and that the stresses are well within acceptable levels.

3.9.2.3 Dynamic Response Analysis of Reactor Internals Under Operational Flow Transients and Steady-State Conditions

Sections 3.9.2.3.1 through 3.9.2.3.5 reflect information resulting from the original analyses of the Ginna Station reactor vessel internals under dynamic loading conditions. It is preserved here for historical information. In anticipation of Extended Power Uprate (EPU), the dynamic response of the internals was reanalyzed (*Reference 31*). This reanalysis incorporated leak-before-break technology as allowed by 1972 General Design Criteria GDC-4.

Consequently, double-ended RCS breaks could be removed from the design basis for the reactor vessel internals (*Reference 32*). This reanalysis is discussed further in Section 3.9.2.3.6.

3.9.2.3.1 Design Criteria

3.9.2.3.1.1 General

The criteria for acceptability is that the core should be coolable and intact following a pipe rupture up to and including a double-ended rupture of the reactor coolant system. This implies that core cooling and adequate core shutdown must be ensured. Consequently, the limitations established on the internals are concerned principally with the maximum allowable deflections and/or stability of the parts.

3.9.2.3.1.2 Critical Internals

Upper Barrel

The upper barrel deformation has the following limits. To ensure reactor trip and to avoid disturbing the rod cluster control assembly guide structure, the barrel should not interfere with any guide tubes. This condition requires a stability check to assure that the barrel will not buckle under the accident loads. The minimum distance between guide tube and barrel is 10 in. This figure is adopted as the limit beyond which proper function can no longer be guaranteed. An allowable deflection of 5 in. has been selected.

Rod Cluster Control Assembly Guide Tubes

The rod cluster control assembly guide tubes in the upper core support package has the following allowable limits. The maximum horizontal transient deflection as a beam shall not exceed 1 in. over the length of the guide tube. The no loss of function limit is 1.5 in. Tests on guide tubes show that when the transverse deflection of the guide tube becomes significant, the cross section of the rod cluster control assembly guide tube changes. A maximum allowable transient transverse deflection of 1.0 in. has been established for the blow-down accident. Beam deflections above these limits produce cross section changes with increasing delay in scram time until the control rod will not scram due to interference between the rods and the guide. With a maximum transient transverse deflection of 1.5 in., the cross section distortion will not exceed 0.072 in. after load removal. This cross section distortion allows control rod insertion. For a maximum transient transverse deflection of 1.0 in., a cross section distortion not in excess of 0.035 in. is anticipated.

Fuel Assemblies

The limitations for this case are related to the stability of the thimbles at the upper end. During the accident, the fuel assembly will have a vertical displacement and could impact the upper and lower packages subjecting the components to dynamic stresses.

The upper end of the thimbles shall not experience stresses above the buckling compressive stresses because any buckling of the upper end of the thimbles will distort the guide lines and could affect the fall of the control rod.

Upper Package

The maximum allowable local deformation of the upper core plate where a guide tube is located is 0.100 in. This deformation will cause the plate to contact the guide tube since the

clearance between plate and guide tube is 0.100 in. This limit will prevent the guide tubes from being put in compression. In order to maintain the straightness of the guide tube a maximum allowable total deflection of 1 in. for the upper support plate and deep beam has been established. The corresponding no loss of function deflection is above 2 in.

3.9.2.3.1.3 Allowable Stress Criteria

The allowable stress criteria fall into two categories dependent upon the nature of the stress state: membrane or bending. A direct state of stress (membrane) has a uniform stress distribution over the cross section. The allowable (maximum) membrane or direct stress is taken to be equal to the stress corresponding to 0.2 of the uniform material strain or the yield strength, whichever is higher. For unirradiated 304 stainless steel at operating temperature the stress corresponding to 20% of the uniform strain is:

(S_m) allowable = 39,500 psi

For irradiated materials, the limit stress is higher.

For a bending state of stress, the strain is linearly distributed over a cross-section. The average strain value is, therefore, one half of the outer fiber strain where the stress is a maximum. Thus, by requiring the average strain to satisfy an allowable criterion similar to that for the direct state of stress, the outer fiber strain may be 0.4 times the uniform strain. The maximum allowable outer fiber bending stress is then taken to be equal to the stress corresponding to 40% of the uniform strain or the yield strength, whichever is higher. For unirradiated 304 stainless steel at operating temperature, we obtain from the stress strain curve:

(S_b) allowable = 50,000 psi

For combinations of membrane and bending stresses, the maximum allowable stress is taken to be equal to the stress corresponding to the maximum outer fiber strain not in excess of 40% uniform strain and average strain not in excess of 20% uniform strain.

3.9.2.3.2 Blowdown and Force Analysis

3.9.2.3.2.1 Computer Program

The MULTIFLEX computer code (*References 11*, *12*) calculates the thermal-hydraulic transient within the RCS and considers subcooled, transition, and early two-phase (saturated) blowdown regimes. The code employs the method of characteristics to solve the conservation laws, assuming one-dimensional flow and a homogeneous liquid-vapor mixture. The RCS is divided into subregions in which each subregion is regarded as an equivalent pipe. A complex network of these equivalent pipes is used to represent the entire primary RCS.

The following operating conditions were considered in establishing the limiting temperatures and pressures for the Ginna Station LOCA hydraulic forces analyses:

- Initial RCS conditions associated with a minimum thermal design flow of 85,100 gpm per loop.
- Uprated core power of 1811 MWt (analyzed NSSS power of 1817 MWt).

- A nominal RCS hot full power (HFP) T_{AVG} range of 564.6°F to 576.0°F. This provides an RCS Tcold range of 528.3°F to 540.2°F.
- An RCS temperature uncertainty of ±4°F.
- A feedwater temperature range of 390.0°F to 435.0°F.
- A nominal RCS pressure of 2250 psia.
- A pressurizer pressure uncertainty of ±60 psi.

Based on these conditions, the LOCA forces were generated at a minimum Tcold of 524.3°F, including uncertainty, and a pressurizer pressure of 2310 psia, including uncertainty.

The hydraulic forcing functions that occur as a result of a postulated LOCA are calculated assuming a limiting break location and break area. The limiting break location and area vary with the RCS component under consideration, but historically the limiting postulated breaks are a limited displacement reactor pressure vessel (RPV) inlet/outlet nozzle break or a double-ended guillotine (DEG) reactor coolant pump (RCP)/steam generator (SG) inlet/outlet nozzle break. General Design Criterion 4 (GDC-4) allows main coolant piping breaks to be "excluded from the design basis when analyses reviewed and approved by the Commission demonstrate that the probability of fluid system piping rupture is extremely low under conditions consistent with the design basis for the piping." This exemption is generally referred to as leak-before-break (LBB).

Furthermore, Constellation Generation Group had requested Westinghouse to exempt all the 10-inch piping connections to the RCS from the dynamic analysis of pipe break loads. Therefore, the next limiting RCS break sizes less than 10-inch diameter are the smaller auxiliary (or branch) lines connected to the RCS. The smaller branch line breaks analyzed for hydraulic forces are the 3-inch pressurizer spray line in the cold leg, the 4-inch upper plenum injection nozzle on the vessel, and the 2-inch safety injection line connection to the hot leg. The 4-inch pressurizer safety valve line on top of the pressurizer was not considered for the Forces analysis because the Forces analysis tracks the acoustic wave propagating through the subcooled fluid of the RCS, while the break for the safety valve line would occur in the voided region of the pressurizer. It would, therefore, be non-limiting as compared to breaks modeled in either the cold or hot legs of the RCS.

The only exception to the use of auxiliary line breaks for structural qualification is the modeling of a limited displacement double-ended guillotine reactor vessel outlet nozzle (RVON) break to demonstrate control rod insertion following a LOCA.

3.9.2.3.2.2 Blowdown Model

The MULTIFLEX computer code calculates the thermal-hydraulic transient within the RCS and considers subcooled, transition, and early two-phase (saturated) blowdown regimes. The code employs the method of characteristics to solve the conservation laws, assuming one-dimensional flow and a homogeneous liquid-vapor mixture. The RCS is divided into subregions in which each subregion is regarded as an equivalent pipe. A complex network of these equivalent pipes is used to represent the entire primary RCS.

The reanalysis performed in support of the Extended Power Uprate has made use of the MULTIFLEX computer code. MULTIFLEX is an extension of the BLODWN-2 computer code and includes mechanical structure models and their interactions with the thermal-hydraulic system. Both versions of the MULTIFLEX code share a common hydraulic modeling scheme, with differences being confined to a more realistic downcomer hydraulic network and a more realistic core barrel structural model that accounts for non-linear boundary conditions and vessel motion. Generally, this improved modeling results in lower, more realistic, but still conservative hydraulic forces on the core barrel. The NRC staff has accepted (*Reference 13*) the use of MULTIFLEX 3.0 for calculating the hydraulic forces on reactor vessel internals (*Reference 14*).

A coupled fluid-structure interaction is incorporated into the MULTIFLEX code by accounting for the deflection of the constraining boundaries, which are represented by separate spring-mass oscillator systems. For the reactor vessel/internals analysis, the reactor core barrel is modeled as an equivalent beam with the structural properties of the core barrel in a plane parallel to the broken inlet nozzle. Mass and stiffness matrices that are obtained from an independent modal analysis of the reactor core barrel are applied in the equations of structural vibration at each of the mass point locations. Horizontal forces are then calculated by applying the spatial pressure variation to the wall area at each of the elevations representative of the mass points of the beam model. The resultant core barrel motion is then translated into an equivalent change in flow area in each downcomer annulus flow channel. At every time increment, the code iterates between the hydraulic and structural subroutines of the program at each location confined by a flexible wall. For the reactor pressure vessel and specific vessel internal components, the MULTIFLEX code generates the LOCA pressure transient that is input to the LATFORC and FORCE2 post-processing codes (*Reference 11*). These codes, in turn, are used to calculate the actual forces on the various components.

3.9.2.3.2.3 LATFORC MODEL

The LATFORC computer code employs the field pressures generated by MULTIFLEX code, together with vessel geometric information (component radial and axial lengths), to determine the horizontal forces on the vessel wall and core barrel. The LATFORC code represents the downcomer region with a model that is consistent with the model used in the MULTI-FLEX blowdown calculations. The downcomer annulus is subdivided into cylindrical segments, formed by dividing this region into circumferential and axial zones. The results of the MULTIFLEX/LATFORC analysis of the horizontal forces are calculated for the initial 500 msec of the blowdown transient and are stored in a computer file. These forcing functions, combined with vertical LOCA hydraulic forces, seismic, thermal, and flow-induced vibration loads, are used by the cognizant structural groups to determine the resultant mechanical loads on the reactor pressure vessel and vessel internals.

3.9.2.3.2.4 FORCE2 MODEL

The FORCE2 computer code calculates the hydraulic forces that the RCS coolant exerts on the vessel internals in the vertical direction. The FORCE2 code uses a detailed geometric description of the vessel components and the transient pressures, mass velocities, and densities computed by the MULTIFLEX code. The analytical basis for the derivation of the mathematical equations employed in the FORCE2 code is the one-dimensional conservation

of linear momentum. Note that the computed vertical forces do not include body forces on the vessel internals, such as deadweight or buoyancy. When the vertical forces on the reactor pressure vessel internals are calculated, pressure differential forces, flow stagnation forces, unrecoverable orifice losses, and friction losses on the individual components are considered. These force components are then summed together, depending upon the significance of each, to yield the total vertical force acting on a given component. The results of the MULTIFLEX/ FORCE2 analysis of the vertical forces are calculated for the initial 500 msec of the blowdown transient and are stored in a computer file. These forcing functions, combined with horizontal LOCA hydraulic forces, seismic, thermal, and flow-induced vibration loads, were used in the structural evaluations to determine the resultant mechanical loads on the vessel and vessel internals.

3.9.2.3.3 Fuel Assembly Thimbles

When the core moves vertically it can impact the upper and lower core plates, which subjects the thimbles to compressive impact stresses. These stresses were obtained from the maximum dynamic impact forces on the fuel assemblies. The maximum impact load applied to the thimbles by the fuel elements was 2,132 lbs. The maximum axial stress was 11,660 psi. Buckling stresses result from the impact load of the fuel assembly onto the lower core plate. This load is distributed through the grids to the thimbles as drag force proportional to the drag force available at each grid. The largest fraction of the load is reacted at the bottom grid because the bottom grid is the highest force grid. The spans that would be considered in this event are the lowest spans. However this design has the tube-in-tube dashpost in those spans, which reinforces them. Therefore the critical span becomes the span where the dashpot tube ends, which has a buckling stress of 4,248 psi and an allowable buckling stress of 7,551 psi (for ZIRLO^{TM} with a yield stress of 18,520 psi at operating temperature). Therefore the distortion will not exceed the allowable limits, and it is concluded that the capability of the control rod insertion is maintained.

3.9.2.3.4 Dynamic System Analysis of Reactor Internals Under Loss-of-Coolant Accident (LOCA)

The response of reactor internals components due to an excitation produced by complete severance of a branch line pipe is analyzed. Assuming a pipe break occurs in a very short period of time of 1 msec, the rapid drop of pressure at the break produces a disturbance which propagates along the primary loop and excites the internal structures.

The LOCA breaks considered for the Ginna Station consist of breaks located at the 3-inch pressurizer spray scoop break and the 4-inch upper plenum injection (UPI) break. The LOCA hydraulic forcing functions (horizontal and vertical forces) that were used in the analyses were generated using MULTIFLEX 3.0 computer code described by Takeuchi, et al (WCAP-9735, Rev. 1, "Multiflex 3.0-A FORTRAN IV Computer Program for Analyzing Thermal-Hydraulic-Structural System Dynamics (III) Advanced Beam Model."

3.9.2.3.4.1 Mathematical Model of the Reactor Pressure Vessel (RPV) System

The mathematical model of the RPV system is a three-dimensional, non-linear finite element model which represents dynamic characteristics of the reactor vessel/internals/fuel in the six

geometric degrees of freedom. The RPV system model was developed using the WECAN computer code (Westinghouse Electric Computer Analysis). The WECAN finite element model consists of three concentric structural sub-models connected by non-linear impact elements and stiffness matrices. The first sub-model represents the reactor vessel shell and associated components. The reactor vessel is restrained by reactor vessel supports and by the attached primary coolant piping. The reactor vessel support system is represented by stiffness matrices.

The second sub-model represents the reactor core barrel assembly (core barrel and thermal shield), lower support plate, tie plates, and secondary core support components. This sub-model is physically located inside the first, and is connected to it by a stiffness matrix at the internals support ledge. Core barrel to vessel shell impact is represented by non-linear elements at the core barrel flange, core barrel nozzle, and lower radial support locations.

The third and innermost sub-model represents the upper support plate, guide tubes, support columns, upper and lower core plates, and the fuel. This sub-model includes the specific properties of the Westinghouse 14x14 422 V+ Fuel. The third sub-model is connected to the first and second by stiffness matrices and non-linear elements.

The WECAN computer code, which is used to determine the response of the reactor vessel and its internals, is a general purpose finite element code. In the finite element approach, the structure is divided into a finite number of members or elements. The inertia and stiffness matrices, as well as the force array, are first calculated for each element in the local coordinates. Employing appropriate transformation, the element global matrices and arrays are then computed. Finally, the global element matrices and arrays are assembled into the global structural matrices and arrays, and used for dynamic solution of the differential equation of motion for the structure:

$$[M]\{\ddot{U}\} + [D]\{u'\} + [K]\{U\} = \{F\}$$

where $[M]$ = Global Inertia Matrix

$[D]$ = Global Damping Matrix
$[K]$ = Global Stiffness Matrix
$\{\ddot{U}\}$ = Acceleration Array
$\{u'\}$ = Velocity Array
$\{U\}$ = Displacement Array
$\{F\}$ = Force Array, including impact, thrust forces hydraulic forces, constraints and weight

(Equation

WECAN solves Equation 1 using the non-linear modal superposition theory. An initial computer run is made to calculate the eigenvalues (frequencies) and eigenvectors (mode shapes) for the mathematical model. This information is stored, and is used in a subsequent computer run which solves Equation 1. The first time step performs a static solution of Equation 1 to determine the initial displacements of the structure due to deadweight and normal operating hydraulic forces. After the initial time step, WECAN calculates the dynamic solution of Equation 1. Time history nodal displacements and impact forces are stored for post-processing.

The following typical discrete elements from the WECAN finite element library are used to represent the reactor vessel and internals components:

- Three-dimensional elastic pipe
- Three-dimensional mass with rotary inertia
- Three-dimensional beam
- Three-dimensional linear spring
- Concentric impact element
- Linear impact element
- 6x6 stiffness matrix
- 18 Card stiffness matrix
- 18 Card mass matrix
- Three-dimensional friction element

3.9.2.3.4.2 Analytical Methods

The RPV system finite element model, as described above, was used to perform the LOCA analysis. Following a postulated LOCA pipe rupture, forces are imposed on the reactor vessel and its internals. These forces result from the release of the pressurized primary system coolant. The release of pressurized coolant results in traveling depressurization waves in the primary system. These depressurization waves are characterized by a wavefront with low pressure on one side and high pressure on the other. The wavefront translates and reflects throughout the primary system until the system is completely depressurized. The rapid depressurization results in transient hydraulic loads on the mechanical equipment of the system.

The LOCA loads applied to the reactor pressure vessel system consist of (a) reactor internal hydraulic loads (vertical and horizontal), and (b) reactor coolant loop mechanical loads. All the loads are calculated individually and combined in a time-history manner.

3.9.2.3.4.3 RPV Internal Hydraulic Loads

Depressurization waves propagate from the postulated break location into the reactor vessel through either a hot leg or a cold leg nozzle.

After a postulated break in the cold leg, the depressurization path for waves entering the reactor vessel is through the nozzle into the region between the core barrel and reactor vessel. This region is called the down-comer annulus. The initial waves propagate up, around, and down the down-comer annulus, then up through the region circumferentially enclosed by the core barrel; that is, the fuel region.

The region of the down-comer annulus close to the break depressurizes rapidly but, because of the restricted flow areas and finite wave speed (approximately 3,000 feet per second), the opposite side of the core barrel remains at a high pressure. This results in a net horizontal force on the core barrel and reactor pressure vessel. As the depressurization wave propagates around the downcomer annulus and up through the core, the barrel differential pressure reduces, and similarly, the resulting hydraulic forces drop.

In the case of a postulated break in the hot leg, the waves follow a dissimilar depressurization path, passing through the outlet nozzle and directly into the upper internals region, depressurizing the core and entering the down-comer annulus from the bottom exit of the core barrel. Thus, after a break in the hot leg, the down-comer annulus would be depressurized with very little difference in pressure across the outside diameter of the core barrel.

A hot leg break produces less horizontal force because the depressurization wave travels directly to the inside of the core barrel (so that the down-comer annulus is not directly involved), and internal differential pressures are not as large as for a cold leg break. Since the differential pressure is less for a hot leg break, the horizontal force applied to the core barrel is less for a hot leg break than for a cold leg break. For breaks in both the hot leg and cold leg, the depressurization waves would continue to propagate by reflection and translation through the reactor vessel and loops.

The MULTIFLEX computer code described by Takeuchi calculates the hydraulic transients within the entire primary coolant system. It considers subcooled, transition, and two-phase (saturated) blowdown regimes. The MULTIFLEX program employs the method of characteristics to solve the conservation laws, and assumes one-dimensionality of flow and homogeneity of the liquid-vapor mixture.

The MULTIFLEX code considers a coupled fluid-structure interaction by accounting for the deflection of constraining boundaries, which are represented by separate spring-mass oscillator systems. A beam model of the core support barrel has been developed from the structural properties of the core barrel; in this model, the cylindrical barrel is vertically divided into various segments and the pressure, as well as the wall motions, is projected onto the plane parallel to the broken inlet nozzle. Horizontally, the barrel is divided into 10 segments; each segment consists of 3 separate walls. The spatial pressure variation at each time step is transformed into 10 horizontal forces, which act on the 10 mass points of the beam model. Each flexible wall is bounded on either side by a hydraulic flow path. The motion of the flexible walls is determined by solving the global equations of motion for the masses representing the forced vibration of an undamped beam.

3.9.2.3.4.4 Reactor Coolant Loop Mechanical Loads

The reactor coolant loop mechanical loads are applied to the RPV nozzles by the primary coolant loop piping. The loop mechanical loads result from the release of normal operating forces present in the pipe prior to the separation as well as transient hydraulic forces in the reactor coolant system. The magnitudes of the loop release forces are determined by performing a reactor coolant loop analysis for normal operating loads (pressure, thermal, and deadweight). The loads existing in the pipe at the postulated break location are calculated and are "released" at the initiation of the LOCA transient by application of the loads to the broken piping ends. These forces are applied with a ramp time of 1 msec because of the assumed instantaneous break opening time. For breaks in the branch lines, the force applied at the reactor vessel would be insignificant. The restraints on the main coolant piping would eliminate any force to the reactor vessel caused by a break in the branch line.

3.9.2.3.4.5 Results of the Analysis

The severity of a postulated break in a reactor vessel is related to three factors: the distance from the reactor vessel to the break location, the break opening area, and the break opening time. The nature of the decompression following a LOCA, as controlled by the internals structural configuration previously discussed, results in larger reactor internal hydraulic forces for pipe breaks in the cold leg than in the hot leg (for breaks of similar area and distance from the RPV). Pipe breaks farther away from the reactor vessel are less severe because the pressure wave attenuates as it propagates toward the reactor vessel. The LOCA hydraulic and mechanical loads described in the previous sections were applied to the WECAN model of the reactor pressure vessel system.

The results of LOCA analysis include time history displacements and non-linear impact forces for all major components. The time history displacements of upper core plate, lower core plate and core barrel at the upper core plate elevation are provided as input for the reactor core evaluations. The impact forces calculated at the vessel-internals interfaces are used to evaluate the structural integrity of the reactor vessel and its internals. Using appropriate post-processors, component linear forces are also calculated.

3.9.2.3.5 Transverse Guide Tube Excitation by Blowdown Forces

3.9.2.3.5.1 General

Since the dynamic loads on the guide tubes are more severe for a loss-of-coolant accident caused by a hot-leg rupture than for a cold-leg rupture, only the hot-leg blowdown accident was analyzed. The guide tubes closest to the ruptured outlet leg are subject to the greatest blowdown forces, with the forces decreasing on guide tubes located at greater distances from the ruptured nozzle.

From a hydraulic analysis of the fluid forces acting on the guide tubes nearest the outlet nozzles during MODES 1 and 2, the net force due to a linearly distributed drag force was found to be $F = 1/2\ C_D A V^2 = 357$ lb. The outlet flow velocity during MODES 1 and 2 was $V_{normal} = 48$ fps.

As a result of the 1 msec hot-leg rupture, the outlet mass flux ($m = \gamma V$) was found to increase from 2060 lb/ft^2-sec for MODES 1 and 2 to 8060 lb/ft^2-sec.

The drag force on the guide tube nearest the ruptured nozzle was found by a ratio of the blowdown outlet velocity

$V_{BLOWDOWN} = 8060 / 42.7 = 188.8$ fps

to the normal outlet velocity of 48 fps when squaring this ratio to determine the blowdown force

$F_{BLOWDOWN} = (188.8/48)^2 \times 357 = 5523 \text{ lb} = W$

3.9.2.3.5.2 Response of Guide Tube

A detailed structural analysis of the guide tubes was performed in order to establish the equivalent cross-section properties and elastic end support conditions. The model was verified by an experimental test using a concentrated force applied at the transition plate. The experimental results also produced a load deflection curve into the plastic range for the guide tubes as well as determining deflection criteria to ensure rod cluster control insertion.

The analytical model was used to establish a correlation between the net hydraulic loading for the linearly distributed drag force and a concentrated force applied at the transition plate requiring the deflection of the transition plate to be the same for both loadings. It was found

$F_c = 0.59W = 3259$ lb

The natural frequency of the guide tube was determined experimentally to be 43 Hz which corresponds to a period of T = 23.3 msec. While the hydraulic drag forces on the guide tube were applied over a finite time interval, it was conservatively assumed that the dynamic amplification factor is 2.0 resulting from an impulse loading in the form of a step function. The value of 2.0 was conservative also by virtue of the fact that if yielding occurred the amplification factor was less than 2.0 which is valid for elastic deflections. Thus the maximum dynamic equivalent concentrated force was

$F_{Max} = 2.0\ (3259) = 6520$ lb

From the experimental load deflection curve, the maximum permanent guide tube deflection was calculated to be 0.31 in., which corresponds to a maximum deflection of 0.75 in. during the transient.

<u>Conclusions</u>

From the experimental study of rod cluster control insertion as a function of guide tube deflection it was concluded that, under the most severe postulated blowdown accident, rod cluster control insertion was ensured and there would be no loss of function of the rod cluster control guide tubes.

3.9.2.3.5.3 Description of Stress Location

The stress values given in Tables 3.9-15 and 3.9-16 are based upon the maximum force experienced during the blowdown excitation. The maximum stresses for various components in general do not occur simultaneously. A description of the location of the various stresses are as follows:

a. Upper core plate - Bending stresses caused by local deformation of upper core plate between upper support columns.

b. Upper support column - Direct stress in columns due to axial load. Stress calculated for minimum cross-sectional area.

c. Fuel assembly top nozzle - Bending stress in the ligaments of the adaptor plate maximum stress occurs in the section adjacent to the side plate of the top nozzle.

d. Barrel flange - The maximum stress occurs at the transition region between the barrel flange and the upper core barrel. The stresses are both axial and bending.

e. Lower support structure - Maximum bending stress at the center hole. Radius equal 8 in.

f. Core barrel - Axial (direct) stresses located in the reduced cross-sectional area between upper and lower core barrel.

g. Lower core plate - Bending stresses caused by local deformation of lower core plate between shroud tubes.

h. Fuel assembly bottom nozzle - Maximum bending stress occurs in the bars of the bottom nozzle in the section adjacent to the side plates.

3.9.2.3.6 Reevaluation of the Dynamic Response of Reactor Internals for Extended Power Uprate (EPU)

The reactor vessel internals are designed to withstand forces due to structure deadweight, preload of fuel assemblies, control rod assembly dynamic loads, vibratory loads and earthquake accelerations. Changes in the reactor coolant system (RCS) operating conditions as a result of Extended Power Uprate (EPU) result in changes to the boundary conditions (loads and temperatures) experienced by the reactor vessel internals. Therefore, a systematic evaluation of the impact of these changes on the short and long term performance of these components was performed (*Reference 31*). This analysis included eight specific tasks described below.

3.9.2.3.6.1 Reactor Pressure Vessel System Thermal-Hydraulic Analysis

Due to the change in primary side conditions, a reactor pressure vessel system thermal hydraulic analysis was performed. The hydraulic forces were used in the assessment of the structural integrity of the reactor internals, core clamping loads generated by the internals hold down spring, and the stresses in the reactor vessel closure studs.

3.9.2.3.6.2 Bypass Flow Analysis

Bypass flow is the total amount of reactor coolant flow bypassing the core region. The driving force for the bypass flow paths is dependent upon the magnitude of the pressure drop in the reactor core. Since variations in the size of some of the bypass flow paths, such as

outlet nozzles and the core cavity region, occur during manufacture, plant specific as-built dimensions were used in order to demonstrate that the bypass flow limits are not violated. Therefore, an analysis was performed to determine actual, best estimate core bypass flow to ensure that the design bypass flow limit for the plant is not exceeded.

3.9.2.3.6.3 Thermal Analysis of the Baffle/Barrel Region

A baffle-barrel region temperature analysis was used to determine the temperature distribution in the baffle plates and in the core barrel. This data was used to evaluate the loadings on the baffle-former bolts, barrel-former bolts and the baffle to baffle edge bolts.

Changes in design transients and in the internal heat generation rates due to gamma heating will affect the relative growth of the barrel and baffle and resulting bolt loads, former plate temperatures, and the skin and bending stresses of all components for which gamma heating is significant. An evaluation was performed to provide thermal data for the structural evaluations of all components that are affected by the changes in the RCS conditions due to the Extended Power Uprate (EPU).

3.9.2.3.6.4 Pressure Drop Across the Baffle Plate Analyses

The hydraulic analysis determines the axial variation in pressure difference across the baffle plates and therefore provides the baffle plate and baffle-barrel region threaded fastener (bolts) pressure loading. This analysis addresses the effects of uncertainties in the relevant hydraulic loss coefficients for the fuel and for the reactor internals. Finally, this information was used as input to the evaluation of the momentum flux of the baffle jets.

3.9.2.3.6.5 Flow Induced Vibration

An assessment of the impact of the new RCS conditions due to Extended Power Uprate (EPU) on flow induced vibration on the reactor internals was performed. This work showed that the vibrational amplitudes of the reactor internals due to the new primary side conditions remain small and have no adverse affect on component structural integrity.

3.9.2.3.6.6 Reactor Internals Structural Integrity

Structural analyses and evaluations were performed to demonstrate that the short and long term structural integrity of the various components of the reactor internals were not adversely impacted by the change in operating conditions. These evaluations addressed changes in hydraulic lift forces as well as changes in component temperature distribution during steady state and transient conditions. In addition, both stress limits and fatigue criteria were addressed.

3.9.2.3.6.7 Control Rod Performance

The effect of the changes in the primary side conditions on the control rod drop times was evaluated.

3.9.2.3.6.8 Vessel/Internals/Fuel/Control Rod Response During Loca Conditions

Detailed time-history analyses were performed to recalculate system interface loads and fuel assembly grid impact loads. Since leak-before-break has been applied to the RCS (*Reference 32*), the limiting breaks considered were an accumulator line break and a pressurizer surge line break. A plant specific dynamic analysis model of the reactor vessel/internals/vessel supports/fuel system was developed using the WECAN code. The reactor pressure vessel model includes the effects of gaps between the reactor internals, fuel and reactor vessel and the non-linear modal superposition method of solution to minimize computing costs. This model was used to develop structural input (beam data) for the Multiflex code. The resulting hydraulic forces were used as input to the time history LOCA structural analysis. Once the time history analyses were performed, stress analysis was performed to determine if stresses and deflections in the Core Support Structures are within the allowable limits for the faulted condition.

3.9.2.3.6.9 Summary of Conclusions

Evaluations have been performed to assess the effect of the Extended Power Uprate (EPU) RCS conditions on the reactor pressure vessel/internals system at Ginna Station. These evaluations used the revised transients along with the consideration of leak-before-break postulated conditions.

The major conclusions reached based on the work described in this report are:

1. The vessel pressure drops, bypass flows and hydraulic lift forces are not significantly affected by the new RCS conditions due to proposed Extended Power Uprate (EPU) program.
2. The design core bypass flow value for Ginna Station is unchanged.
3. Acceptable control rod drop times will be achieved. The current Technical Specification limit of 1.8 seconds remains acceptable.
4. The structural integrity of the reactor internals is maintained with the new RCS conditions.

3.9.2.4 Asymmetric Loss-of-Coolant Accident Loading Analysis

The capability of the reactor vessel internal structures to maintain their functional integrity in the event of a major loss-of-coolant accident was evaluated during the resolution of the Unresolved Safety Issue A-2, Asymmetric Loading. Analysis performed for limited size breaks reported in WCAP 9748 (*Reference 18*), showed that the appropriate systems and components will maintain their functional capability to ensure a safe plant shutdown with a coolable core geometry. The systems and components examined were the reactor vessel assembly including internals, fuel, control rod drive mechanisms, vessel and component supports, reactor coolant loop piping, and attached emergency core cooling piping.

3.9.2.5 Seismic Evaluation of Reactor Vessel Internals

3.9.2.5.1 Analysis Procedure

These structures were analyzed assuming that the operating basis earthquake and the safe shutdown earthquake (0.20g) have equal horizontal and vertical components. Dynamic

methods of analysis were used according to the following, with the core and the reactor internals being analyzed as part of a complex reactor structure because of the interconnection of their masses and stiffness.

The general procedure for the dynamic analysis can be summarized as follows:

A. The reactor structure from the ground to the core was reduced to a continuous structural network consisting of elements with variable stiffness, mass distribution, and cross section; concentrated masses, intermediate supports, and local releases (i.e., connections, as between fuel assemblies and core plates that are assumed to be hinges).

B. The canless fuel assembly mechanical design used in the core is composed of fuel rods arranged in a square array, with spring-clip grids locating and holding the fuel rods in the precise array required. Effective stiffness and natural frequency values were determined to establish the response of a fuel assembly to a dynamic excitation. An important characteristic of these structures is that they present a very high internal damping produced by the slippage of the rods on the finger grids. The fact that their own frequency is relatively low with respect to the supporting structure ensured that a resonance phenomenon with the support will not occur. This condition was confirmed by the dynamic analysis.

C. The lower natural transverse frequencies and normal modes were obtained for this complex structure taking into account shear deformations and using numerical methods.

D. The maximum response of the structure under horizontal earthquake excitation was obtained from the superposition of the normal modes responses (with the conservative assumption that all the modes were in phase and that all the peaks occur simultaneously) and using response curves normalized for 0.08g and 0.20g maximum ground accelerations using 1% damping.

E. After obtaining the maximum possible response under earthquake excitation, the stress values at the critical structure points were computed.

F. For the vertical earthquakes the same general method was employed but using an equivalent one degree of freedom system.

3.9.2.5.2 Analysis Results

Stresses and deflections of reactor internals and core were determined using the method explained above. The vertical and horizontal components of the ground accelerations were considered separately. The stress distribution for each case was calculated after obtaining the maximum response of the structure. These stresses were then combined with stresses of other origin (pressure stresses, thermal stresses, etc.) to obtain maximum stresses which must be within the limits given by the allowable stress criteria. The maximum stresses were, therefore, conservatively determined on whichever combination of simultaneous conditions yield the highest stress condition.

The maximum deflections under seismic accelerations were computed and combined with deflections from other loadings. These deflections were sufficiently small to permit normal operation and do not necessarily coincide in time with maximum stresses.

Stresses of earthquake origin were considered as primary stresses. For the reactor internals the primary membrane stresses induced by earthquake loadings (0.08g and 0.20g maximum ground accelerations) combined with induced primary membrane stresses from other loading conditions, as described above, remained within the design stress intensity values established by the ASME Boiler and Pressure Vessel Code, Section III. Primary bending and secondary stresses which included thermal stresses were also limited following the criteria and methods prescribed by the ASME Code, Section III.

For the fuel assemblies, stress levels are such that the fuel assembly functional integrity is maintained under the action of the imposed loads including seismic effects.

Tables 3.9-17 through 3.9-19 summarize the primary principal stress results at various elevations in the reactor. Table 3.9-20 presents the maximum primary stress intensities. These values are seen to be considerably below the allowable value of 24,000 psi. Table 3.9-21 summarizes the primary plus secondary principal stress results at various elevations in the reactor. Table 3.9-22 presents the maximum primary plus secondary stress intensities. These values are seen to be considerably below the allowable value of 48,000 psi.

3.9.3 COMPONENT SUPPORTS AND CORE SUPPORT STRUCTURES

3.9.3.1 Loading Combinations, Design Transients, and Stress Limits

The loadings and design transients used are the same as those used for the piping, equipment, and component analyses given in Section 3.9.1. The bases for the original design of Ginna Station are as follows:

> All piping, components, and supporting structures of the reactor coolant system were designed as Seismic Category I equipment, i.e., they are capable of withstanding:
>
> 1. Within code allowable, working stresses for the design seismic ground acceleration.
> 2. The maximum potential seismic ground acceleration acting in the horizontal and vertical direction simultaneously with no loss function.

The loadings, load combinations, and stress limits used in the original design and during the Systematic Evaluation Program (SEP) reevaluation are given in Table 3.9-1 and Table 3.9-11, respectively.

3.9.3.2 Component Supports

The reactor coolant system components and supports were designed as Seismic Category I.

3.9.3.2.1 Reactor Vessel

The vessel is supported on six individual pedestals. Each pedestal rests upon plates which are in turn supported upon the circular concrete primary shield wall.

The reactor vessel has six supports comprising four support pads located one on the bottom of each of the primary nozzles and two gusset support pads. One of the reactor inlet nozzles is centered approximately 2 degrees counterclockwise from the 90-degree axis and the other is centered approximately 2 degrees counterclockwise from the 270-degree axis.

Each support bears on a support shoe, which is fastened to the support structure. The support shoe is a structural member that transmits the support loads to the supporting structure. The support shoe is designed to restrain vertical, lateral, and rotational movement of the reactor vessel, but allows for thermal growth by permitting radial sliding at each support, on bearing plates.

3.9.3.2.2 Steam Generators

Each steam generator is supported on a structural system consisting of four vertical support columns and two (upper and lower) support systems. The vertical columns, which are pin-connected to the steam generator support feet, serve as vertical restraint for operating weights, pipe rupture, and seismic considerations while permitting movement in the horizontal plane. The support systems, by using a combination of stops, guides, and snubbers, prevent rotation and excessive movement of the steam generator in any horizontal plane.

The lower support system consists of an arrangement of structural steel shapes in combination with steel plates that are in a horizontal plane. The system is designed to restrain excessive horizontal movement of the steam generator and also to accommodate thermal growth. The upper support system consists of three sets of rigid struts and one set of hydraulic snubbers (see Figure 3.9-6a). The snubbers function under tension or compression loads while the struts are compression only elements. The struts were installed so that there are minimal gaps between the strut and the corresponding support element on the steam generator. The steam generator support structures were originally designed for loads resulting from ruptures of the main steam piping and primary coolant piping. These loads exceeded the seismic loads. The upper support rings were constrained by eight hydraulic snubbers, a pair in each of the four lateral directions.

Generic Letter 87-11 eliminated the requirement to consider the dynamic effects of arbitrary intermediate pipe ruptures and removed the postulated main steam line rupture in the first horizontal run of main steam line as the controlling design load for the steam generator upper lateral support system. RG&E applied the leak-before-break theory to remove the primary coolant line rupture as the next highest design load for the support system. The removal of these two controlling loads permitted the replacement of six of the hydraulic snubbers for each steam generator with the rigid bumpers in the upper support system. The new support system was evaluated for the load combinations and allowable stress limits defined in Table 3.9-23.

3.9.3.2.3 Reactor Coolant Pumps

Each reactor coolant pump is supported by a structural system consisting of three vertical columns and a system of stops. The vertical columns are bolted to the pump support feet and permit movement in the horizontal plane to accommodate reactor coolant pipe expansion. Horizontal restraint is accomplished by a combination of tie rods and stops which limit horizontal movement for pipe rupture and seismic effects.

Support structures of the steam generators and reactor coolant pump components were designed for loads resulting from ruptures of the primary coolant piping and main steam piping. Equivalent static seismic forces equal to the component weight, accelerated by the

peal response of the applicable seismic response spectra, applied through the component center of gravity, were evaluated against the corresponding pipe rupture loads. For both the steam generators and reactor coolant pumps, the resulting seismic forces were smaller than the pipe rupture loads; therefore, supports were designed for pipe rupture loads.

3.9.3.2.4 Pressurizer

The pressurizer is supported on a heavy concrete slab spanning between the concrete shield walls for the steam generator compartment. The pressurizer is a bottom skirt supported vessel.

3.9.3.2.5 Reactor Coolant Piping

The reactor coolant piping layout is designed on the basis of providing floating supports for the steam generator and reactor coolant pump in order to permit the thermal expansion from the fixed or anchored reactor vessel. A comprehensive thermal analysis was performed to ensure that stresses induced by linear thermal expansion are within code limits.

3.9.3.3 Pipe Supports

3.9.3.3.1 Original Analysis

The pipe stress analysis performed during the original design of Ginna Station also gave the pipe support reactions. The results of the analysis indicated that the margin between the ultimate support capacity and the support reactions for 0.2g ground acceleration was sufficient to handle building amplification.

For the Seismic Category I piping 2 in. nominal size and under, the support reactions were well below the capacity of the supports (*Reference 4*). For pipes falling in this category, the minimum hanger rod diameter was found to be 1/2 in. for outdoor installations and 3/8 in. for indoor installations. The 3/8-in. rods had an ultimate capacity of the order of 3700 lb. The horizontal supports had an ultimate capacity, in shear, of the order of 1100 lb. For the heaviest pipe in this category, the support reactions were of the order of 100 lb, i.e., well below the ultimate capacity of the supports.

A few pipe runs had lateral support spacing two to three times that suggested by USAS B31.1 for vertical supports. The support reactions for the heaviest pipe of this category were of the order of 200 lb and well within the ultimate capacity of the supports.

3.9.3.3.2 IE Bulletin Reanalysis

Subsequent to the original design of the Ginna Station piping, several dynamic analyses of the piping system were performed that included the later developed loading requirements and regulatory changes. The analyses performed for the residual heat removal loop, the main steam line loop, safety injection system piping, and charging line in response to IE Bulletin 79-07 are described in Section 3.9.2.1. The pipe support reactions calculated from these analyses using as-built conditions and the design loads for the residual heat removal loop, main steam line loop, and charging line are given in Table 3.9-24 through Table 3.9-26. Results indicate the adequacy of these pipe supports.

3.9.3.3.3 Seismic Piping Upgrade Program

3.9.3.3.3.1 Applicable Supports

Supports for Seismic Category I piping systems listed in Section 3.7.3.7.1 were included in the Seismic Piping Upgrade Program.

3.9.3.3.3.2 Load Combinations and Stress Limits

The piping system supports were evaluated for the following piping system imposed loads and support inertial effects:

a. Normal condition: deadweight and maximum operating thermal.

b. Design condition: deadweight, maximum operating thermal, and operating-basis earthquake.

c. Safe shutdown earthquake condition: deadweight, normal operating thermal, and safe shutdown earthquake.

The loading combinations and associated stress limits are given in Table 3.9-27. The allowable stress criteria were in accordance with Subsection NF of the ASME Section III Code, 1974. Faulted condition stress allowables from Appendix F of the ASME Section III Code and Regulatory Guide 1.124 were used to analyze the supports for the safe shutdown earthquake condition. The variance in allowable criteria between the piping and supports will not cause over-or under-designs to occur, as the satisfaction of the operating-basis earthquake condition to the working stress limits will in all cases be most stringent. The component support embedments were evaluated using current analytical techniques in accordance with the anchor bolt manufacturer’s Technical Information and ACI-349, Appendix B. The expansion anchorages must meet the requirements set forth in IE Bulletin 79-02.

3.9.3.3.3.3 Structural Requirements

For anchors that separate SeismicCategory I piping systems from nonseismic piping, the loads from the Seismic Category I side were doubled. The effects of friction on supports was considered for pipes having thermal movements greater than 0.1 in. The value of was 0.35 and was used conservatively to increase support loads but not reduce loads.

The stiffness of the supports was considered in the piping system models. The local subsystem stiffness of all piping and equipment supports was determined considering the pipe or equipment supports along with the structural steel and/or concrete effect. The localized subsystem stiffness of all piping and equipment supported by reinforced-concrete members (including concrete pedestals) was considered when significant. The stiffness was based on the face of concrete interface.

Rigid supports were modeled in accordance with the following criteria:

Nominal Pipe Size (in.)	K_{min} Rigid (lb/in.)	K_{min} Rigid (in.-lb/rad)
≤2	1 x 10^5	1 x 10^7
2-1/2 to 4	5 x 10^5	5 x 10^7
≥6	1 x 10^6	1 x 10^8

Use of the above guidelines eliminates excessive support stiffness calculation effort, while yielding satisfactory support displacement results (i.e., thermal deflections <0.02 in., rotations <0.0002 radians).

"Common pipe supports" refer to those supports to which two or more pipes are attached in such a way that significant coupling occurs between the pipes. When all attached pipes are the same size and the distances to adjacent supports are similar, the local subsystem stiffness is based on the deflections resulting from an equal load acting at all support points. When different size pipes are attached, or if the distances to adjacent supports are not similar, a stiffness matrix relating the forces and displacements at the points of attachments to one another was provided to the piping analyst for use in uncoupling the piping systems.

Hydraulic seismic supports (snubbers) generally lock up at an excitation frequency of approximately 1 Hz, with a piping displacement of 0.05 in. Mechanical snubbers activate in a frequency range of 1 to 6 Hz with a similar piping displacement of 0.05 in. As piping system frequencies seldom exist below this range, seismic supports were modeled as active during all seismic events.

Supports were considered active statically in any given direction provided the support gap in that direction does not exceed 0.125 in. This 0.125 in. tolerance is essentially construction variance, which does not alter the designed function of the support. Supports with gaps greater than 0.125 in. were incorporated as follows. System analysis first assumed that the support was not active; piping displacements resulting from this run were then used to ascertain the validity of this assumption. If incorrect, reanalysis incorporated an active support statically.

The inertial effects of the supports own mass was considered. The additional inertial loads were determined based on a review of the support flexibility, support mass, and applicable response spectra.

All supports were analyzed and modified if necessary to be in compliance with IE Bulletin 79-02 criteria. Any existing support with anchor bolts subject to tension loads and which were previously only subject to compression or shear loads were inspected or tested to confirm installation adequacy.

The effects of new support loads generated by the piping reanalysis upon the existing structures were evaluated.

Piping supports were modeled as described in Section 3.7.3.7.10.

3.9.3.3.4 Base Plate Flexibility

In general, calculation of anchor bolt loads for pipe supports at Ginna Station assumed rigid base plates. This included both the shell type concrete expansion anchor bolts used in the original plant design and the wedge type which were generally used for plant modifications.

In order to assess the significance of rigid versus flexible plate assumptions, a representative sample of typical pipe support base plates were reanalyzed. The reanalysis was performed assuming both the base plate and bolts as elastic and using separate procedures for moment and axial loadings.

It was not possible to reanalyze, using flexible plate assumptions, the base plates on all pipe supports in the testing and replacement program prior to initiation. Therefore, a representative sample of 10 typical pipe support base plates has been analyzed, using rigid plate assumptions, for both existing and replacement designs. The results of these analyses are shown in Table 3.9-28. In all cases, bolt capacity has been increased in the replacement designs. In two cases, additional analyses, using flexible plate assumptions, were performed. These analyses showed minimum factors of safety of 5.00 and 5.35, respectively, for the replacement designs. The design factor of safety for the wedge type anchor bolts used in the replacement designs was 4.00. Therefore, it was determined that the design bolt capacities provide sufficient margins of safety to account for any load increases due to flexibility.

In general, pipe supports at Ginna Station with base plates using concrete expansion anchor bolts are of similar design. They are typical of the type used in Seismic Category I systems throughout the plant.

The capacity of concrete expansion anchor bolts to withstand cyclic loads (seismic as well as high cyclic operating loads) were evaluated in fast flux test facility tests. The test results indicated that

A. The expansion anchors successfully withstood two million cycles of long-term fatigue loading at a maximum intensity of 0.2 of the static ultimate capacity. When the maximum load intensity was steadily increased beyond that value and cycled for 2000 times at each load step, the observed failure load was about the same as the static ultimate capacity.

B. The dynamic load capacities of the expansion anchors under simulated seismic loading were about the same as the corresponding static ultimate capacities.

Based on the above data, it could be concluded that the design requirements for preloaded concrete expansion anchor bolts under cyclic loads are the same as for the static loads.

3.9.3.3.5 Snubbers

3.9.3.3.5.1 Design Loads

The mechanical and hydraulic suppressors (snubbers) installed on Seismic Category I piping systems and the steam generators at Ginna Station were designed to restrain seismic loads. Hydraulic snubbers installed on pressurizer safety valve discharge piping were designed to

restrain hydraulic loads resulting from safety valve discharges. The loads which the snubbers had to meet were calculated by seismic or thermal hydraulic analysis, as appropriate. Standard available snubbers were purchased with rated loads greater than or equal to the calculated loads. A review of the various snubbers installed on these systems and components showed that they were capable of functioning with loads at least 1.33 times their rated loads and were structurally designed for loads at least 2.0 times their rated loads.

The hydraulic snubbers were designed to operate with an internal fluid pressure of 3000 psi and to limit fluid pressure to 4000 psi by means of a spring-loaded relief valve(*Reference 4*). When the compressive load exceeded 14.7 kips and 28 kips for the 11 kips and 21 kips snubbers, respectively, the spring-loaded relief valves opened. If this load was sustained, the snubber would eventually get solid. The mechanical ultimate capability was about four times the design capacity, i.e., 84 kips and 44 kips for 21 kips and 11 kips snubbers, respectively.

Therefore, the seismic loads associated with 0.2g ground acceleration were found not to cause mechanical failure of these snubbers. The only potential effect could be some movement of the snubber rod because of temporary loss of fluid. However, because of the dynamic nature of the seismic loads and the inherent flexibility of the supported pipes, the potential limited snubber movement would not induce stresses in the feedwater and steam lines above tolerable limits.

A review was made of the capability of the various snubbers to lock up upon application of their design loads. Since the basic seismic analysis method utilized at the time Ginna Station was designed was a static, lumped mass approach, specific dynamic requirements were not established by the seismic analysis. However, a conservative analysis of the minimum velocities that could be experienced during a seismic event, based on a frequency of 33 Hz and a ground acceleration of 0.08g, gives a result of approximately 60 in./minute. Hydraulic snubbers installed at Ginna Station are capable of locking up with velocities no greater than 10 in./ minute.

3.9.3.3.5.2 Surveillance Program

A surveillance program for snubbers has been instituted at Ginna Station. The current requirements for inspection and functional testing of snubbers are included in Interface Procedure IP-IIT-5, Snubber Inspection and Testing Program.

3.9.4 CONTROL ROD DRIVE SYSTEMS

3.9.4.1 Description

<u>3.9.4.1.1 General</u>

The control rod drive mechanisms are used for withdrawal and insertion of the control rods into the reactor core and to provide sufficient holding power for stationary support. Fast total insertion (reactor trip) is obtained by simply removing the electrical power allowing the rods to fall by gravity.

The complete drive mechanism, shown in Figures 3.9-7 and 3.9-8, consists of the internal (latch) assembly, the pressure vessel, the operating coil stack, the drive shaft assembly, and the position indicator coil stack.

Each assembly is an independent unit which can be dismantled or assembled separately. Each drive is threaded into an adaptor on top of the reactor pressure vessel and is connected to the control rod (directly below) by means of a grooved drive shaft. The upper section of the drive shaft is suspended from the working components of the drive mechanism. The drive shaft and control rod remain connected during reactor operation, including tripping of the rods.

Main coolant fills the pressure containing parts of the drive mechanism. All working components and the shaft are immersed in the main coolant.

Three magnetic coils, which form a removable electrical unit and surround the rod drive pressure housing, induce magnetic flux through the housing wall to operate the working components. They move two sets of latches which lift or lower the grooved drive shaft.

The three operating coils are sequenced by solid-state switches for the control rod drive assemblies. The sequencing of the magnets produces step motion over the 144 in. of normal control rod travel.

The mechanism develops a lifting force approximately two times the static lifting load. Therefore, extra lift capacity is available for overcoming mechanical friction between the moving and the stationary parts. Gravity provides the drive force for rod insertion and the weight of the whole rod assembly is available to overcome any resistance.

A multiconductor cable connects the mechanism operating coils to the 125-V dc power supply. The power supply includes the necessary switchgear to provide power to each coil in the proper sequence.

In 1996, the NRC issued NRC Bulletin 96-01 (*Reference 26*) to alert licensees to problems encountered during events in which control rods failed to completely insert upon the scram signal and to have licensees assess control rod operability at their facilities. RG&E's response to IEB 96-01 (*References 27 through 30*) addressed training performed in relation to the issues, operability determinations made, justification for not performing rod drop testing and gathering recoil data at the end of Cycle 25, and future plans, and transmitted core map information and control rod drag testing results. In addition, RG&E stated that based on a review of the rod drag testing data, both Westinghouse and RG&E concluded that there was no concern for rod cluster control assembly insertion anomalies at burnups tested for Ginna.

3.9.4.1.2 Latch Assembly

The latch assembly contains the working components which withdraw and insert the drive shaft and attached control rod. It is located within the pressure housing and consists of the pole pieces for three electromagnets. They actuate two sets of latches which engage the grooved section of the drive shaft.

The upper set of latches move up or down to raise of lower the drive rod by 5/8 in. The lower set of latches have 1/32-in. axial movement to shift the weight of the control rod from the upper to the lower latches.

3.9.4.1.3 Pressure Vessel

The pressure vessel consists of the pressure housing and rod travel housing. The pressure housing is the lower portion of the vessel and contains the latch assembly. The rod travel housing is the upper portion of the vessel. It provides space for the drive shaft during its upward movement as the control rod is withdrawn from the core.

3.9.4.1.4 Operating Coil Stack

The operating coil stack is an independent unit which is installed on the drive mechanism by sliding it over the outside of the pressure housing. It rests on a pressure housing flange without any mechanical attachment and is removed and installed while the reactor is pressurized.

The operating coils (A, B, and C) are made of round copper wire which is insulated with a double layer of filament-type glass yarn.

3.9.4.1.5 Drive Shaft Assembly

The main function of the drive shaft is to connect the control rod to the mechanism latches. Grooves for engagement and lifting by the latches are located throughout the 144 in. of control rod travel. The grooves are spaced 5/8 in. apart to coincide with the mechanism step length and have 45 degree angle sides.

The drive shaft is attached to the control rod by the coupling. The coupling has two flexible arms which engage the grooves in the spider assembly. A 1/4-in. diameter disconnect rod runs down the inside of the drive shaft. It utilizes a locking button at its lower end to lock the coupling and control rod.

During plant operation, the drive shaft assembly remains connected to the control rod at all times. It can be attached and removed from the control rod only when the reactor vessel head is removed.

3.9.4.1.6 Position Indicator Coil Stack

The position indicator coil stack slides over the rod travel housing section of the pressure vessel. It detects drive rod position by means of discrete cylindrically wound coils that are spaced at 7.5 in. (12 step) intervals along the rod travel (144 in.).

3.9.4.2 Design Loads, Stress Limits, and Allowable Deformation

The mechanisms are designed to operate in water at 650°F and 2485 psig. The temperature at the mechanism head adaptor will be much less than 650°F because it is located in a region where there is limited flow of water from the reactor core, while the pressure is the same as in the reactor pressure vessel.

The design operating temperature of the coils is 232°C. Coil temperature can be determined by resistance measurement. Forced air cooling along the outside of the coil stack maintains a coil temperature of approximately 200°C.

3.9.4.3 Control Rod Drive Mechanism Housing Mechanical Failure Evaluation

An evaluation of the possibility of damage to adjacent control rod drive mechanism housings in the event of a circumferential or longitudinal failure of a rod housing located on the vessel head is presented.

3.9.4.3.1 Housing Description

The control rod drive mechanism schematic is shown in Figure 3.9-8. The operating coil stack assembly of this mechanism has a 10.8 in. by 10.8 in. cross section and a 39.55 in. length. The position indicator coil stack assembly (not shown in the figure) is located above the operating coil stack assembly. It surrounds the rod travel housing over nearly its entire length.

The rod travel housing outside diameter is 3.8 in. and the position indicator coil stack assembly inside and outside diameters are approximately 4 in. and 7 in., respectively. This assembly consists of a 1/8-in. thick stainless steel tube on which are mounted 20 coils. The coils are mounted at 12 step (7.5 inch) intervals along the tube. This assembly is held together by two end plates (the top end plate is square), an outer sleeve, and four axial tie rods.

3.9.4.3.2 Effects of Rod Travel Housing Longitudinal Failures

Should a longitudinal failure of the rod travel housing occur, the region of the stainless steel tube opposite the break would be stressed by the reactor coolant pressure of 2250 psia. The most probable leakage path would be provided by the radial deformation of the position indicator coil assembly, resulting in the growth of the axial flow passages between the rod travel housing and the stainless steel tube. A radial free water jet is not expected to occur because of the small clearance between the stainless steel tube and the rod travel housing, and the considerable resistance of the combination of the stainless steel tube and the position indicator coils to internal pressure. Calculations based on the mechanical properties of stainless steel and copper at reactor operating temperature show that an internal pressure of at least 4000 psia would be necessary for the combination of the stainless steel tube and the coils to rupture.

Therefore, the combination of stainless steel tube and copper coils stack is more than adequate to prevent formation of a radial jet following a control rod housing split which ensures the integrity of the adjacent rod housings.

3.9.4.3.3 Effect of Rod Travel Housing Circumferential Failures

If circumferential failure of a rod travel housing should occur, the broken-off section of the housing would be ejected vertically because the driving force is vertical and the position indicator coil stack assembly and the drive shaft would tend to guide the broken-off piece upwards during its travel. Travel is limited to less than 2 ft by the missile shield, thereby limiting the projectile acceleration. When the projectile reaches the missile shield, it would

partially penetrate the shield and dissipate its kinetic energy. The water jet from the break would push the broken-off piece against the missile shield.

If the broken-off piece were short enough to clear the break when fully ejected, it could rebound after impact with the missile shield. The top end plates of the position indicator coil stack assemblies and the coil stacks would prevent the broken piece from directly hitting the rod travel housing of a second drive mechanism. Even if a direct hit by the rebounding piece were to occur, the low kinetic energy of the rebounding projectile would not be expected to cause significant damage.

3.9.4.3.4 Summary

The considerations given above lead to the conclusion that failure of a control rod housing due to either longitudinal or circumferential cracking would not cause damage to adjacent housings that would increase the severity of the initial accident.

3.9.5 REACTOR PRESSURE VESSEL INTERNALS

3.9.5.1 Design Arrangements

The reactor pressure vessel internals are shown in Figures 3.9-9 and 3.9-10. The internals, consisting of the upper and lower core support structure, are designed to support, align, and guide the core components, direct the coolant flow to and from the core components, and to support and guide the in-core instrumentation.

The components of the reactor internals are divided into three parts consisting of the lower core support structure (including the entire core barrel and thermal shield), the upper core support structure, and the in-core instrumentation support structure.

3.9.5.1.1 Lower Core Support Structure

3.9.5.1.1.1 Support Structure Assembly

The major containment and support member of the reactor internals is the lower core support structure. This support structure assembly consists of the core barrel, the core baffle, the lower core plate and support columns, the thermal shield, the intermediate diffuser plate, and the bottom support plate which is welded to the core barrel. All the major material for this structure is type 304 stainless steel. The core support structure is supported at its upper flange from a ledge in the reactor vessel head flange and its lower end is restrained in its transverse movement by a radial support system attached to the vessel wall. Within the core barrel are axial baffle and former plates which are attached to the core barrel wall and form the enclosure periphery of the assembled core. The lower core plate is positioned at the bottom level of the core below the baffle plates and provides support and orientation for the fuel assemblies.

3.9.5.1.1.2 Lower Core Plate

The lower core plate is a 1.5-in.-thick member through which the necessary flow distributor holes for each fuel assembly are machined. Fuel assembly locating pins (two for each assembly) are also inserted into this plate. Columns are placed between this plate and the

bottom support plate of the core barrel in order to provide stiffness to this plate and transmit the core load to the bottom support plate. Intermediate between the support plate and lower core support plate is positioned a perforated plate to diffuse uniformly the coolant flowing into the core.

3.9.5.1.1.3 Thermal Shield

The thermal shield is a solid, relatively thick (3.56 in.) cylinder that is supported from the core barrel at both the top and bottom end.

The upper end of the shield is rigidly connected to the core barrel at six equally spaced points through mounting pads projecting from the core barrel. This connection is designed to prevent relative motion between the shield and barrel in both the radial and axial direction.

To provide for a difference in axial elongation between the shield and core barrel resulting from the temperature distribution at operation conditions, the lower connection is designed to allow axial movement between the two members but restrict the radial movement. This is accomplished by means of six flexible strap connections between the shield and barrel. These relatively thin straps are sufficiently flexible to withstand the axial displacement between the shield at core barrel but have sufficient width and cross-section area to restrict the radial motion.

A rigid connection is used at the upper end of the shield to obtain the inherent stability of suspending a heavy mass from the top and also because field and model tests have indicated that the maximum disturbing forces occur at the upper end.

Response of the thermal shield to the design dynamic loading was determined for both ring and beam mode vibration. The resulting force and moment reactions were used in determining the design requirements of the upper and lower connections.

The design dynamic loading used was considerably greater than any expected loading, based on measurements of actual pressure fluctuations during hot functional tests and also from model tests. The total stress was obtained by combining the thermal stresses, resulting from axial and radial elongation, with the anticipated dynamic stresses.

Irradiation baskets in which materials samples can be inserted and irradiated during reactor operation are attached to the thermal shield. The irradiation capsule basket supports are welded to the thermal shield. There is no extension of this support above the thermal shield as was done in the older designs. Thus, the basket has been removed from the high flow disturbance zone. The welded attachment to the shield extends the full length of the support except for small interruptions about 1 in. long. This type of attachment has an extremely high natural frequency. The specimens are held in position within the baskets by a stop at the bottom and a slotted cylindrical spring at the top which fits against a relief in the basket. The specimen does not extend through the top of the basket and thus is protected by the basket from the flow.

3.9.5.1.1.4 Coolant Flow Passages

The lower core support structure and the core barrel serve to provide passageways and control for the coolant flow. Inlet coolant flow from the vessel inlet nozzles proceeds down the annulus between the core barrel and the vessel wall, flows on both sides of the thermal shield, and then into a plenum at the bottom of the vessel. It then turns and flows up through the lower support plate, passes through the intermediate diffuser plate and then through the lower core plate. The flow holes in the diffuser plate and the lower core plate are arranged to give a very uniform entrance flow distribution to the core. After passing through the core, the coolant enters the area of the upper support structure and then flows, generally radially, to the core barrel outlet nozzles and directly through the vessel outlet nozzles.

A small amount of water also flows between the baffle plates and core barrel to provide additional cooling of the barrel. Similarly, a small amount of the entering flow is directed into the vessel head plenum and exits through the vessel output nozzles.

3.9.5.1.1.5 Support and Alignment Arrangements

Vertical downward loads from weight, fuel assembly preload, control rod dynamic loading, and earthquake acceleration are carried by the lower core plate, partially into the lower core plate support flange on the barrel shell and partially through the lower support columns to the bottom support plate. From there the loads are carried through the core barrel shell to the core barrel flange supported by the vessel head flange. Transverse loads from earthquake acceleration, coolant cross flow, and vibration are carried by the core barrel shell to be shared by the lower radial support to the vessel head flange. Transverse acceleration of the fuel assemblies is transmitted to the core barrel shell by direct connection of the lower core support plate to the barrel shell, by direct connection of the lower core support plate to the barrel wall, and by a radial support type connection of the upper core plate to slab-sided pins pressed into the core barrel.

The main radial support system of the core barrel is accomplished by key and keyway joints to the reactor vessel wall. At equally spaced points around the circumference, an Inconel block is welded to the vessel I.D. Another Inconel block is bolted to each of these blocks, and has a keyway geometry. Opposite each of these is a key which is attached to the internals. At assembly, as the internals are lowered into the vessel, the keys engage the keyways in the axial direction. With this design, the internals are provided with a support at the furthest extremity and may be viewed as a beam fixed at the top and simply supported at the bottom.

Radial and axial expansions of the core barrel are accommodated but transverse movement of the core barrel is restricted by this design. With this system, cycle stresses in the internal structures are within the ASME Section III limits. This eliminates any possibility of failure of the core support.

3.9.5.1.2 Upper Core Support Assembly

The upper core support assembly consists of the top support plate, deep beam sections, and upper core plate between which are contained support columns and guide tube assemblies. The support columns establish the spacing between the top support plate, deep beam sections,

and the upper core plate and are fastened at top and bottom to these plates and beams. The support columns transmit the mechanical loadings between the two plates and serve the supplementary function of supporting thermocouple guide tubes. The guide tube assemblies sheath and guide the control rod drive shafts and control rods, but provide no other mechanical function. They are fastened to the top support plate and are guided by pins in the upper core plate for proper orientation and support. Additional guidance for the control rod drive shafts is provided by the control rod shroud tube which is attached to the upper support plate and guide tube.

The upper core support assembly, which is removed as a unit during the MODE 6 (Refueling) operation, is positioned in its proper orientation with respect to the lower support structure by flat-sided pins pressed into the core barrel which in turn engage in slots in the upper core plate. At an elevation in the core barrel where the upper core plate is positioned, the flat-sided pins are located at equal angular positions. Slots are milled into the core plate at the same positions. As the upper support structure is lowered into the main internals, the slots in the plate engage the flat-sided pins in the axial direction. Lateral displacement of the plate and of the upper support assembly is restricted by this design. Fuel assembly locating pins protrude from the bottom of the upper core plate and engage the fuel assemblies as the upper assembly is lowered into place. Proper alignment of the lower core support structure, the upper core support assembly, the fuel assemblies, and control rods is ensured by this system of locating pins and guidance arrangement. The upper core support assembly is restrained from any axial movements by a large circumferential spring which rests between the upper barrel flange and the upper core support assembly and is compressed by the reactor vessel head flange.

Vertical loads from weight and fuel assembly preload are transmitted through the upper core plate via the support columns to the deep beams and top support plate and then through the circumferential spring to the reactor vessel head. Transverse loads from coolant cross flow, earthquake acceleration, and possible vibrations are distributed by the support columns to the top support plate and upper core plate. The top support plate is particularly stiff to minimize deflection.

3.9.5.1.3 In-Core Instrumentation Support Structures

The in-core instrumentation support structures consist of an upper system to convey and support thermocouples penetrating the vessel through the head and a lower system to convey and support flux thimbles penetrating the vessel through the bottom.

The upper system utilizes the reactor vessel head penetrations. Instrumentation port columns are slip-connected to in-line columns that are in turn fastened to the upper support plate. These port columns protrude through the head penetrations. The thermocouples are carried through these port columns and the upper support plate at positions above their readout locations. The thermocouple conduits are supported from the columns of the upper core support system. The thermocouple conduits are sealed stainless steel tubes.

In addition to the upper in-core instrumentation, there are reactor vessel bottom port columns which carry the retractable, cold-worked stainless steel flux thimbles that are pushed upward into the reactor core. Conduits extend from the bottom of the reactor vessel down through the

concrete shield area and up to a thimble seal line. The minimum bend radii are about 90 in. and the trailing ends of the thimbles (at the seal line) are extracted approximately 13 ft during MODE 6 (Refueling) of the reactor in order to avoid interference within the core. The thimbles are closed at the leading ends and serve as the pressure barrier between the reactor pressurized water and the containment atmosphere.

Mechanical seals between the retractable thimbles and the conduits are provided at the seal line. During MODES 1 and 2, the retractable thimbles are stationary and move only during MODE 6 (Refueling) or for maintenance, at which time a space of approximately 13 ft above the seal line is cleared for the retraction operation.

The in-core instrumentation support structure is designed for adequate support of instrumentation during reactor operation and is rugged enough to resist damage or distortion under the conditions imposed by handling during the MODE 6 (Refueling) sequence.

The flux mapping system includes a drive and control system for inserting the in-core detectors. A portion of the drive system, which includes the fifteen-path rotary transfer devices and the isolation valves, is mounted on the movable seal cart, which is normally located above the seal table (see Section 7.7.4.2.3). The seal cart is mounted on a rail structure used to move the seal cart out of the way during refueling. The seal cart is designed and restrained to prevent the flux mapping system from collapsing onto the seal table during a seismic event and jeopardizing the seal table reactor coolant system pressure boundary. The reactor bottom-mounted instrumentation system is Seismic Category I.

3.9.5.2 Loading Conditions

The internals are designed to withstand the forces due to weight, reload of fuel assemblies, control rod dynamic loading, vibration, and earthquake acceleration. Under the loading conditions, including conservative effects of design earthquake loading, the structure satisfies stress values prescribed in ASME Section III.

The reactor internal components are designed to withstand the stresses resulting from startup, steady-state operation with any number of pumps running, and shutdown conditions. The abnormal design conditions assume blowdown effects due to an accumulator line break or pressurizer surge line break.

3.9.5.3 Design Bases

The criteria for acceptability is that the core should be coolable and intact following a pipe rupture up to and including a double-ended rupture of the reactor coolant system. This implies that core cooling and adequate core shutdown must be ensured. Consequently, the limitations established on the internals are concerned principally with the maximum allowable deflections and/or stability of the parts. The allowable stress criteria is discussed in Section 3.9.2.3.1.3.

For abnormal operation the criteria for acceptability are that the reactor be capable of safe shutdown and that the engineered safety features are able to operate as designed. The limitation established on the internals for these types of loads are also concerned principally

with the maximum allowable deflections. The deflection criteria for critical structures under abnormal operation are presented in Table 3.9-29.

3.9.6 INSERVICE INSPECTION OF PUMPS AND VALVES

3.9.6.1 General

The following information defines the Inservice Pump and Valve Testing Program for the period starting January 1, 2010, through December 31, 2019. Included in this program are the quality groups A and B pumps which are provided with an emergency power source and those quality groups A, B, and C valves which are required to shut down the reactor or to mitigate the consequences of an accident and maintain the reactor in a safe shutdown condition. Quality groups A, B, and C components correspond to those defined in Regulatory Guide 1.26.

This program has been developed as required by Section 50.55a(g) of 10 CFR 50 following the guidance of the ASME OM Code-2004, "Code for Operation and Maintenance of Nuclear Power Plants." The program follows the guidance of Generic Letter 89-04 with possible exceptions approved by the NRC. The program was submitted to the NRC. The NRC has reviewed and approved the program and acted on program relief requests (*Reference 19*).

Further addenda and editions of ASME OM Code-2004 will be used for clarification of test requirements and performance.

The Inservice Pump and Valve Testing Program substantially augments but does not affect the pump and valve surveillance program required by the Technical Specifications. Technical Specifications requirements associated with pump and valve surveillance will continue to be implemented as specified. When changes to Technical Specifications create conflicts with the program, the revised Technical Specifications will provide guidance until the program is revised to incorporate the changes.

The motor-operated valve analysis and test system (MOVATS) program described in Section 5.4.9.3 supports the Inservice Pump and Valve Testing Program via Code Case OMN1.

When a valve, pump, or its control system has been replaced or repaired or has undergone maintenance that could affect its performance and prior to the time it is returned to service, it will be tested as necessary to demonstrate that the performance parameters which could have been affected by the replacement, repair, or maintenance are within acceptable limits.

Code Edition and Testing Interval

The Inservice Pump and Valve Testing Program for the period January 1, 2010, through December 31, 2019, was developed using the 2004 Edition of the ASME OM Code, "Code for Operation and Maintenance of Nuclear Power Plants."

3.9.6.2 Inservice Testing of Pumps

The inservice pump testing program was developed in accordance with the requirements of subsection ISTB of the ASME OM Code. This program includes all quality group A and B pumps, which are provided with an emergency power source and are required to perform a

specific function in shutting down the reactor or in mitigating the consequences of an accident and maintain the reactor in a safe shutdown condition.

The pumps to be tested and the test parameters and frequencies are specified in the inservice pump and valve testing program.

Testing of a pump need not be performed if that pump is declared inoperable without the testing. Consistent with the Technical Specifications, specified intervals may be extended by 25% to accommodate normal test schedules.

Records for the inservice pump testing program are developed and maintained in accordance with Subsection ISTA-9000, "Records and Reports" of the Code for Operation and Maintenance of Nuclear Power Plants.

3.9.6.3 Inservice Testing of Valves

The inservice valve testing program was developed in accordance with the requirements of subsection ISTC of the ASME OM Code. All those valves that are required to perform a specific function either to shut down the reactor to the MODE 5 (Cold Shutdown) condition or in mitigating the consequences of an accident and maintain the reactor in a safe shutdown condition are included in the program.

The inservice valve testing program requirements for category A, B, and C valves are included in the Pump and Valve Testing Program. Category D valves are not included in this testing program because there are none included in Ginna Station design.

Some exceptions and exemptions to the testing requirements of ISTC have been taken based on operational interference, placing the plant in an unsafe condition, and Technical Specifications requirements. All exceptions and exemptions are listed and explained in the Pump and Valve Testing Program.

Records for the inservice valve testing program are developed and maintained in accordance with Subsection ISTA-9000, "Records and Reports" of the Code for Operation and Maintenance of Nuclear Power Plants.

3.9.7 EXTENDED POWER UPRATE (EPU)

During the 2006 RFO, Ginna Station implemented Plant Change Request, PCR 2004-0009, "Ginna Station Extended Power Uprate (EPU) Project." Additional information to support EPU can be obtained from plant records associated with PCR 2004-0009 and *References 31* and *33*.

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Table 3.9-1
ORIGINAL DESIGN LOADING COMBINATIONS AND STRESS LIMITS

Loading Combinations	Vessels and Reactor Internals	Piping	Supports
Normal + design earthquake loads	$P_m \leq S_m$ $P_L + P_B \leq 1.5\ S_m$	$P_m \leq 1.2\ S$ $P_L + P_B \leq 1.2\ S$	Working stresses
Normal + maximum potential earthquake loads	$P_m \leq 1.2\ S_m$ $P_L + P_B \leq 1.2\ (1.5\ S_m)$	$P_m \leq 1.2\ S$ $P_L = P_B \leq 1.2\ (1.5\ S)$	Within yield after load redistribution
Normal + pipe rupture loads	$P_m \leq 1.2\ S_m$ $P_L + P_B \leq 1.2\ (1.5\ S_m)$	$P_m \leq 1.2\ S_m$ $P_L + P_B \leq 1.2\ (1.5\ S)$	Within yield after load redistribution

Where:

P_m = primary general membrane stress or stress intensity.

P_L = primary local membrane stress or stress intensity.

P_B = primary bending stress or stress intensity.

S_m = stress intensity value from ASME B&PV Code, Section III.

S = allowable stress from USAS B31.1 Code for Pressure Piping.

Table 3.9-2
RESIDUAL HEAT REMOVAL LOOP A STRESS SUMMARY

Description	Original[a] Design (psi)	As-Built[b] Condition (psi)	Allowable Stress (psi)
SEISMIC STRESSES			
Operating-basis earthquake			
Vertical + Z-horizontal	---	3,356	---
Vertical + X-horizontal	---	3,900	---
Safe shutdown earthquake			
Vertical + Z-horizontal	10,564	8,284	---
Vertical + X-horizontal	5,674	9,716	---
COMBINED STRESSES			
Operating-basis earthquake + pressure + deadweight	---	9,436	19,080
Safe shutdown earthquake + pressure + deadweight	16,715	15,252	28,620

a. Results obtained using WESTDYN and 1969 model which considers the supports rigid.

b. Results obtained using WESTDYN and as-built conditions considering support stiffnesses.

Table 3.9-3
MAIN STEAM LINE-LOOP B STRESS SUMMARY[a]

Description	As-Built Condition	
	Dynamic[a] Results (psi)	Allowable Stress (psi)
SEISMIC STRESSES		
Operating-basis earthquake		
Vertical + Z-horizontal	965	---
Vertical + X-horizontal	963	---
Safe shutdown earthquake		
Vertical + Z-horizontal	2,373	---
Vertical + X-horizontal	2,238	---
COMBINED STRESSES		
Operating-basis earthquake + pressure + deadweight	7,278	16,440
Safe shutdown earthquake + pressure + deadweight	8,686	24,660
NOTE: Additional evaluations to support Ginna Extended Power Uprate are available from plant records associated with PCR 2004-0009 and *Reference 31*.		

a. Stresses given are obtained using B31.1-1973 Summer Addenda, formula 12.

Table 3.9-4
CHARGING LINE STRESS SUMMARY[a]

Description	As-Built Dynamic Analysis Condition (psi)	Allowable Stress (psi)
SEISMIC STRESSES		
Operating-basis earthquake		
Vertical + Z-horizontal	150	---
Vertical + X-horizontal	245	---
Safe shutdown earthquake		
Vertical + Z-horizontal	436	---
Vertical + X-horizontal	638	---
COMBINED STRESSES		
Operating-basis earthquake + pressure + deadweight	6,941	20,580
Safe shutdown earthquake + pressure + deadweight	7,334	30,870

a. Stresses given are obtained using B31.1-1973 Summer Addenda, formula 12.

Table 3.9-5
LOAD COMBINATIONS AND ACCEPTANCE CRITERIA FOR PRESSURIZER SAFETY AND RELIEF VALVE PIPING AND SUPPORTS - UPSTREAM OF VALVES

Combination	Plant/System Operating Condition	Load Combination [a]	Piping Allowable Stress Intensity
1	Normal	N	1.0 S_h
2	Upset	N + OBE + SOT_U	1.2 S_h
3	Emergency	N + SOT_E	1.8 S_h
4	Faulted	N + MS/FWPB or DBPB + SSE + SOT_F	2.4 S_h
5	Faulted	N + LOCA + SSE + SOT_F	2.4 S_h

a. Definitions of load abbreviations are in Table 3.9-7.

Table 3.9-6
LOAD COMBINATIONS AND ACCEPTANCE CRITERIA FOR PRESSURIZER SAFETY AND RELIEF VALVE PIPING AND SUPPORTS - SEISMICALLY DESIGNED DOWNSTREAM PORTION

Combination	Operating Condition	Load Combination [a]	Piping Allowable Stress Intensity
1	Normal	N	1.0 S_h
2	Upset	N + SOT_U	1.2 S_h
3	Upset	N + OBE + SOT_U	1.8 S_h
4	Emergency	N + SOT_E	1.8 S_h
5	Faulted	N + MS/FWPB or DBPB + SSE + SOT_F	2.4 S_h
6	Faulted	N + LOCA + SSE + SOT_F	2.4 S_h

a. Definitions of load abbreviations are in Table 3.9-7.

Table 3.9-7
DEFINITIONS OF LOAD ABBREVIATIONS [a]

N	Sustained loads during normal plant operation
SOT	System operating transient
SOT_U	Relief valve discharge transient
SOT_E	Safety valve discharge transit
SOT_F	Maximum of SOT_U and SOT_E; or transition flow
OBE	Operating-basis earthquake
SSE	Safe shutdown earthquake
MS/FWPB	Main steam or feedwater pipe break
DBPB	Design-basis pipe break
LOCA	Loss-of-coolant accident
S_h	Basic material allowable stress at maximum (hot) temperature

a. Abbreviations used in TABLES 3.9-5 and 3.9-6.

Table 3.9-8
LOADING COMBINATIONS AND STRESS LIMITS FOR PIPING FOR SEISMIC UPGRADE PROGRAMS

	Loading Combinations	**Stress Limits**
DEADWEIGHT	Design Pressure + Deadweight	$P_m \leq S_h$; $P_L + P_B \leq S_h$
OBE SEISMIC	Design Pressure + Deadweight Design + Earthquake Loads (OBE)	$P_m \leq 1.2\ S_h$; $P_L + P_B \leq 1.2\ S_h$
SSE	Operating Pressure + Deadweight + Maximum Potential Earthquake Loads (SSE)	$P_m \leq 1.8\ S_h$; $P_L + P_B \leq 1.8\ S_h$
THERMAL	Maximum Operating Thermal + OBE Displacements	$S_E \leq S_A$
	Design Pressure + Deadweight + Maximum Operating Thermal + OBE Displacements	$P_L + P_B \leq (S_h + S_A)$

Where:

OBE = operating-basis earthquake

P_m = primary general membrane stress; or stress intensity

P_L = primary local membrane stress; or stress intensity

P_B = primary bending stress; or stress intensity

S_A, S_h = allowable stress from USAS B31.1 Code for pressure piping

S_E = thermal expansion stress from USAS B31.1 code for pressure piping

SSE = safe shutdown earthquake

Table 3.9-9
ALLOWABLE STEAM GENERATOR NOZZLE LOADS

Condition	Fx	Fy	Fz	Mx	My	Mz
FEEDWATER NOZZLE						
Thermal	15	40	40	1000	1500	1500
Weight	5	15	5	250	500	500
Seismic operating-basis earthquake	75	75	75	1500	2000	2000
Seismic design-basis earthquake	100	100	100	2000	3000	3000
STEAM NOZZLE						
Thermal	100	50	50	6000	5000	5000
Weight	20	10	10	500	500	750
Seismic operating-basis earthquake	150	150	150	5000	5000	5000
Seismic design-basis earthquake	200	200	200	7500	7500	7500

Notes:

1. All loads are ± unless indicated.
2. Units are kips and in -kips.
3. Coordinate system

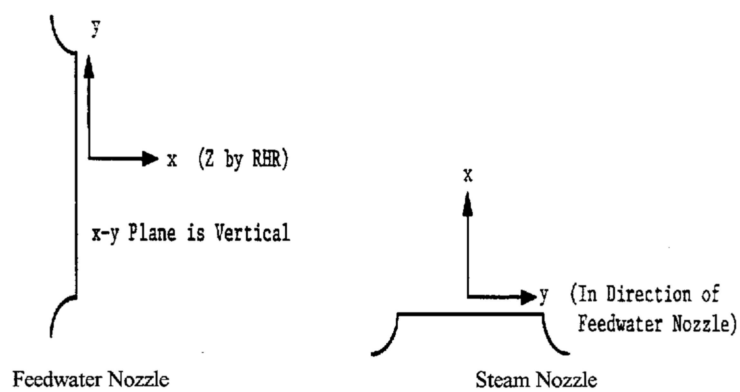

Table 3.9-10
REACTOR COOLANT PUMP AUXILIARY NOZZLE UMBRELLA LOADS

Nozzle	Condition /Load	Fx (lb)	Fy (lb)	Fz (lb)	Mx (in.-lb)	My (in.-lb)	Mz (in.-lb)
Seal injection	Thermal	350	100	300	3500	2800	2000
	Dead-weight	10	-80	10	300	250	400
	Seismic OBE	250	50	225	1600	4500	2000
	Seismic SSE	800	250	350	3200	15000	4000
No. 1 seal bypass	Thermal	75	70	40	300	315	1525
	Dead-weight	5	-25	1	75	50	350
	Seismic OBE	50	50	45	900	1200	900
	Seismic SSE	160	170	170	1650	2550	2000
No. 1 seal leakoff	Thermal	400	200	300	2000	2000	2000
	Dead-weight	10	-80	5	300	250	400
	Seismic OBE	500	400	500	1000	5000	2000
	Seismic SSE	800	500	600	2000	8000	3500
No. 2 seal leakoff	Thermal	75	100	100	300	350	1600
	Dead-weight	5	-25	5	75	75	400
	Seismic OBE	50	100	100	900	1500	1200
	Seismic SSE	160	170	170	1650	2500	2000

Nozzle	Condition /Load	Fx (lb)	Fy (lb)	Fz (lb)	Mx (in.-lb)	My (in.-lb)	Mz (in.-lb)
No. 3 seal injection	Thermal	90	45	45	290	290	180
	Dead-weight	15	35	10	90	45	180
	Seismic OBE	90	150	150	480	560	480
	Seismic SSE	180	300	300	960	1120	960
No. 3 seal leakoff	Thermal	90	45	45	290	290	180
	Dead-weight	15	35	10	90	45	180
	Seismic OBE	90	150	150	480	560	480
	Seismic SSE	180	300	300	960	1120	960
Thermal barrier component cooling water in and out	Thermal	75	200	150	3200	1300	2500
	Dead-weight	20	-75	1	5	5	150
	Seismic OBE	100	250	100	1000	1200	1200
	Seismic SSE	200	700	200	4500	3000	3600
Upper bearing oil cooler and air cooler component cooling water in and out	Thermal	100	100	100	300	300	200
	Dead-weight	5	-80	5	100	50	200
	Seismic OBE	100	300	300	500	600	500
	Seismic SSE	200	600	600	1000	1200	1000
Lower bearing oil cooler component cooling water in and out	Thermal	95	340	305	470	480	525
	Dead-weight	10	-35	10	100	125	125
	Seismic OBE	90	90	90	290	290	180
	Seismic SSE	90	90	90	290	290	180

Nozzle	Condition /Load	Fx (lb)	Fy (lb)	Fz (lb)	Mx (in.-lb)	My (in.-lb)	Mz (in.-lb)

Note:
1. Values at ± unless otherwise specified.
2. Loads on the No. 3 seal connections apply only if a No. 3 "Double Dam" seal is supplied.
3. Loads on pump nozzles are to be applied at the nozzle to shell juncture.
4. Loads on motor nozzles are to be applied at the flange end.
5. Coordinate system.
6. OBE = operating-basis earthquake.
7. SSE = safe shutdown earthquake.

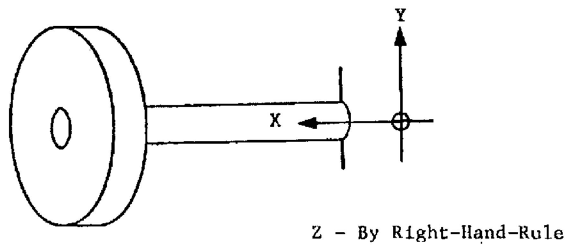

Table 3.9-11
SYSTEMATIC EVALUATION PROGRAM STRUCTURAL BEHAVIOR CRITERIA FOR DETERMINING SEISMIC DESIGN ADEQUACY

Components	**Systematic Evaluation Program Criteria, Safe Shutdown Earthquake**	
Vessels, pumps, and valves	$S_{m\ (all)} \leq 0.7\ S_u$ and $1.6\ S_y$	ASME III Class 1 (Table F 1322.2.1)
	$S_{m\ (all)} \leq 0.67\ S_u$ and $1.33\ S_y$	ASME III Class 2 (NC 3217)
	$\sigma_{m\ (all)} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 2 (NC 3321)
	$\sigma_{m\ (all)} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 3 (ND 3321)
Piping	$S_{m\ (all)} \leq 1.0\ S_u$ and $2.0\ S_y$	ASME III Class 1 (Table F 1322.2.1)
	$S_h \leq 0.6\ S_u$ and $1.5\ S_y$	ASME III Class 2 and Class 3 (NC 3611.2)
Tanks	No ASME III Class 1	
	$\sigma_{m\ (all)} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 2 and Class 3 (NC 3821)
Electric equipment	$S_{(all)} \leq 1.0\ S_y$	
Cable trays	$S_{(all)} \leq 1.0\ S_y$	
ASME supports	$S_{(all)} \leq 1.2\ S_y$ and $0.7\ S_u$	ASME III Appendices XVII, F for Class 1, 2 and 3
Other supports	$S_{(all)} \leq 1.6\ S$	Normal AISC S allowable increased by 1.6 consistent with NRC Standard Review Plan, Sec. 3.8
Bolting	$S_{(all)} \leq 1.4\ S$	ASME Section III Appendix XVII for bolting where S is the allowable stress for design loads
NOTE:—$S_{(all)}$ = Stress Allowable.		

Table 3.9-12
MECHANICAL COMPONENTS SELECTED FOR SEP SEISMIC REVIEW

Item	Mechanical Component Description	Reason for Selection
1	Essential service water pump	This item has a long vertical unsupported intake section which was originally statically analyzed for seismic effects.
2	Component cooling heat exchanger	This item is supported on what appears to be a relatively flexible structural steel framing and by two saddles.
3	Component cooling surge tank	Same as Item 2.
4	Diesel-generator air tanks	This item is a skirt-supported vertical tank.
5	Boric acid storage tank	This item is a column-supported vertical tank.
6	Refueling water storage tank (RWST)	Evaluate anchor-bolt systems for in-structure flat-bottom tanks that are flexible.
7	Motor-operated valves	A general concern with respect to motor-operated valves, particularly for lines 4 in. or less in diameter, is that the relatively large eccentric mass of the motor will cause excessive stresses in the attached piping if the valves are not externally supported.
8	Steam generators	Items are particularly critical to ensure reactor coolant system integrity.
9	Reactor coolant pumps	Same as Item 8.
10	Pressurizer	Same as Item 8.
11	Control rod drive mechanism	Same as Item 8.
12	Reactor coolant system supports	Same as Item 8.

Table 3.9-13
MAXIMUM STRESS HOT-LEG BREAK (ORIGINAL ANALYSIS)

-	Stresses			Allowable	
Components	**Direct**	**Bending**	**Total**	**Direct**	**Total**
Core plate	0	17,800	17,800	39,500	50,000
Upper support columns	15,000	---	15,000	39,500	50,000
Top nozzle (minor)	0	24,800	24,800	39,500	50,000
Top nozzle (major)	0	20,600	20,600	39,500	50,000
Flange barrel	4,000	31,800	35,800	39,500	50,000
Lower support structure	0	7,670	7,670	39,500	50,000
Barrel	3,200	0	3,200	39,500	50,000
Fuel assembly thimbles	40,400	---	40,400	45,000	---

Table 3.9-14
MAXIMUM STRESS COLD-LEG BREAK (ORIGINAL ANALYSIS)

-	Stresses			Allowable	
Components	**Direct**	**Bending**	**Total**	**Direct**	**Total**
Upper core plate	0	4,800	4,800	39,500	50,000
Upper support column	8,700	0	8,700	39,500	50,000
Bottom nozzle (minor assembly)	0	45,200	45,200	39,500	50,000
Bottom nozzle (major assembly)	0	47,800	47,800	39,500	50,000
Flange barrel	4,000	31,800	35,800	39,500	50,000
Lower support structure	0	21,400	21,400	39,500	50,000
Barrel	11,500	0	11,500	39,500	50,000
Lower core plate	0	8,400	8,400	39,500	50,000
Fuel assembly thimbles	40,400	---	40,400	45,000	---

Table 3.9-15
MAXIMUM CORE BARREL STRESS AND DEFLECTION UNDER HOT-LEG BLOWDOWN (ORIGINAL ANALYSIS)

Rupture Time (msec)	Maximum Deflection (in.)	Allowable Deflection (in.)	Maximum Stress (psi)	Allowable Stress (psi)	Compressive Wave (psi)	Critical Pressure (psi)
1	0.031	5	14,110	39,500	450	2,612

Table 3.9-16a
MAXIMUM STRESS INTENSITIES AND DEFLECTION COLD-LEG BLOWDOWN (ORIGINAL ANALYSIS) - IN THE UPPER BARREL

Rupture Time (msec)	Maximum Stress Intensity (psi)	Allowable Stress Intensity (psi)	Maximum Membrane Stress (psi)	Allowable Membrane Stress (psi)	Maximum Deflection (mils)
1	44,500	50,000	36,750	39,500	150
5	34,500	50,000	26,750	39,500	95
20	34,500	50,000	26,750	39,500	95

Table 3.9-16b
MAXIMUM STRESS INTENSITIES AND DEFLECTION COLD-LEG BLOWDOWN (ORIGINAL ANALYSIS) - AT THE UPPER BARREL ENDS

Rupture Time (msec)	Rise Time (msec)	Peak Pressure (psi)	Maximum Upper Bending Stress (psi)	Maximum Lower Bending Stress (psi)	Allowable (psi)
1	2	750	49,800	26,850	50,000
5	4.5	650	40,370	21,755	50,000
20	4.5	650	40,370	21,755	50,000

Table 3.9-17
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

-	Primary Principal Stresses		
Barrel Flange Weld	S_1 (psi) (Tangential)	S_2 (psi) (Longitudinal)	S_3 (psi) (Radial)
OUTSIDE SURFACE			
Normal operating	2159	2797	-1655
0.08g vertical earthquake	0	141	0
0.08g horizontal earthquake	0	90	0
Normal operating + 0.08g earthquake	2159	3028	-1655
0.20g vertical earthquake	0	235	0
0.20g horizontal earthquake	0	150	0
Normal operating + 0.20g earthquake	2159	3413	-1655
INSIDE SURFACE			
Normal operating	3378	-1825	-1618
0.08g vertical earthquake	0	14	0
0.08g horizontal earthquake	0	90	0
Normal operating + 0.08g earthquake	3378	-1594	-1618
0.20g vertical earthquake	0	235	0
0.20g horizontal earthquake	0	150	0
Normal operating + 0.20g earthquake	3378	-1209	-1618
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).			

Table 3.9-18
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

-	**Primary Principal Stresses**		
Barrel Middle Girth Weld	**S_1 (psi) (Tangential)**	**S_2 (psi) (Longitudinal)**	**S_3 (psi) (Radial)**
OUTSIDE SURFACE			
Normal operating	-5686	-9347	-2250
0.08g vertical earthquake	0	307	0
0.08g horizontal earthquake	0	235	0
Normal operating + 0.08g earthquake	-5686	-8805	-2250
0.20g vertical earthquake	0	512	0
0.20g horizontal earthquake	0	392	0
Normal operating + 0.20g earthquake	-5686	-7901	-2250
INSIDE SURFACE			
Normal operating	-5414	-8295	-2200
0.08g vertical earthquake	0	307	0
0.08g horizontal earthquake	0	235	0
Normal operating + 0.08g earthquake	-5414	-7753	-2200
0.20g vertical earthquake	0	512	0
0.20g horizontal earthquake	0	392	0
Normal operating + 0.20g earthquake	-5414	-6849	2200
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).			

Table 3.9-19
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

-	Primary Principal Stresses		
Barrel Lower Girth Weld	S_1 (psi) (Tangential)	S_2 (psi) (Longitudinal)	S_3 (psi) (Radial)
OUTSIDE SURFACE			
Normal operating	-4059	-6608	0
0.08g vertical earthquake	0	165	
0.08g horizontal earthquake	0	35	0
Normal operating + 0.08g earthquake	-4059	-6408	-609
0.20g vertical earthquake	0	275	0
0.20g horizontal earthquake	0	58	0
Normal operating + 0.20g earthquake	-4059	-6075	-609
INSIDE SURFACE			
Normal operating	1103	7962	916
0.08g vertical earthquake	0	165	0
0.08g horizontal earthquake	0	35	0
Normal operating + 0.08g earthquake	1103	8162	916
0.20g vertical earthquake	0	275	0
0.20g horizontal earthquake	0	58	0
Normal operating + 0.20g earthquake	1103	8495	916
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).			

Table 3.9-20
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

Barrel Flange Weld	Maximum Primary Stress Intensity (psi)
Outside Surface	
Normal operating + 0.08g earthquake	4683
Normal operating + 0.20g earthquake	5068
Inside Surface	
Normal operating + 0.08g earthquake	4996
Normal operating + 0.20g earthquake	4996
Barrel Middle Girth Weld	
Outside Surface	
Normal operating + 0.08g earthquake	6555
Normal operating + 0.20g earthquake	5651
Inside Surface	
Normal operating + 0.08g earthquake	5553
Normal operating + 0.20g earthquake	4649
Barrel Lower Girth Weld	
Outside Surface	
Normal operating + 0.08g earthquake	5799
Normal operating + 0.20g earthquake	5466
Inside Surface	
Normal operating + 0.08g earthquake	7246
Normal operating + 0.20g earthquake	7579
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).	

Table 3.9-21
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

	Primary Plus Secondary Principal Stresses		
	S_1 (psi) (Tangential)	S_2 (psi) (Longitudinal)	S_3 (psi) (Radial)
Barrel Flange Weld			
OUTSIDE SURFACE			
Normal operating + 0.08g earthquake	10,289	20,135	-1,640
Normal operating + 0.20g earthquake	10,289	20,520	-1,640
INSIDE SURFACE			
Normal operating + 0.08g earthquake	6,298	-4,963	-1,603
Normal operating + 0.20g earthquake	6,298	-4,578	-1,603
Barrel Middle Girth Weld			
OUTSIDE SURFACE			
Normal operating + 0.08g earthquake	2,768	4,071	-2,261
Normal operating + 0.20g earthquake	2,768	4,975	-2,261
INSIDE SURFACE			
Normal operating + 0.08g earthquake	-17,206	-20,666	-2,211
Normal operating + 0.20g earthquake	-17,206	-19,762	-2,211
Barrel Lower Girth Weld			
OUTSIDE SURFACE			
Normal operating + 0.08g earthquake	-4,059	-6,408	-609

	Primary Plus Secondary Principal Stresses		
	S_1 (psi) (Tangential)	S_2 (psi) (Longitudinal)	S_3 (psi) (Radial)
Normal operating + 0.20g earthquake	-4,059	-6,075	-609
INSIDE SURFACE			
Normal operating + 0.08g earthquake	1,103	8,162	916
Normal operating + 0.20g earthquake	1,103	8,459	916
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).			

Table 3.9-22
CORE BARREL STRESSES (ORIGINAL ANALYSIS)

	Maximum Primary Plus Secondary Stress Intensity (psi)
Barrel Flange Weld	
OUTSIDE SURFACE	
Normal operating + 0.08g earthquake	21,775
Normal operating + 0.20g earthquake	22,160
INSIDE SURFACE	
Normal operating + 0.08g earthquake	11,261
Normal operating + 0.20g earthquake	10,876
Barrel Middle Girth Weld	
OUTSIDE SURFACE	
Normal operating + 0.08g earthquake	6,332
Normal operating + 0.20g earthquake	7,263
INSIDE SURFACE	
Normal operating + 0.08g earthquake	18,455
Normal operating + 0.20g earthquake	17,551
Barrel Lower Girth Weld	
OUTSIDE SURFACE	
Normal operating + 0.08g earthquake	5,799
Normal operating + 0.20g earthquake	5,466
INSIDE SURFACE	
Normal operating + 0.08g earthquake	7,246
Normal operating + 0.20g earthquake	7,579

Note: The values in this Table remains bounding for Extended Power Uprate (EPU).

Table 3.9-23a
LOAD COMBINATIONS AND ALLOWABLE STRESS LIMITS FOR PRIMARY EQUIPMENT SUPPORTS EVALUATION - FOR PLANT EVENTS

	Plant Event	Plant Operating Conditions	Service Loading Combinations[a]	Service Level Stress Limits [b]
1.	Normal operation (MODES 1 and 2)	Normal	Sustained loads	A
2.	Plant/system operating transients (SOT) + OBE	Upset	Sustained loads + SOT + OBE	B
3.	DBPB	Emergency	Sustained loads + DBPB	C
4.	SSE	Faulted	Sustained loads + SSE	D
5.	DBPB (or MS/FWPB) + SSE	Faulted	Sustained loads + (DBPB D or MS/FWPB + SSE)	

a. The pipe break loads and SSE loads are combined by the square root sum of the squares method.

b. Stress levels are defined by ASME Code, Section III, Subsection NF, 1974 edition.

Table 3.9-23b
LOAD COMBINATIONS AND ALLOWABLE STRESS LIMITS FOR PRIMARY EQUIPMENT SUPPORTS EVALUATION - DEFINITION OF LOADING CONDITIONS FOR PRIMARY EQUIPMENT SUPPORTS EVALUATION IN TABLE 3.9-23a

1.	Sustained loads	DW, deadweight +P, operating pressure +TN, normal operating thermal
2.	Transients	SOT, system operating transient
3.	Overtemperature transient	TA
4.	Operating-basis earthquake	OBE
5.	Safe shutdown earthquake	SSE
6.	Design basis pipe break / design basis accident	DBPB/DBA
	Residual heat removal line	RHR
	Accumulator line	ACC
	Pressurizer surge line	SURG
7.	Main steam line break	MS
8.	Feedwater pipe break	FW

Table 3.9-24
RESIDUAL HEAT REMOVAL LOOP A SUPPORT LOADS[a] CALCULATED FOR IE BULLETIN 79-07

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
RH-34 vertical	Operating-basis earthquake		3600
	Vertical + Z-Horizontal	2820	
	Vertical + X-Horizontal	2720	
	Safe shutdown earthquake		5400
	Vertical + Z-Horizontal	3370	
	Vertical + X-Horizontal	3110	
RH-8 vertical	Operating-basis earthquake		1680
	Vertical + Z-Horizontal	1110	
	Vertical + X-Horizontal	1260	
	Safe shutdown earthquake		2520
	Vertical + Z-Horizontal	1340	
	Vertical + X-Horizontal	1680	
RH-7 vertical	Operating-basis earthquake		2160
	Vertical + Z-Horizontal	1080	
	Vertical + X-Horizontal	1090	
	Safe shutdown earthquake		3240
	Vertical + Z-Horizontal	1200	
	Vertical + X-Horizontal	1220	
RH-6 horizontal	Operating-basis earthquake		5640
	Vertical + Z-Horizontal	990	

a. Support load combination is seismic plus deadweight.

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + X-Horizontal	860	
	Safe shutdown earthquake		8460
	Vertical + Z-Horizontal	2390	
	Vertical + X-Horizontal	2030	
RH-5 vertical	Operating-basis earthquake		2160
	Vertical + Z-Horizontal	740	
	Vertical + X-Horizontal	740	
	Safe shutdown earthquake		3240
	Vertical + Z-Horizontal	930	
	Vertical + X-Horizontal	930	
RH-4 horizontal	Operating-basis earthquake		3720
	Vertical + Z-Horizontal	600	
	Vertical + X-Horizontal	780	
	Safe shutdown earthquake		5580
	Vertical + Z-Horizontal	1390	
	Vertical + X-Horizontal	1850	
RH-3 vertical	Operating-basis earthquake		2160
	Vertical + Z-Horizontal	1910	
	Vertical + X-Horizontal	1880	
	Safe shutdown earthquake		3240

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + Z-Horizontal	2250	
	Vertical + X-Horizontal	2180	
RH-2 vertical	Operating-basis earthquake		2160
	Vertical + Z-Horizontal	1600	
	Vertical + X-Horizontal	1600	
	Safe shutdown earthquake		3240
	Vertical + Z-Horizontal	1920	
	Vertical + X-Horizontal	1930	
RH-1 vertical	Operating-basis earthquake		2160
	Vertical + Z-Horizontal	1780	
	Vertical + X-Horizontal	1870	
	Safe shutdown earthquake		3240
	Vertical + Z-Horizontal	2200	
	Vertical + X-Horizontal	2420	
RH-1 horizontal	Operating-basis earthquake		3720
	Vertical + Z-Horizontal	324	
	Vertical + X-Horizontal	880	
	Safe shutdown earthquake		5580
	Vertical + Z-Horizontal	780	
	Vertical + X-Horizontal	2150	

Table 3.9-25a
MAIN STEAM LINE LOOP B SUPPORT LOADS[a] CALCULATED FOR IE BULLETIN 79-07 - SEISMIC SUPPORT

Seismic Supports	Description	As-Built Conditions (lb)	Design Load (lb)
MS-7	Operating-basis earthquake		
	Vertical + Z-Horizontal	3,040	21,000
	Vertical + X-Horizontal	6,930	21,000
	Safe shutdown earthquake		
	Vertical + Z-Horizontal	6,200	21,000
	Vertical + X-Horizontal	14,060	21,000
MS-8	Operating-basis earthquake		
	Vertical + Z-Horizontal	6,140	21,000
	Vertical + X-Horizontal	5,260	21,000
	Safe shutdown earthquake		
	Vertical + Z-Horizontal	15,350	21,000
	Vertical + X-Horizontal	13,240	21,000

a. Support load combination is seismic plus deadweight.

Table 3.9-25b
MAIN STEAM LINE LOOP B NOZZLE LOADS CALCULATED FOR IE BULLETIN -79-07 - NOZZLE LOADS

NOZZLE LOADS	-	-	-	-	-	-
		WESTDYN Local Coordinate System				
Description		KIPS			IN-KIPS	
OBE induced load	9	2	4	300	209	514
Seismic OBE allowable loads	150	150	150	5000	5000	5000
SSE induced loads	15	5	4	649	279	1160
Seismic SSE allowable loads	200	200	200	7500	7500	7500

Table 3.9-26
CHARGING LINE SUPPORT LOADS[a] CALCULATED FOR IE BULLETIN 79-07

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
S-35 vertical	Operating-basis earthquake		1,500
	Vertical + Z-Horizontal	570	
	Vertical + Z-Horizontal	580	
	Safe shutdown earthquake		2,250
	Vertical + Z-Horizontal	620	
	Vertical + Z-Horizontal	600	
S-60 vertical	Operating-basis earthquake		1,500
	Vertical + Z-Horizontal	20	
	Vertical + Z-Horizontal	20	
	Safe shutdown earthquake		2,250
	Vertical + Z-Horizontal	30	
	Vertical + Z-Horizontal	30	
S-135 vertical	Operating-basis earthquake		8,850
	Vertical + Z-Horizontal	40	
	Vertical + Z-Horizontal	40	
	Safe shutdown earthquake		12,750
	Vertical + Z-Horizontal	40	
	Vertical + Z-Horizontal	40	
S-135 axial	Operating-basis earthquake		8,500
	Vertical + Z-Horizontal	65	

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + Z-Horizontal	65	
	Safe shutdown earthquake		12,750
	Vertical + Z-Horizontal	65	
	Vertical + Z-Horizontal	65	
S-145 vertical	Operating-basis earthquake		1,500
	Vertical + Z-Horizontal	10	
	Vertical + Z-Horizontal	10	
	Safe shutdown earthquake		2,250
	Vertical + Z-Horizontal	20	
	Vertical + Z-Horizontal	20	
S-210 vertical	Operating-basis earthquake		8,500
	Vertical + Z-Horizontal	50	
	Vertical + Z-Horizontal	50	
	Safe shutdown earthquake		12,750
	Vertical + Z-Horizontal	50	
	Vertical + Z-Horizontal	50	
S-210 axial	Operating-basis earthquake		8,500
	Vertical + Z-Horizontal	65	
	Vertical + Z-Horizontal	65	
	Safe shutdown earthquake		12,750
	Vertical + Z-Horizontal	65	

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + Z-Horizontal	65	
S-225 vertical	Operating-basis earthquake		1,500
	Vertical + Z-Horizontal	10	
	Vertical + Z-Horizontal	10	
	Safe shutdown earthquake		2,250
	Vertical + Z-Horizontal	20	
	Vertical + Z-Horizontal	10	
N 404 horizontal (2 in.)			
	Operating-basis earthquake		375
	Vertical + Z-Horizontal	0	
	Vertical + Z-Horizontal	10	
	Safe shutdown earthquake		562
	Vertical + Z-Horizontal	10	
	Vertical + Z-Horizontal	10	
N 404 horizontal (3 in.)	Operating-basis earthquake		375
	Vertical + Z-Horizontal	40	
	Vertical + Z-Horizontal	40	
	Safe shutdown earthquake		562
	Vertical + Z-Horizontal	50	
	Vertical + Z-Horizontal	60	
N 405 vertical (2 in.)	Operating-basis earthquake		500

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + Z-Horizontal	90	
	Vertical + Z-Horizontal	90	
	Safe shutdown earthquake		750
	Vertical + Z-Horizontal	100	
	Vertical + Z-Horizontal	100	
N 405 horizontal (2 in.)	Operating-basis earthquake		150
	Vertical + Z-Horizontal	20	
	Vertical + Z-Horizontal	20	
	Safe shutdown earthquake		225
	Vertical + Z-Horizontal	30	
	Vertical + Z-Horizontal	30	
N 405 horizontal (3 in.)	Operating-basis earthquake		1,150
	Vertical + Z-Horizontal	210	
	Vertical + Z-Horizontal	210	
	Safe shutdown earthquake		1,725
	Vertical + Z-Horizontal	230	
	Vertical + Z-Horizontal	230	
N 405 horizontal (3 in.)	Operating-basis earthquake		400
	Vertical + Z-Horizontal	70	
	Vertical + Z-Horizontal	70	
	Safe shutdown earthquake		600
	Vertical + Z-Horizontal	80	

Supports	Description	As-Built Conditions (lb)	Design Load (lb)
	Vertical + Z-Horizontal	80	

a. Support load combination is seismic plus deadweight.

Table 3.9-27
LOADING COMBINATIONS AND STRESS LIMITS FOR SUPPORTS ON PIPING SYSTEMS

Loading Combination	**Stress Limits**
Normal	
D or $(D + F + T)$[a]	≤ Working Stress[b]
Upset	
$D \pm E$ or $(D + F + T \pm E)$[a]	≤ Working Stress[b]
Faulted	
$D \pm E'$ or $(D + F + T_o \pm E')$[a]	≤ Faulted Stress[c]
Deadweight and thermal are combined algebraically	
D = Deadweight T = Maximum operating thermal condition for system F = Friction load[d] E = OBE (inertia load + seismic differential support movement) E′ = SSE (inertia load + seismic differential support movement) T_o = Thermal - operating temperature	

a. For each loading condition, the greater of the two load combinations shall be used.

b. Working stress allowable per Appendix XVII of ASME Code, Section III.

c. Faulted stress allowable per Appendix XVII, Subsection NF, and Appendix F of ASME Code Section III, and Regulatory Guide 1.124. Safety Class 1 supports will be evaluated and designed in accordance with Regulatory Guide 1.124.

d. Whenever the thermal movement of the pipe causes the pipe to slide over any member of a support, friction shall be considered. The applied friction force applied to the support is the lesser of μ, W, or the force generated by displacing the support an amount equal to the pipe displacement.

   μ =0.35

   W =Normal load (excluding seismic) applied to the member on which the pipe slides.

Table 3.9-28
ANALYSIS OF TYPICAL PIPE SUPPORT BASE PLATES CALCULATED FOR IE BULLETIN 79-02

-	Existing Design				-	Replacement Design				-
-	Bolt Load		Bolt Capacity			Bolt Load		Bolt Capacity		
Support No.	Tension	Shear	Tension	Shear	Factor of Safety	Tension	Shear	Tension	Shear	Factor of Safety
ACH-106	75	0	7285	5760	97.0	75	0	14100	15195	188.0
ACH-118	241	293	7285	5760	11.9	241	293	14100	15195	27.5
SWAH-19	3161	1435	26880	26880	5.8	1452	975	14100	15195	6.0
SWAH-23	2963	1345	26880	26880	6.2	1257	897	14100	15195	6.8
SWAH-24	1972	895	26880	26880	9.4	837	597	14100	15195	10.1
SWCH-63	6	0	7285	5760	1121.0	7	0	11550	15195	1650.0
SWCH-73	18	0	7285	5760	399.0	19	0	11550	15195	608.0
SWCH-74	14	0	7285	5760	520.0	14	0	11550	15195	825.0
ACH-100	262	0	7285	5760	27.8	340	126	14100	15195	30.9
SWAH-37	499	220	7285	5760	9.4	455	250	14100	15195	20.5

Table 3.9-29
INTERNALS DEFLECTIONS UNDER ABNORMAL OPERATION

	Calculated Deflection (in.)	Allowable Limit (in.)	No loss of Function Limit (in.)
UPPER BARREL			
expansion/compression (to ensure sufficient inlet flow area / and to prevent the barrel from touching any guide tube to avoid disturbing the rod cluster control guide structure)	0.150	5	10
UPPER PACKAGE			
axial deflection (to maintain the control rod guide structure geometry)	0.005	1	2
ROD CLUSTER CONTROL GUIDE TUBE			
deflection as a beam (to be consistent with conditions under which ability to trip has been tested)	0.75	1.0	1.5
FUEL ASSEMBLY THIMBLES			
cross-section distortion (to avoid interference between the control rods and the guides)	0	0.035	0.072
Note: The values in this Table remains bounding for Extended Power Uprate (EPU).			

3.10 SEISMIC QUALIFICATION OF SEISMIC CATEGORY I INSTRUMENTATION AND ELECTRICAL EQUIPMENT

3.10.1 SEISMIC QUALIFICATION CRITERIA

3.10.1.1 Original Criteria

At the time that Ginna Station was designed and constructed, critical electrical equipment was required by specification to be capable of withstanding the maximum seismic loads postulated for the plant site. Most components in the Class 1E electric power distribution system were designed to withstand forces due to electrical faults, which were much larger than the inertial forces due to a severe seismic event.

In the original design of Ginna Station, no in-structure response spectra were developed for the analysis of equipment. Instead, Seismic Category I items were qualified on an individual and often generic basis. Table 3.10-1 provides a list of items and the basis of seismic qualification for Ginna electrical equipment.

Seismic design requirements for Seismic Category I instrumentation and controls were originally specified in equipment specifications as follows:

A. Control room - The racks were assembled and the mounting and wiring of all components were designed such that the functions of the circuits or equipment would be performed in accordance with prescribed limits when subjected to seismic accelerations of 0.21g in the horizontal direction and in the vertical direction simultaneously. In addition, the mounting and wiring of all components were done such that simultaneous accelerations of 0.52g in the horizontal and vertical planes would not dislodge, cause relative movement, or result in any loss or change of function of circuits of equipment.

B. Containment and auxiliary building - The mounting and wiring of all components were designed such that simultaneous accelerations of 0.52g in the horizontal and vertical planes would not dislodge, cause relative movement, or result in any loss or change of function of circuits or equipment.

3.10.1.2 Current Criteria

When making modifications at Ginna Station, RG&E requires seismic qualification in accordance with the current standard when possible. When major Class 1E components that are independently anchored to Seismic Category I structures are designed and procured, it is done in accordance with the current seismic standard. This has resulted in an evaluation of seismic qualification in Ginna electrical equipment to increasingly severe standards including IEEE 344-1975.

The Systematic Evaluation Program (SEP) seismic input for determining the seismic design adequacy of mechanical and electrical equipment and distribution systems were based on in-structure (floor) response spectra for the elevations at which the equipment is supported. The floor spectra used in this reassessment, which are based on Regulatory Guide 1.60 spectra, are given in Section 3.7 (*Reference 7*). For electrical equipment, a composite 7% equipment damping was used in the evaluation for the 0.2g safe shutdown earthquake. For cable trays,

the damping levels to be used in design depend greatly on the tray and support construction and the manner in which the cables are placed in the trays. Damping could be as high as 20% of critical damping. For structural evaluation, the stress criterion used was that the total stress must be less than or equal to the yield stress.

For the review of anchorage and support of safety-related electrical equipment in accordance with IE Bulletin 80-21, RG&E developed a program of inspection, analysis, testing, and modification, if necessary.

For the anchorage system of the electrical equipment, the required anchor load capacity as determined by the analysis phase, would be compared with the verified anchor load capacity for the anchor bolts associated with that component or assembly, as determined by the test and modification phase. If the verified anchor load capacity is found to be equal to or greater than the required anchor load capacity, then no modification would be required. However, if the verified anchor load capacity is found to be less than the required anchor load capacity for an electrical assembly, additional anchors would be added.

The analysis of each anchoring system to determine the minimum anchoring requirement to safely secure the equipment during a seismic event was to be performed using the following criteria and assumptions.

The static analysis described in Section 5.3 of IEEE 344-1975 was the basis for establishing shear and tensile stresses expected in the electrical equipment anchors being evaluated. Specifically, the seismic response of all floor-mounted equipment would be assumed to be the peak of the required response spectra for the equipment floor location, using damping values in accordance with Regulatory Guide 1.61, multiplied by a static coefficient of 1.5 to account for multifrequency and multimode responses. The inertial forces acting on the equipment center of mass would then be evaluated. A multianchor computer model would then be used to determine the shear and tensile stresses for all floor-mounted equipment. The stresses thus determined would establish the required anchor load capacity which would be compared to the verified anchor load capacity to establish anchor adequacy. Wall-mounted electrical equipment would be assumed to be rigid and the zero period acceleration values would be used to determine the seismic forces. The tensile and shear stresses would be calculated using the multianchor model.

3.10.2 SEISMIC QUALIFICATION OF ELECTRICAL EQUIPMENT AND INSTRUMENTATION

3.10.2.1 Introduction

The SEP Seismic Review Team selected electrical equipment representative of items installed in the reactor coolant system and safe shutdown systems at Ginna Station and evaluated them for structural integrity and electrical and mechanical functional operability. Electrical components that potentially have a high degree of seismic fragility were identified for review during a site visit by members of the team. A representative sample of components was selected for review by one of two methods:

A. Selection based on a walk-through inspection of Ginna Station by the SEP Seismic Review Team. Based on their experience, team members selected components as to the potential degree of seismic fragility for the component's category. Particular attention was paid to the component's support structure.
B. Categorization of the safe shutdown components into generic groups such as motor control centers and motors.

Rochester Gas and Electric provided seismic qualification data on the selected components from each group. Table 3.10-2 lists five components selected for review and includes the reasons for their selection. The details of the analyses and conclusions reached regarding the adequacy of these components is described in Sections 3.10.2.2 through 3.10.2.6.

3.10.2.2 Battery Racks

These racks were manufactured by Gould-National Battery Inc. The racks are seismically qualified in accordance with IEEE standard 344-1975 and RG&E site specific response spectra for floor elevation 253'-0". Rack design incorporates minimum cell spacing requirements imposed by the manufacturer.

3.10.2.3 Motor Control Centers 1L and 1M

A previous computer analysis was made of a Westinghouse type W ac motor control center which was originally tested at Wyle Laboratories in October 1972 to meet the seismic requirements recommended by IEEE Standard 344-1971. The calculations determined the acceleration levels and type of motion response that were excited in the equipment by a simultaneous horizontal and vertical sine beat type of motion input (5 cycles/beat). Subsequently, a similar dynamic analysis was made of the equipment as modified for Ginna, with attention focused on the new panelboard and distribution transformers.

The original Ginna response spectra, as specified for the safe shutdown earthquake condition, gave a total rms vector input acceleration of 0.79g calculated as 0.56 times the square root of the sum of the squares value of the following three components:

x-direction (front to rear) = 0.707 x 0.56g = 0.4g

y-direction (side to side) = 0.707 x 0.56g = 0.4g

z-direction (vertical) = 1.0 x 0.56g = 0.56g

The value of 0.56g was specified for the Ginna test. The Wyle Laboratories response spectra, on the other hand, gave a total rms vector input acceleration of 1.49g.

The response spectra at the auxiliary building platform and operating floor centers of gravity were compared to the Wyle Laboratories spectrum. Above 5 Hz, the acceleration levels throughout the equipment were greater when calculated for the 5 cycles/beat test at the 8.5 Hz fundamental natural frequency, compared to an envelope of the Ginna in-structure response spectra.

Based on review of the test results and comparison of input response spectra, as well as corresponding acceleration levels sustained in the equipment, it was concluded that the

existing fragility level tests performed at Wyle Laboratories could be used to qualify the Ginna motor control centers, which have fundamental frequencies above 5 Hz.

3.10.2.4 Switchgear

The previous seismic qualification of Westinghouse type DB-50 reactor trip switchgear for Ginna was performed at the Westinghouse Astronuclear Laboratory. The reports present results of seismic simulation testing for the "low seismic" (safe shutdown earthquake peak acceleration not exceeding 0.2g) and "high seismic" (safe shutdown earthquake between 0.2g and 0.4g) classes of plants over the frequency range 1 to 35 Hz. The simulated seismic tests consisted of three elements:

A. Inputting a sine beat type acceleration to the base of the equipment being tested.

B. Monitoring the resulting accelerations at various locations in the equipment.

C. Monitoring the electrical functions of the equipment both during and after the tests to check for any loss of function.

Each sine beat of the vibration input consisted of 10 cycles of the test frequency with the amplitude of the beat (i.e., the acceleration of the vibration) increasing from a small value to the specified maximum value and returning to the initial value in sine wave fashion. The maximum required vertical input acceleration of the sine beat, as a function of test frequency for the "low seismic" plant classification, was 0.5g up to 10 Hz and reduced to a minimum value of 0.2g at 25 Hz. For horizontal excitation, the maximum required acceleration level of the sine beat was 0.8g up to 10 Hz and reduced to a minimum value of 0.2g at 25 Hz. Corresponding values for the "high seismic" plant classification were 0.93g up to 10 Hz, reducing to 0.32g at 25 Hz for vertical excitation and 1.4g up to 10 Hz, reducing to 0.5g at 25 Hz for horizontal excitation.

The applicable SEP reassessment response spectra for the switchgear were higher than both the "low seismic" and "high seismic" horizontal acceleration input curves for frequencies between 15 and 30 Hz. Based on the review of the tests performed at the Westinghouse Astronuclear Laboratory, it was concluded that the Westinghouse type DB-50 reactor trip switchgear would maintain its electrical function during a safe shutdown earthquake event. This conclusion was based on the assumption that there were no resonant frequencies in the 15 to 30 Hz range, or, if such resonances existed, that the response spectra developed from the sine beat test at the resonant frequency for 7% of critical damping enveloped the Ginna spectra (*Reference 1*).

3.10.2.5 Control Room Electrical Panels

The structural integrity of the main control board was evaluated for seismic loads for the safe shutdown earthquake as part of the SEP review (*Reference 3*). The seismic stresses were calculated using the modal response properties of the main control board determined by in-situ modal testing. A response spectrum analysis was used to calculate the seismic inertial load in each significant mode for three mutually perpendicular directions of eathquake motion. The inertial loads were then used in a static analysis to determine forces, moments, and stresses in critical elements of the seismic load path of the main control board. The

results of the analysis indicated that the main control board would survive the safe shutdown earthquake. However, RG&E decided to provide some additional stiffeners and supports in order to enhance the structural integrity of the control board. These modifications were implemented in 1984.

3.10.2.6 Electrical Cable Raceways

The cable tray and conduit support anchors were installed using the manufacturers recommended procedures. As a result of SEP seismic review, a comprehensive testing and analysis program to demonstrate the seismic adequacy of electrical cable trays and conduit raceways of the type used in SEP plants was initiated by the SEP Owners Group. By letter of October 15, 1984, from R. M. Kacich, Chairman of the SEP Owners Group, to C. I. Grimes of the NRC (*Reference 4*), the SEP Owners Group responded to concerns relative to the seismic capability of cable trays as follows:

> The overall conclusion of the SEP cable tray test and evaluation program indicates that it is highly unlikely that any of the cable tray systems used in SEP plants will suffer structural collapse during a safe shutdown earthquake of the magnitude specified for eastern SEP plants. This conclusion is based on the fact that no system failures occurred in any of over 200 full-scale shake table tests of cable tray configurations selected, based on detailed plant walk-downs, as being typical of those in SEP plants. This conclusion is also supported by actual earthquake experience data from power plants and industrial facilities that have experienced strong motion earthquakes.
>
> Based on the results of the Owners Group efforts to date, it is concluded that the existing raceway systems in SEP plants possess substantial inherent seismic resistance and that the seismic qualification of raceway systems is not a significant safety issue. Therefore, no further work on this issue by the SEP owners is planned.
>
> As noted above, world-wide experience in power plants which have undergone significant earthquakes strongly supports the conclusion of the test and evaluation program. These experience data are expected to be documented as part of the ongoing efforts of the Seismic Qualification Utilities Group.

3.10.2.7 Constant Voltage Transformers

The constant voltage transformers are located in the battery rooms of the control building at elevation 253.7 ft. The constant voltage transformers are seismically qualified in accordance with IEEE Standard 344-1975 and RG&E site-specific response spectra for floor elevation 253.7 ft. Mounting requirements have been analyzed to this response spectra.

3.10.3 SEISMIC QUALIFICATION OF SUPPORTS OF ELECTRICAL EQUIPMENT AND INSTRUMENTATION

The SEP Seismic Review Team recommended that all safety-related equipment at Ginna Station be checked for adequately engineered anchorage; that is, the anchorage should be found to be adequate on the basis of analysis or tests employing design procedures (load stress and deformation limits, materials fabrication procedures, and quality acceptance) in accordance with a recognized structural design code.

Rochester Gas and Electric Corporation initiated a three-phase Seismic Action Plan (*Reference 5*) to provide assurance that the electrical equipment anchorage systems will perform their design function during the safe shutdown earthquake. Phase I consisted of inspection and preparation of as-built sketches for all safety-related electrical equipment as listed below. Anchor bolts used on this equipment were field inspected. As-built sketches were prepared showing all necessary information to perform Phase II. Phase II consisted of an analysis of each electrical equipment anchoring system, the results of which were compared to the test information. Phase III consisted of testing the anchor bolts and performing any resulting modifications required to upgrade the existing anchoring system to the criteria described in the analysis section of Phase II.

3.10.3.1 Equipment Addressed

The action plan included all Class 1E electrical systems and components. Certain Class 1E equipment installed during recent modifications in accordance with IEEE 344-1975 requirements was known to be seismically anchored and was not considered in the study.

The following electrical assemblies and/or components were evaluated by the Seismic Action Plan:

- Relay rack assemblies.
- 480-V 1E buses.
- 480-V (ac) 1E motor control center.
- 125-V (dc) 1E starters.
- Power panels.
- 1E battery racks.
- 1E battery chargers.
- Instrument racks.
- Control panels.
- Diesel-generator panels.
- Non-1E items (ancillary items).

All internally mounted components and devices weighing more than 25 lb were analyzed as separate assemblies. The results of the seismic evaluation program are described in *References 6* and *7*. The details are summarized in Section 3.10.3.2.

3.10.3.2 Raceway Anchorages

<u>3.10.3.2.1 Test Program</u>

All trays and conduit runs in the safety-related buildings had their anchorage systems inspected, tested, and, if required, reworked. No attempt was made to distinguish between Class 1E and non-1E raceways in any of the Seismic Category I structures.

Test criteria were established including the information necessary to test the anchorage of the supports making up the raceway system. Specific test procedures were prepared, consistent with the test criteria, for each category of anchorage included in the program. The categories of anchorages were

A. Expansion anchors for both conduit and tray supports in ceiling and/or wall locations.

B. Clips and unistrut hardware that rely on frictional resistance.

C. Embedded hardware such as keystone Q deck nuts, embedded unistrut, and poured-in-place anchors.

Detailed sketches of each of the embedded hardware type anchors are shown in Figures 3.10-1, 3.10-2, and 3.10-3.

The test program included all the hardware comprising the load path for each specific type of support. The bolts suspending the strut members to the ceiling or wall section were tested on a generic basis if they were the embedded hardware type and sample tested if they were shell anchors. The hardware used to attach the strut members to the anchor bolts and which rely on friction was also tested. Figure 3.10-4 shows the various generic strut support configurations in use at Ginna Station that were part of the friction bolt testing program.

3.10.3.2.2 Test Loads

In order to establish test load per bolt requirements for the shell anchors and embedded anchors, the original plant specification for cable trays was consulted. Section 4 of Specification SP-5375, (*Reference 8*), specifies the design load for the cable tray type as 100 lb/ft. This load, applied to any of the specified cable tray widths, should produce no more than 0.25 in. deflection at midspan when calculated on a simple beam basis. In addition to the tray loads, the supports were designed to carry a 200-lb person standing at any position in the tray. The design span lengths were assumed to be 8 ft. The 8-ft span lengths carry a total load of 800 lb between supports or 4000 lb for a stack of five trays. Two vertical members were assumed per support. A 2000-lb test load was used on each vertical support member to test the anchorages.

The test load for the frictional anchors was based on the manufacturer's design manual, Unistrut General Engineering Catalog No. 9 (*Reference 9*). The design torque values for various bolt sizes needed to maintain a resistance to slippage of at least 1500 lb for a 1/2-in. bolt used on P1000 strut were determined to be as follows:

1.

	Bolt Size			
	1/4 in.	5/16 in.	3/8 in.	1/2 in.
Torque (ft-lb)	6	11	19	50

The torque values shown above were used in the test procedures for qualifying the unistrut stud/nut hardware assemblies and includes a minimum safety factor of 3.

3.10.3.2.3 Expansion Anchor Test Results

Expansion anchors were selected for testing by inspecting and testing 25% of the cable tray vertical support members using shell type anchors and 10% of the rigid conduit supports using shell anchors. The lower sampling rate for conduit was used since all Class 1E conduit is rigid and has a very low design load. However, the 2000-lb test load was used on conduit anchors. All expansion anchors were tested on each of the sample supports.

The selected anchors were inspected and load tested to 2000 lb in accordance with RG&E Ginna Station Procedures. The acceptance criteria is that the shell anchors hold the required load without excessive movement.

The results of the shell anchor testing program are summarized in Table 3.10-3.

3.10.3.2.4 Frictional Anchor Test Results

The unistrut stud/nut testing criteria (frictional anchors) used were as follows:

A. All accessible unistrut stud nuts used for cable tray supports were tested. The total number of Class 1E supports is shown in Table 3.10-4.

B. The unistrut nuts/bolts that were tested were those used to attach the strut members to the ceiling Q deck bolts or angle clips. These attachments rely on friction and must be torqued to at least a minimum value which was established to ensure a safety factor of at least 3. Figure 3.10-4 shows the various configurations of strut supports used throughout Ginna Station. The unistrut joints affected by the procedures are marked by an arrow.

C. The "as-found" torque of all the unistrut stud nuts on a particular support was recorded. All inaccessible bolts were identified and recorded. Torque wrench adapters (i.e., crow's foot) were used to reduce the number of inaccessible nuts or bolts. Those bolts still inaccessible were wrench-tightened where possible.

D. The design torque values for the various bolt sizes were derived from the following manufacturer's data:

	Bolt Size			
	1/4 in.	5/16 in.	3/8 in.	1/2 in.
Torque (ft-lb)	6	11	19	50

If the "as found" torque values were less than the minimum values specified by the manufacturer then the proper torque values were applied to each bolt. Both the as-found and final torque values were recorded.

All accessible supports were tested. The results of the friction bolt testing program are summarized in Table 3.10-4.

3.10.3.2.5 Embedded Anchor Test Results

The keystone steel decking test criteria (embedded hardware anchors including embedded unistrut and poured-in-place anchors) were developed and the following generic test was performed to ensure that the load capacity of the Q deck was sufficient to sustain the required loads. Fourteen in-situ tests were performed at different plant locations. These locations were in convenient open areas and not in an actual support location. Ten in-situ unistrut and 12 poured-in-place anchor tests were also completed.

The results of the embedded anchor programs are summarized in Table 3.10-5.

3.10.3.3 Class 1E Equipment Anchorage Qualification Program

As-built drawings were prepared for 115 electrical assemblies. These drawings represent all Class 1E and non-1E equipment which are floor-mounted, mounted on structural steel, poured wall mounted or block wall mounted. Each drawing lists the size, shape, number, and type of existing anchor bolts for a particular assembly. This information was obtained from field measurements.

The weights were assessed based on the area, gauge size of the enclosure steel, and the weights of all the internally mounted components, including wire and terminal blocks. The total equipment weights were then determined including 25% of the enclosure weight for conservatism.

The minimum loading that the existing anchorage must be capable of carrying during a seismic event (safe shutdown earthquake) at Ginna Station was determined during this program. The calculated loads (tensile and shear) were compared to the published load capabilities for the specific anchors used on each assembly. If the calculated load values were within the published capability of the bolts used on a particular assembly, then the calculated loads were used as the test loads for that assembly, provided the bolts were accessible. For wall-mounted equipment that had safety factors in excess of 10, no modification or testing was performed. If it was determined that the existing anchorages were inadequate, then those assemblies were modified taking no credit for the existing anchors.

The horizontal and vertical forces were determined by using one-and-a-half times the peak acceleration shown on the floor response spectrum for each assembly location. All proposed expansion anchor bolts used a minimum safety factor of 5.7 in tension and 4 in shear.

The final phase of the program involved the installation of generic modifications using specific construction drawings for each assembly to be modified. A typical generic modification included the welding of structural plates or angles to the outside of the enclosure frame, the installation of hilti bolts or through bolts depending on location, and the stitch welding of the enclosure cabinets to the frames.

Non-class 1E evaluations were conducted for those assemblies permanently mounted in Seismic Category I buildings that are not safety-related. The anchorage acceptance criteria for those assemblies were the same as for the Class 1E assemblies.

Internally mounted components were categorized and a generic design analysis was developed to evaluate the methods of attaching these components to the cabinets. If any one component is classified Class 1E in an enclosure, then all components were assumed to be Class 1E.

Non-class 1E enclosures were not surveyed. It was assumed that the enclosure will retain any loose component during a safe shutdown earthquake.

3.10.3.4 Conclusions

The NRC has reviewed the RG&E report of the upgrading of anchorage and support of safety-related electrical equipment (*Reference 6*) and concluded that the electrical equipment anchorage design and internal mounted devices and component evaluations and modifications were adequate (*Reference 2*). The required modifications have been completed as designed.

3.10.4 FUNCTIONAL CAPABILITY OF COMPONENTS

The NRC initiated a generic program to develop criteria for the seismic qualification of equipment in operating plants as an Unresolved Safety Issue (USI A-46). Under this program, an explicit set of guidelines (or criteria) to be used to judge the adequacy of the seismic qualifications (both functional capability and structural integrity) of safety-related mechanical and electrical equipment at all operating plants was developed.

The NRC Staff as a result of the seismic review of the R. E. Ginna Nuclear Power Plant has concluded that, since the ground response spectrum (0.2g Regulatory Guide 1.60 spectrum) used for Ginna seismic reevaluation envelops the Ginna site-specific ground response spectrum, additional safety margins in the structures, systems, and components do exist for resisting seismic loadings. The staff also concluded that Ginna Station has an adequate seismic capacity to resist a postulated safe shutdown earthquake, and there is reasonable assurance that the operation of the facility will not endanger the health and safety of the public. (*Reference 2*).

RG&E submitted the Ginna Station response to USI A-46 in January of 1997 (*Reference 13*). In June of 1999 the NRC issued a Safety Evaluation Report (SER) accepting RG&E's analysis and modifications (*Reference 16*).

3.10.5 SEISMIC CATEGORY I TUBING

3.10.5.1 Codes and Standards

The original design of Seismic Category I tubing and tubing supports at Ginna Station was performed to then current (1967) standard industry practice, which was based on the experience of the journeyman instrument installer and did not require conformance to specific industry codes or standards.

Current (1988) design requirements for Seismic Category I tubing and supports include the following:

3.10.5.1.1 Tubing Design Requirements

Instrument Standard of America Standard ISA-S67.02 and Regulatory Guide 1.151 (*References 10* and *11*) are used as guidance for the design, fabrication, installation, and testing of tubing.

Tubing is designed using the stress evaluation equations contained in ANSI B31.1 (1973) with allowable stress limits as included in Table 3.10-6 except that the stress intensification factor, I, applicable to bending moments is taken equal to 1.3 for all joint and fitting configurations because of the relatively low allowable stress permitted by Table 3.10-6 compared to ASME Section III allowables.

Welder qualifications, welding, and examination procedures are in accordance with:
ASME Sections III, V, VIII and XI code; 2004 Edition with no Addenda.
ASME Section IX code; current Edition and Addenda.
ASME Section XI code; 2004 Edition with no Addenda for IWE Containment (metallic liner).
ANSI/ASME B31.1 Power piping; 2004 Edition with no Addenda.

The loads and load causing phenomena to be considered in the qualification and design of tubing shall include the following.

- Dead weight.
- Pressure.
- Temperature.
- Seismic inertia.
- Support motions due to
    1. Thermal.
    2. Seismic.

3.10.5.1.2 Tubing Supports Design Requirements

Tubing supports are standard manufactured tubing supports (clips or clamps) plus any auxiliary steel used to protect tubing (channels) and provide a support path to the building structure. Tubing supports that attach the tubing to auxiliary or building steel shall be standard manufactured tubing supports qualified for their intended use by load rating using the procedure contained in ASME Code Section III-NF-3380, Design by Load Rating, 1986 edition.

Channels or other structural steel used to protect and support tubing and other auxiliary steel used in the tubing support path to the building structure shall be designed to the AISC specification given in *Reference 12* for the limiting loads developed from the spacing tables and charts or as otherwise calculated for individual tubing runs evaluated by analysis. The particular loads and load-causing phenomena used to design supports are the same as given above for tubing, except for pressure. Allowable stresses for the load combinations identified are given in *Reference 12*.

Tubing spans in space, in those areas adjacent to normal personnel access (i.e., within 7 ft 0 in. height of platforms, floor walkway areas, etc.), over 3 ft 0 in. in length, shall be contained in channels or similarly supported or protected against potential damage.

3.10.5.2 Load Conditions

3.10.5.2.1 Tubing

The tubing shall be analyzed for the following loading conditions:

A. Design condition - deadweight plus design pressure.

B. Severe environmental condition$_{(1)}$ - deadweight plus operating pressure plus OBE (inertia).

C. Severe environmental condition$_{(2)}$ - deadweight plus operating pressure plus OBE (inertia) plus OBE (SAM) displacements plus maximum operating thermal effects including thermal support motions.

D. Extreme environmental condition - deadweight plus operating pressure plus SSE (inertia).

E. Abnormal condition - deadweight plus operating pressure plus loss-of-coolant-accident induced thermal effects (application limited to inside containment).

3.10.5.2.2 Tubing Supports

The tubing system supports will be evaluated to the following combinations of tubing system imposed loads:

A. Severe environmental condition$_{(1)}$ (Equation 4 of Table Q1.5.7.1 of *Reference 12*):

   Deadweight plus OBE (inertia).

B. Severe environmental condition$_{(2)}$ (Equation 6 of Table Q1.5.7.1 of *Reference 12*):

   Deadweight plus maximum operating thermal including restraint of free end displacement and thermal support motions plus OBE (inertia) and (seismic anchor motion) effects.

C. Extreme environmental condition (stress limit coefficient from Table Q1.5.7.1 is 1.6, Equation 8 of *Reference 12*):

   Deadweight plus SSE (inertia).

D. Abnormal (stress limit coefficient from Table Q1.5.7.1 is 1.7, Equation 11 of *Reference 12*) (application limited to inside containment):

   Deadweight plus maximum accident thermal including restraint of free end displacement and thermal support motions.

Included in the design of horizontally run channels provided to protect or support tubing runs defined as deadweight shall be a requirement to support an external vertical load of 50 lb, to protect the tubing during construction and normal plant maintenance, placed to cause the highest bending and shear stresses in the channel.

3.10.5.3 Routing Requirements

Instrument sensing lines shall be routed to prevent violating required separation between redundant instrument channels. Separation between redundant instrument sensing lines shall be provided by free air space or barriers, or both, such that no single failure can cause the failure of more than one redundant sensing line.

The minimum separation between redundant instrument sensing lines shall be at least 18 in. in air, in nonmissile, non-high-energy jet stream, non-pipe-whip or nonhostile areas. As an alternative, a suitable barrier shall be used, which extends at least 1 in. beyond the line of sight between redundant sensing lines and shall be designed and mounted to Seismic Category I requirements. In hostile areas potentially subject to high-energy jet stream, missiles, and pipe whip, the separation shall be provided by space in air, steel or concrete barriers, or both, and documented with analyses or calculations as necessary to prove that the separation protects the redundant sensing lines from failure due to a common cause. All barriers shall be designed and mounted to Seismic Category I requirements.

Instrument sensing lines shall be run along walls, columns, or ceilings whenever practical, avoiding persons supporting themselves on the lines or damage of the sensing lines by pipe whip, missiles, jet forces, or falling objects.

Supports, brackets, clips, or hangers shall not be fastened to the instrument sensing lines for the purposes of supporting cable trays or any other equipment.

Routing of the nuclear-safety-related instrument sensing lines shall ensure that the function of the lines is not affected by vibration, abnormal heat, or stress.

REFERENCES FOR SECTION 3.10

1. R. C. Murray, et al., Seismic Review of the Robert E. Ginna Nuclear Power Plant as Part of the Systematic Evaluation Program, NUREG/CR-1821, November 15, 1980.

2. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Safety Topics III-6, Seismic Design Consideration and III-11, Component Integrity, dated January 29, 1982.

3. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic II-6, Seismic Considerations (Seismic Structural Evaluation of the Main Control Board), dated January 9, 1984.

4. Letter from R. M. Kacich, SEP Owners Group, to C. I. Grimes, NRC, Subject: SEP Topic III-6, Seismic Design Considerations, SEP Owners Group Cable Tray/Conduit Test Program, dated October 15, 1984.

5. Letter from L. D. White, Jr., to D. L. Ziemann, NRC, Subject: The Seismic Action Plan, Anchorage and Support of Safety-Related Electrical Equipment, dated February 11, 1980.

6. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Anchorage and Seismic Support of Safety-Related Electrical Equipment, Final Report, dated December 22, 1980.

7. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Anchorage and Support of Safety-Related Electrical Equipment, Final Report, dated February 27, 1981.

8. Gilbert Associates, Inc., Cable Trays and Electrical Circuits Power, Control and Instrumentation, Ginna Station Unit No. 1, Technical Specification SP-5375, dated March 17, 1967.

9. Unistrut General Engineering Catalog No. 9, Unistrut Corporation, Wayne, Michigan.

10. Instrument Society of America, Nuclear Safety-Related Instrument Sensing Line Piping and Tubing Standard - 1980 for Use in Nuclear Power Plants, ISA-S67.02, 1983.

11. U.S. Nuclear Regulatory Commission, Instrumentation Sensing Lines, Regulatory Guide 1.151, July 1983.

12. American Institute of Steel Construction, Nuclear Facilities - Steel Safety-Related Structures for Design, Fabrication, and Erection, Specification ANSI/AISC N690, 1984.

13. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Resolution of Generic Letter 87-02 Supplement 1 and 88-20 Supplements 4 and 5, dated January 31, 1997.

14. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to NRC "RAI" on USI A-46, May 27, 1998.

15. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to NRC second "RAI" on USI A-46, dated February 2, 1999.

16. Letter to R. C. Mecredy, RG&E, from G. S. Vissing, NRC, Subject: Plant Specific Safety Evaluation Report for USI A-46, dated June 17, 1999.

Table 3.10-1
MAJOR CLASS 1E COMPONENTS AND THE BASIS FOR SEISMIC QUALIFICATION

		System/Component	Basis for Seismic Qualification
I.		**EMERGENCY POWER SYSTEM**	
	A.	Low voltage (600-V) switchgear (excluding unit transformer) (Westinghouse DB 15, 25, 50, and 75 breakers)	Post-construction testing.
	B.	Motor control centers (Westinghouse type W)	Post construction testing and analysis in accordance with IEEE 344-1971. Upgraded by analysis to IEEE 344-1975.
	C.	Motor-operated valve operators (ac/dc)	Post-construction testing.
	D.	Vital 120-V ac Distribution panels 1A and 1C Inverters (Solidstate Controls, Inc.) Constant voltage transformers (CVT)	Postconstruction testing. Installed in 1978 qualified by test in accordance with IEEE 344-1975. CVTs qualified to IEEE 344-1975.
	E.	125-V dc power system 125-V, 60-cell batteries (Gould) and racks Battery chargers	Design specification; 0.52g simultaneous horizontal and vertical. Racks qualified to IEEE 344-1975. Battery cells qualified to IEEE 344-1987.
	F.	Diesel generators (Alco/Westinghouse)	Design specification; 0.47g simultaneous horizontal and vertical acceleration.
	G.	Reactor building cable penetrations (Crouse-Hinds)	Postconstruction testing.
	H.	Conduit supports and tray supports	SEP Owners Group.
	I.	Electrical equipment anchors	Modification program.
II.		**SAFEGUARDS INSTRUMENTATION AND CONTROL**	
	A.	Transmitters (Barton, Foxboro)	Post-construction testing.
	B.	Reactor trip switchgear (DB 50)	Post-construction testing.
	C.	Main control board (Wolf and Mann)	Design specification; 0.52g simultaneous horizontal and vertical acceleration.
	D.	Reactor trip system racks (A/D conversion)	Design specification; 0.52g simultaneous horizontal and vertical acceleration. Modification to racks.

	System/Component	Basis for Seismic Qualification
E.	Protective relay racks (safety injection and reactor trip logic)	Design specification; 0.52g simultaneous horizontal and vertical acceleration.
F.	Safeguards racks (engineered safety features actuation (ESFAS output)	Design specification; 0.52g simultaneous horizontal and vertical acceleration.
G.	Control switches (Westinghouse type W2 and OT2)	Post-construction testing.

Table 3.10-2
ELECTRICAL COMPONENTS SELECTED FOR SEISMIC REVIEW

Item Description	Reason for Selection
Battery racks	Evaluate capacity of the bracing to develop lateral load capacity.
Motor control centers	Typical seismically qualified electrical equipment. Functional design adequacy may not have been demonstrated. Check anchorage to floor structure.
Switchgear	Same as motor control centers.
Control room electrical panels	The control panels appear to be adequately anchored at the base. However, there is a need to check components which are cantilevered off of the front panel and to check front panel stiffness.
Electrical cable raceways	The cable tray support systems did not have any specific seismic qualification testing.

Table 3.10-3
SHELL ANCHOR TEST SUMMARY

Location	Total Number of Anchors	Number of Anchors That Held Load	Number of Anchors That Did Not Hold Load	Inaccessible
Auxiliary building basement floor	11	11	0	0
Auxiliary building intermediate floor	16	16	0	0
Screen house basement floor	9	9	0	0
Cable tunnel ceiling	5	5	0	0
Containment building basement	2	2	0	0
Relay room	6	5	0	1
Battery rooms	4	4	0	0
Diesel-generator pits	22	21	1	0
Total	75	73	1	1

Table 3.10-4
FRICTION BOLT TEST RESULT SUMMARY

Location	Total Number of Bolts	Acceptable Torque	Bolts Wrench Tightened	Bolts Not Accessible
Auxiliary building basement floor	227	217	1	9
Auxiliary building intermediate floor	202	133	17	52
Intermediate building, elevation 271 ft 0 in	28	14	2	12
Intermediate building, elevation 278 ft 4 in	320	305	11	4
Screen house basement floor	144	142	2	0
Cable tunnel	649	532	15	102
Relay room	361	315	1	45
Battery rooms	215	213	0	2
Diesel-generator pits	84	84	0	0
Containment basement floor	112	112	0	0
Containment intermediate floor	338	337	0	1
Total	2680	2404	49	227

Table 3.10-5
CATEGORY 3 ANCHORS TEST SUMMARY

Location	Number of Poured-In-Place Tested	Unistrut Tests	Q-Deck Tests	Total Tests	Held Load	Did Not Hold Load
Auxiliary building basement floor	0	2	0	2	2	0
Auxiliary building intermediate floor	0	2	0	2	2	0
Intermediate building, elevation 271 ft 0 in	0	0	2	2	2	0
Screen house basement floor	0	2	0	2	2	0
Containment basement floor	0	2	2	4	4	0
Containment intermediate floor	12	2	2	4	4	0
Relay room	0	0	2	2	2	0
Battery rooms	0	0	6	6	6	0
Total	12	10	14	24	24	0

Table 3.10-6
STRESS LIMITS FOR TUBING

Condition	Stress Limits
Design	$P_m + P_b \leq S_h$
Severe environmental $_1$	$P_m + P_b \leq 1.2\ S_h$
Severe environmental $_2$	$P_m + P_b + P_e + P_{SAM} \leq (S_h + S_A)$
Extreme environmental	$P_m + P_b \leq 1.8\ S_h$
Abnormal [a]	$P_m + P_b + P_e + P_{AAM} \leq$ the stress limit for system operability

Where

P_m =	Primary general membrane stress; $P\ D_o / 4\ t_n$
P_b =	Primary bending stress; M_i / Z and M_T / Z
$S_{A'}$, $S_{h'}$, S_e =	Allowable stress from ANSI B31.1 Code for material at design temperature
P_e =	Restraint of free end displacement (thermal and differential support motion stress)
P_{SAM} =	Stresses due to differential OBE seismic support motions
P_{AAM} =	Stress due to accident-induced support motions
M_T =	Torsional moment on pipe
M_i =	Bending moment on pipe

a. Application limited to inside containment.

3.11 ENVIRONMENTAL DESIGN OF MECHANICAL AND ELECTRICAL EQUIPMENT

3.11.1 BACKGROUND

3.11.1.1 Initial Design Considerations

Section 6.1.2 discusses environmental considerations in the selection of engineered safety features materials. Sections 6.2.2.1, 6.3.2.1, and 6.5.1.2 discuss environmental protection design features for components of the containment ventilation (containment recirculation fan cooler), emergency core cooling, and containment air filtration systems located inside containment.

3.11.1.2 Review of Environmental Qualification of Safety-Related Electrical Equipment

The review of the environmental qualification of safety-related electrical equipment for Ginna Station was initiated in 1977 under Topic III-12 of the Systematic Evaluation Program (SEP). In February 1980, the NRC redirected the review program for SEP plants and provided Division of Operating Reactors (DOR) guidelines for evaluating environmental qualification and for identifying safety-related equipment for which environmental qualification was to be addressed (*Reference 1*). On June 25, 1982, the NRC issued an interim regulation (*Reference 2*), which suspended the June 30, 1982, deadline for qualification of electrical equipment pursuant to the DOR Guidelines and NUREG 0588. Subsequently, 10 CFR 50.49 was issued (February 22, 1983).

Ginna Station submitted the initial report concerning the environmental qualification of electrical equipment by letter, dated February 24, 1978 (*Reference 3*). This submittal was reformatted and resubmitted on December 1, 1978 (*Reference 4*). It was revised and resubmitted again on April 25, 1980 (*Reference 5*), and on October 31, 1980 (*Reference 6*). On June 1, 1981, the NRC issued its Safety Evaluation Report (SER) for the Environmental Qualifications of Safety-Related Electrical Equipment at the R. E. Ginna Nuclear Power Plant (*Reference 7*). The letter included the SER by the Office of Nuclear Reactor Regulation (NRR), the Draft Interim Technical Evaluation Report (TER C5257-178) by the NRC Consultant, Franklin Research Center, and a request that Ginna Station provide additional information. Ginna Station responded to the June 6, 1981 SER by letters dated September 4, 1981 (*Reference 8*),
November 6, 1981 (*Reference 9*), and February 18, 1982 (*Reference 10*). The NRC transmitted an SER by the NRR, and a Technical Evaluation Report by Franklin Research Center, TER C5257-454, on December 13, 1982 (*Reference 11*), based on RG&E responses in *References 8*, *9*, and *10*. Rochester Gas and Electric Corporation provided additional information in *References 12*, *13*, *14*, and *15*. In the responses (*Reference 16*) to NRC Generic Letter 84-24, RG&E certified program compliance with 10 CFR 50.49. It was also noted that the Environmental Qualification Program is not adversely impacted by the IE bulletins and notices listed in Generic Letter 84-24. In *Reference 17*, the NRC concluded that the Environmental Qualification Program complies with the 10 CFR 50.49 and that the issues raised in *Reference 11* are satisfactorily resolved.

Based on the DOR guidelines, the Ginna Station Environmental Qualification Program addresses the safety-related electrical equipment which must function to mitigate the

consequences of loss-of-coolant accidents (LOCA) or high-energy line breaks inside or outside containment and whose environment would be adversely affected by the accident.

3.11.2 EQUIPMENT IDENTIFICATION

In accordance with the DOR guidelines, Ginna Station was directed to establish a list of systems and display instrumentation needed to mitigate the consequences of a LOCA or high-energy line break inside or outside containment and to reach a safe shutdown. The display instrumentation selected includes parameters to monitor overall plant performance as well as to monitor the systems on the list. The list of systems was established on the basis of the functions that must be performed for mitigation of the consequences of a LOCA or high-energy line break and to effect safe shutdown without regard to the location of the equipment relative to a potentially hostile environment. The systems considered were those required to achieve or support (1) emergency reactor shutdown, (2) containment isolation, (3) reactor core cooling, (4) containment heat removal, (5) core residual heat removal, and (6) prevention of significant releases of radioactive material to the environment. The list of equipment requiring environmental qualification is included in the Ginna Station October 31, 1980, report (*Reference 6*), as supplemented in *References 8 through 10* and *12 through 14*. The current "Master List" relative to 10 CFR 50.49 is contained in a plant procedure.

3.11.3 IDENTIFICATION OF LIMITING ENVIRONMENTAL CONDITIONS

This section defines the bases for and references to the environmental conditions encountered throughout the plant. A tabular summary is provided in Table 3.11-1.

3.11.3.1 Inside Containment

<u>3.11.3.1.1 Post Loss-of-Coolant Accident Environment</u>

Postaccident environmental conditions inside containment are discussed in Section 6.1.2.1. The limiting conditions resulted from LOCA analyses. The temperature and pressure profiles are given in Figures 6.1-1 and 6.1-2 with peak values being 286°F and 60 psig, respectively. The radiation environment for Ginna Station is presented in Figures 6.1-4 and 6.1-5 from data in Tables 3.11-2 and 3.11-3. Material compatibility with postaccident chemical environment is also discussed in detail in Section 6.1.2.1. For a LOCA, containment conditions were analyzed as part of SEP Topic VI-2.D by the Lawrence Livermore National Laboratory for the NRC (*Reference 18*). It was determined that the peak pressure is 59.3 psig, which is less than the design pressure of 60 psig. In the long term (10,000 to 20,000 sec), the containment temperature stays above the original environmental qualification envelope (250°F versus 225°F). The Ginna Station limiting temperature has thus been increased accordingly. The NRC determined that this small variation had no effect on the qualification status of Ginna Station equipment. The peak temperature of 285.26°F is also less than the design temperature of 286°F. *Reference 36* covers the impact of Extended Power Uprate (EPU).

An evaluation was performed to determine the effect of the BWI replacement steam generators (RSGs) at Ginna Station on the results of the containment response following a LBLOCA. The RSGs have approximately 0.9 percent more mass in the primary system than the original steam generators (OSGs). This would cause the peak reactor building pressure and temperature to increase by approximately 0.5 psi and approximately 1 °F, respectively.

The peak pressure and temperature remain below the acceptance criteria of 60 psig and 286 °F, respectively.

Figure 3.11-1 is of historical interest and shows the nomogram reproduced from Appendix B of the DOR Guidelines. Ginna Station (Pre-uprate power level 1520 MWt, containment volume 997,000 ft^3) is represented by the line shown in Figure 3.11-1.

In June 1984, the NRC issued Revision 1 to Regulatory Guide 1.89. Appendix D of Regulatory Guide 1.89, Revision 1, provides a methodology for determining the qualification radiation dose.

A comparison of the detailed assumptions in developing the dose information contained in Tables D-1 and D-2 of Regulatory Guide 1.89, Revision 1, (reproduced as Tables 3.11-4 and 3.11-5) and Ginna Station is shown in Table 3.11-6.

Although the Ginna Station fan coolers have iodine removal capability, no credit is taken for iodine removal by the filters for conservatism.

The dose rate at the centerline of containment in Tables 3.11-4 and 3.11-5 was determined by the specific activity of the containment atmosphere (i.e., curies/cubic feet). The specific activity is directly proportional to the reactor power level and inversely proportional to the containment volume. The specific activity and therefore the containment centerline dose rate for Ginna Station assuming reactor power of 1811 MWt (or 102% of 1775 MWt which takes into account power measurement uncertainties and is consistent with assumptions used in Section 15.6) is shown below. The equation includes a 4% on reactor power to accomodate variations in the fuel management schemes, a conservative estimate for containment free volume of 997,000 ft^3, and a time dependent scaling factor to address the difference in the fuel cycle length (SF_{BURNUP}).

(1811 MWt/ 4100 MWt) x 1.04 x (2,520,000 ft^3 / 997,000 ft^3) × SF_{BURNUP} x tabulated values shown in Tables 3.11-4 and 3.11-5

or

1.161 x SF_{BURNUP} x the tabulated values of Tables 3.11-4 and 3.11-5.

The time-dependent dose at the containment centerline of Ginna Station is contained in Tables 3.11-2 and 3.11-3.

Reference 38 through 42 cover containment radiation dose due to EPU.

Submergence of valves inside containment is discussed in *Reference 19* where it has been shown that operation following submergence is not required. Submergence of instrumentation is discussed in *Reference 20*. All instrumentation required to function for postaccident monitoring has been elevated to prevent submergence with the exception of two resistance temperature detectors (RTDs) for the reactor vessel level indication system, which included submergence in their environmental qualification.

3.11.3.1.2 Post Main Steam Line Break Environment

The peak pressure following a main steam line break is contained in Section 6.2.1.2.3.2. The temperature associated with the main steam line break is higher than that of the LBLOCA, but was determined by the NRC not to be limiting, however, for qualification of equipment required following a main steam line break because

A. The high temperature transient is very brief and there is super-heated steam (with a lower heat transfer capability), as opposed to saturated steam.

B. The equipment is protected from the direct effects of the steam line break by concrete floors and shields.

C. The sensitive portions of the electrical equipment are not directly exposed to the environment but are protected by housing, cable jackets, and the like.

For these reasons, the humidity and steam environment following a LOCA remains limiting. This is consistent with the NRC Position 4.2 of the Guidelines for Evaluating Environmental Qualification of Class 1E Electrical Equipment in Operating Reactors. Radiation levels in containment following a main steam line break are not limiting since fuel failures are not projected to result from a main steam line break. Chemical environment and submergence are bounded by the LOCA conditions.

The NRC further examined a generic issue concerning main steam line break with continued feedwater addition. In a February 9, 1983, SER (*Reference 21*) the NRC concluded that the results of SEP Topic VI-2.D calculations were acceptable because (1) the main feedwater system is automatically isolated and the preferred auxiliary feedwater system limits flow to the steam generators, (2) the preferred auxiliary feedwater pumps are protected from the effects of runout flow, and (3) all potential water sources were identified and although a reactor return to power would occur, there is no violation of specified acceptable fuel design limits.

3.11.3.2 Auxiliary Building

3.11.3.2.1 Heating, Ventilation, and Air Conditioning

The auxiliary building has a heating, ventilation, and air conditioning system which provides clean, filtered, and tempered air to the operating floor of the auxiliary building. Air from within the Auxiliary Building sweeps the surface of the decontamination pit and spent fuel storage pool. The system exhausts air from the equipment rooms and open areas of the auxiliary building, and from the decontamination pit and spent fuel pool (SFP) through a closed exhaust system. The exhaust system includes a 100%-capacity bank of high efficiency particulate air filters and redundant 100%-capacity fans discharging to the atmosphere via the plant vent. The auxiliary building ventilation system (ABVS) is included in Drawings 33013-1869 through 33013-1872 and is discussed in Section 9.4.2. This arrangement ensures the proper direction of air flow for removal of airborne radioactivity from the auxiliary building.

Included in the auxiliary building exhaust system is a separate charcoal filter circuit, which exhausts from rooms where fission-product activity may accumulate during MODES 1 and 2 in concentrations exceeding the average levels expected in the rest of the building. Although no credit for this system is assumed in the plant safety analysis, this circuit is capable of

providing exhaust ventilation from the areas containing pumps and related piping and valving which are used to recirculate containment sump liquid following a LOCA. A full-flow charcoal filter bank is provided in the circuit, along with two 50%-capacity exhaust fans. The air-operated suction and discharge dampers associated with each fan are interlocked with the fan such that they are fully open when the fan is operating and fully closed when the fan is stopped. These dampers fail to the open position on loss of control signal or control air. The fans discharge to the main auxiliary building exhaust system containing the high efficiency particulate air (HEPA) filter bank. To ensure a path for the charcoal (and HEPA) filtered exhaust to the plant vent if the main exhaust fans are not operating, a fail-open damper is installed in a bypass circuit around the two main exhaust fans. In addition to the main auxiliary building ventilation system (ABVS), the residual heat removal, safety injection, containment spray, and charging pump motors are provided with additional cooling provisions when the pumps are operating. The safety injection and containment spray pump motors are located in an open area in the basement of the auxiliary building and share three service-water-cooled heat exchangers. In 1992, service water to these heat exchangers was blanked off (see Section 9.4.9.1). The charging pumps and residual heat removal pumps are located in individual rooms, each room being provided with two cooling units consisting of redundant fans, water-cooled heat exchangers, and ductwork for circulating the cooled air. The capacity of each charging pump cooling unit is sufficient to maintain acceptable room-ambient temperatures with the minimum number of pumps required for system operation in service. The cooling units in the residual heat removal pump pit are not required for the operation of the residual heat removal pumps, even if both pumps are operating.

In the event of a loss of offsite power, the auxiliary building ventilation system (ABVS) main supply and exhaust fans would be inoperable. However, all other fans in the auxiliary building ventilation system (ABVS) are supplied by emergency diesel power, including the pump cooling circuits for safety-related pump motors, as described above. Analysis has shown that the three levels of the auxiliary building and the residual heat removal pump pit would remain within acceptable limits when the outside air was at its maximum expected temperature and there were no cooling units operating. Since the auxiliary building is a very large volume building, it is not expected that there would be a significant postaccident temperature increase except in some local areas near hot piping and large motors. This situation exists in the basement of the auxiliary building where the safety-related pumps and recirculated sump fluid piping are located.

For the case where a loss-of-coolant accident (LOCA) occurs concurrently with the loss of offsite power, a temperature increase in the auxiliary building operating level could also occur due to spent fuel pool (SFP) heatup, in the event that service water to the spent fuel pool heat exchangers were required to be isolated. The safety-related pumps and associated equipment are qualified for the resulting environments.

3.11.3.2.2 Loss of Ventilation

Normal convective cooling, supplemented by the ventilation system as described above, is adequate to maintain the postaccident temperature within normal ambient levels. In the event that all ventilation were lost, it has been determined that the pumps and associated valves would be capable of operating in the resultant environment for the time required to mitigate

the accident without significant reduction in the available operating life of the equipment (see Section 9.4.2.4).

As part of SEP Topic III-5.B, an extensive review was performed of high and moderate energy pipe breaks. In the auxiliary building it was determined that steam heating line breaks would adversely affect the environmental qualification of safety-related electrical equipment. In response to this postulated pipe break scenario, RG&E provided pipe whip and jet impingement protection for a 6-in. steam line to protect certain cable trays. Also, RG&E made available spare electrical breakers and cable required for operation of a charging pump, as well as procedures and administrative controls. The calculated peak pressure and temperature conditions in the auxiliary building for the event are 150°F and 0.1 psig.

3.11.3.2.3 Radiation Levels

The radiation levels in the auxiliary building would increase in the event of a LOCA. Using conservative postaccident fission-product activity levels, the postaccident environment in the auxiliary building was calculated. This is discussed in detail in Section 12.4.3.3. The only major radiation field in terms of equipment qualification is in the vicinity of the recirculating fluid and in front of containment penetrations. *Reference 43* addresses radiation in front of containment penetrations.. *Reference 6*, as amended by the evaluation performed for the extended power uprate and discussed in *References 35, 38, 39 and 43*, addresses the required qualification doses for all the affected equipment.

3.11.3.2.4 Flooding

Flooding is not a concern in the auxiliary building. A review of potential equipment failures was conducted as part of the *Appendix R* fire protection review as well as SEP Topic III-6, Seismic Design Considerations. It was determined that actuation of the fire protection sprinklers or failure of all nonseismic tanks would not flood required safety-related equipment.

3.11.3.3 Intermediate Building

Implementation of an augmented inservice inspection program for high-energy piping outside containment has reduced the probability of pipe breaks in these systems to acceptably low levels (Section 3.6.2.1). A 6-in. main steam line branch connection break is the intermediate building design-basis event. An analysis of this event resulted in calculated steam conditions of 0.25 psig and 212°F (*References 32*, *33*, and *34*). A pipe crack or branch line that could not be isolated is the limiting design-basis event for the intermediate building environment. Mass and energy release in this case would be limited by the dryout of the steam generators with the duration of the environment dependent on the size of the leak or break. Based on flow through a main steam safety valve (a 6-in. line) of 247 lb/sec at a steam line pressure of 1100 psia and the inventory available for release from a main steam break (see Table 15.6-7), the mass and energy flow will continue for at least 11 minutes. Smaller leaks may continue substantially longer. It is expected that within 30 minutes to 1 hour, action could be taken to provide added ventilation to the building by opening doors. Within several hours, return to near ambient conditions could be accomplished. The exact duration is not critical in terms of affected equipment qualification; therefore, no explicit calculations have been performed. Chemical spray is not a design consideration in this building. The effects of submergence

need not be considered, as discussed in *References 22*, *23*, and *24*. *Reference 8* presents the result of an analysis performed to ensure that safety-related equipment would not be flooded in the event of a feed line break in the intermediate building.

The turbine-driven auxiliary feedwater pump (TDAFW) area was analyzed to determine the resultant environmental conditions if all ventilation were lost. The purpose was to obtain data to assess the feasibility of performing manual operation of certain valves in the area. The analysis showed that the peak temperature would reach 145°F within the first hour and then stabilize (*Reference 31*).

The radiation environment was reviewed in response to the TMI Lessons Learned commitments. With the exception of areas in front of containment penetrations (*Reference 43*), the radiation environment is not significant in terms of equipment qualification.

As part of SEP Topic III-5.B, a review was made of high-energy line failures which could affect the steam and feedwater lines in the intermediate building. Potential cracks in the steam and feedwater piping were determined to be insignificant in terms of damaging required safe shutdown equipment. An evaluation was made of the postulated consequences of intermediate building block wall failure due to a high-energy line break in the turbine building. It was determined that failure of the safety and relief valves would not be limiting and that auxiliary feedwater flow would be maintained. However, RG&E did commit to evaluate, and modify as necessary, the structural integrity of steam and feedwater lines, main steam isolation valves, and auxiliary feedwater connections in conjunction with the Ginna Station Structural Upgrade Program (*Reference 25*) in order to provide protection from the failure of the adjacent wall. This information is provided in more detail in Section 3.6.2.

3.11.3.4 Cable Tunnel

Since the cable tunnel is effectively open to the intermediate building, the limiting environmental conditions for the cable tunnel are identical to the intermediate building conditions. However, physical separation is such that no concern exists with respect to direct effects such as jet impingement due to postulated high-energy line breaks.

3.11.3.5 Control Building

The limiting environmental conditions of the control building, which includes the control room, relay room, and battery rooms, is normal ambient conditions. Protection against high-energy line breaks and circulating water line breaks which could occur outside the control building and affect the control building environment are identified and discussed in *References 20 through 24* and *26 through 30*.

The air conditioning system for the control room is described in *Sections 6.4*, and consists of a single train of non-safety related NORMAL Control Room HVAC, plus two trains of Safety Related Control Room Emergency Air Treatment System (CREATS). Any of these 3 trains is capable of maintaining Control Room temperatures in a comfortable range for continuous long-term human occupancy, however, the value for post accident service conditions in the Control Room remains at 104°F so that future equipment specified for installation in the

Control Room will be specified to withstand the higher localized temperatures that occur inside of cabinets and control cabinets.

The relay room is normally cooled by two non-safety-related air conditioning systems, which can be manually aligned to the emergency buses by closing the proper bus-tie breakers. Use of portable air conditioning units and fans are options available to maintain environmental conditions within the required specifications.

The battery rooms have a set of inlet and exhaust fans, as well as an air conditioning system. Additional fans powered directly from the batteries have also been installed.

As part of the SEP Topic III-5.B review, RG&E determined that steam heating coils in the control building would result in a harsh environment due to a postulated failure. These sources of steam have been removed from the control building.

3.11.3.6 Diesel Generator Rooms

The emergency diesel generator rooms each have their own heating, ventilation, and air conditioning systems, powered from the diesels. As soon as the diesels are brought up to speed, stabilized, and their respective circuit breakers closed to their emergency buses, the heating, ventilation, and air conditioning systems (ventilating fans) are energized.

Failure of a steam heating line would affect only one diesel. The other diesel, as well as off-site power, would still be available. This configuration has been reviewed by the NRC in *Reference 28* and found acceptable. Protection against events outside the rooms is described in *References 20*, *23*, *26*, *27*, and *30*. The limiting environment in the diesel generator rooms, therefore, is normal ambient conditions.

To provide protection from flooding in the diesel-generator rooms due to a circulating water line break, 18-in.-high steel curbs were installed in the diesel generator rooms. Subsequent installation of the "superwall" at the turbine building interface precludes the necessity for the curbs at that location.

3.11.3.7 Turbine Building

The turbine building does not require a heating, ventilation, and air conditioning system per se, but rather utilizes roof vent fans, wall vent fans, windows, and unit heaters for control of the turbine building environment.

In the event of loss of power to fans in this building, there would be no significant temperature rise since it is a large volume building with sufficient openings (windows and access doors) to adequately circulate the outside air.

Analyses have shown that the limiting pressure is caused by an instantaneous break in the 20-in. feed line in the turbine building (see Section 3.6.2.5.1). Peak pressures are 1.14 psig on the lower two levels of the building and 0.70 psig on the operating floor. Failure of portions of the exterior wall limits the duration of the pressure pulse to a few seconds. Pressure and temperature is limited by the failure capacity of the exterior walls. Assuming saturation conditions, the limiting temperature is approximately 220°F. A 100% humidity steam-air

mixture is assumed. Isolation of the main steam and feed system will isolate the source of energy to the turbine building. For conservatism, it has been assumed that the peak pressure and temperature condition persists for 30 minutes with return to ambient being accomplished in a total of 3 hours. The exact duration of high environmental conditions is not critical in terms of affected equipment qualification; therefore, no explicit calculations have been performed.

The limiting flood condition resulting from a circulating water system pipe break is 18 in. of water level in the basement of the building (*Reference 20*).

3.11.3.8 Auxiliary Building Annex

This structure houses the standby auxiliary feedwater system. The limiting environment in this structure is normal ambient conditions. The cooling system for this building is redundant and seismically qualified. Flooding is not a concern since all safety-related equipment associated with the standby auxiliary feedwater system (SAFW) is elevated and there is no large volume of water stored in the building.

3.11.3.9 Screen House

The screen house, like the turbine building, does not require a heating, ventilation, and air conditioning system, but utilizes roof vent fans, wall vent fans, windows and unit heaters for control of the environment. In the event of a loss of power to the fans, there would be no significant temperature rise, since it is a large volume building with sufficient openings to adequately circulate outside air.

The limiting environment in the screen house is normal ambient conditions. A review was conducted as part of SEP Topic III-5.B to evaluate the effects of high and moderate-energy line breaks in the screen house. It was determined that no protection was needed because alternative shutdown means are available, which do not rely upon service water from the screen house. Curbs were installed in the screen house in 1975 to protect critical equipment from the flooding source of a potential circulating water line break.

3.11.4 EQUIPMENT QUALIFICATION INFORMATION

Complete and auditable records which include supporting documentation for environmental qualification of safety-related electrical equipment are maintained by Ginna Station. The documentation which includes test results, specifications, reports, and other data has been identified by documentation reference citings in the Ginna Station reports to the NRC on the environmental qualification program.

3.11.5 ENVIRONMENTAL QUALIFICATION PROGRAM

The Nuclear Policy Manual defines the additional quality assurance program requirements for replacement and maintenance of environmentally qualified equipment to ensure compliance with the requirements of 10 CFR 50.49. The environmental qualification program is embedded in procedures for design, installation, and maintenance of systems and components. The Equipment Qualification Master List is arranged by system. The Nuclear Policy Manual is the controlling document for the environmental qualification program and assigns

the Engineering Department the responsibility for establishing an evaluation process that documents the basis for any changes in the Equipment Qualification Master List.

REFERENCES FOR SECTION 3.11

1. Letter from D. L. Ziemann, NRC, to L. D. White, Jr., RG&E, Subject: Electrical Equipment Environmental Qualification, dated February 15, 1980.

2. NRC Interim Rule, 10 CFR Part 50.49, Environmental Qualification of Electric Equipment, June 25,1982.

3. Letter from L. D. White, Jr., RG&E, to A. Schwencer, NRC, Subject: Environmental Qualification of Electrical Equipment, dated February 24, 1978.

4. Letter from L. D. White, Jr., RG&E, to D. L. Ziemann, NRC, Subject: Environmental Qualification of Electrical Equipment, dated December 1, 1978.

5. Letter from L. D. White, Jr., RG&E, to D. L. Ziemann, NRC, Subject: Environmental Qualification of Electrical Equipment, Revision 2, dated April 25, 1980.

6. Letter from J. E. Maier, RG&E, to D. G. Eisenhut, NRC, Subject: Environmental Qualification of Electrical Equipment, dated October 31, 1980.

7. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Equipment Qualification of Safety-Related Electrical Equipment, dated June 1, 1981.

8. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Environmental Qualification of Safety-Related Electrical Equipment, dated September 4, 1981.

9. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Environmental Qualification of Electrical Equipment, dated November 6, 1981.

10. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Schedule for Environmental Qualification of Electrical Equipment, dated February 18, 1982.

11. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Safety-Related Evaluation Report for Environmental Qualification of Safety-Related Electrical Equipment, dated December 13, 1982.

12. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: 10 CFR 50.49, Environmental Qualification of Electrical Equipment, dated May 19, 1983.

13. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Environmental Qualification of Electrical Equipment, dated February 1, 1983.

14. Letter from R. W. Kober, RG&E, to D. M. Crutchfield, NRC, Subject: Environmental Qualification of Electrical Equipment, dated March 30, 1984.

15. Letter from R. W. Kober, RG&E, to W. Paulson, NRC, Subject: Environmental Qualification of Electrical Equipment, dated August 30, 1984.

16. Letter from R. W. Kober, RG&E, to J. A. Zwolinski, NRC, Subject: Generic Letter 84-24, Environmental Qualification of Electrical Equipment, dated January 24, 1985.

17. Letter from J. A. Zwolinski, NRC, to R. W. Kober, RG&E, Subject: Environmental Qualification of Electrical Equipment Important to Safety, dated February 28, 1985.

18. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topics VI-2.D and VI-3, dated November 3, 1981.

19. Letter from L. D. White, Jr., RG&E, to R. A. Purple, NRC, Subject: Valves Subject to Flooding, dated June 16, 1975.

20. Letter from R. A. Purple, NRC, to L. D. White, Jr., RG&E, Subject: Emergency Core Cooling System Valve Modification, dated July 3, 1975.

21. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Main Steam Line Break with Continued Feedwater Addition, dated February 9, 1983.

22. Letter from L. D. White, Jr., RG&E, to D. L. Ziemann, NRC, Subject: High-Energy Line Breaks Outside Containment, dated June 27, 1979.

23. Letter from K. W. Amish, RG&E, to A. Giambusso, NRC, Subject: Transmittal of GAI Report No. 1815 on Effects of Postulated Pipe Breaks Outside the Containment Building, dated November 1, 1973.

24. Letter from K. W. Amish, RG&E, to E. G. Case, NRC, Subject: Pipe Breaks Outside Containment, dated November 1, 1974.

25. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-5.B, Pipe Break Outside Containment, dated July 20, 1983.

26. Letter from K. W. Amish, RG&E, to A. Schwencer, NRC, Subject: Pressure Shielding Steel Diaphragm in Turbine Building, dated February 6, 1978.

27. Letter from R. A. Purple, NRC, to L. D. White, Jr., RG&E, Subject: Amendment No. 7 to Provisional Operating License DPR-18, and transmittal, dated May 14, 1975.

28. Letter from L. D. White, Jr., RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-5.B, Pipe Break Outside Containment, dated August 7, 1980.

29. Letter from D. M. Crutchfield, NRC, to L. D. White, Jr., RG&E, Subject: SEP Topic III-5.B, Pipe Break Outside Containment, dated June 24, 1980.

30. Letter from L. D. White, RG&E, to B. C. Rusche, NRC, Subject: Long-Term Cooling, dated May 13, 1975.

31. Devonrue, Engineering Evaluation of R. E. Ginna Nuclear Power Plant Ventilation System, dated July 1998.

32. Letter from JoEllen West, SAIC, to George Wrobel, subject: Analysis of steam line break in the Intermediate Building, dated October 17, 1986 (ref. UCNs 2/1014 and 2/620).

33. Letter from JoEllen West, SAIC, to George Wrobel, subject: Analysis of steam line break in the Intermediate Building, dated October 29, 1986 (ref. UCNs 2/1014 and 2/620).

34. Letter from JoEllen West, SAIC, to George Wrobel, subject: Analysis of steam line break in the Intermediate Building, dated December 12, 1986 (ref. UCNs 2/1014 and 2/620).

35. Letter from M.G. Korsnick, Ginna Station, to NRC, Subject: "R.E. Ginna Nulear Power Plant, Licensing Amendment Request Regarding Extended Power Uprate, dated July 7, 2005.

36. Letter from Westinghouse Electric Company, Nuclear Services, to D. Graves, Stone & Webster Engineering, RGE-05-52, Ginna Extended Poser Uprate Program, Transmittal of Final Containment Accident Heat Loads, Pressures, Temperatures, and Sump Water Temperatures to Stone & Webster and Ken Rubin Enterprises, dated June 24, 2005.

37. Stone & Webster, Calculation No. 109682-M-014, HVAC System EPU Evaluation, Revision 0, dated March 18, 2005.

38. Stone & Webster, Calculation No. 109682-UR-002, Impact of EPU on Normal Operation Radiation Levels, Shielding Adequacy and Normal Operation Radiation Environments in EQ Zones, Revision 0, dated January 19, 2005.

39. Stone & Webster, Calculation No. 109682-UR-006, Impact of EPU on Post-Accident Radiation Environments in EQ Zones, Revision 0, dated December 29, 2004.

40. Stone & Webster, Calculation No. 109682-UR-007, Post-LOCA Direct Shine Dose from the Containment Recirculation Fan Cooler (CRCF) Charcoal Filters, Revision 0, dated April 25, 2005.

41. Stone & Webster, Calculation No. 109682-UR-008, Post-LOCA Direct Shine Dose through Containment Wall in the Intermediate Building due to Airborne activity within Containment, Revision 0, dated April, 27, 2005.

42. Stone & Webster, Calculation No. 109682-UR-009, Post-LOCA Direct Shine Dose from the Containment Recirculation Fan Cooler (CRCF) HEPA Filters, Revision 1, dated July 7, 2014.

43. Constellation Energy, Constellation Generation Group, Fuel Operations Support Unit, Calculation CA06589, Post Accident Penetration Streaming Doses in the Ginna Intermediate and Auxiliary Buildings, Revision 0, dated June 3, 2005.

44. Stone & Webster, Calculation No. 109682-UR-003, Post-Accident Dose and Dose Rate Scaling Factors to Address the Impact of EPU on Environmental Service Zones (EQ) and Vital Area Access Mission Dose, Revision 1, dated July 7, 2014.

Table 3.11-1
ENVIRONMENTAL SERVICE CONDITIONS FOR EQUIPMENT DESIGNED TO MITIGATE DESIGN-BASIS EVENTS

INSIDE CONTAINMENT

Normal Operation (MODES 1 and 2)

Temperature	60°F to 125°F
Pressure	0 psig
Humidity	50% (nominal)
Radiation[a]	Less than 1 rad/hr. general. Can be higher or lower near specific components.

Accident Conditions (LOCA)

Temperature	Figure 6.1-1 (286°F maximum)
Pressure	Figure 6.1-2 (60 psig design)
Humidity	100%
Radiation[b]	Tables 3.11-2 and 3.11-3; 1.82 x 10^7 rads gamma; 2.99 x 10^8 rads beta
Chemical spray	Solution of boric acid (2750 to 3050 ppm boron) plus NaOH in water. Sump solution pH between 7.8 and 9.5, spray pH < 10.25.
Flooding	7-feet (approximately). Maximum submergence elevation is 242 ft. 8 in.

AUXILIARY BUILDING

Normal Operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Less than 24 mrad/hr. general, with areas near residual heat removal piping less than 120 mrad/hr. during shutdown operation.

Accident Conditions (LOCA or steam line break in containment)

Pressure	0 psig
Humidity	60% (nominal)
Operating floor - 271-ft. elevation	
Temperature	Peak of 131o F within 26 hours, due to terminating SFP Cooling immediately following a LOCA. The temperature cycles between 120o F and 130o F over a 24 hour period; due to solar gain effects, until SFP temperature is reduced by reestablishing SFP cooling.
Radiation near bus 14 and motor control center 1C and 1L.[c]	132 rad
Radiation[c] at other areas.	Less than 50 rad total
Intermediate floor - 253-ft. elevation	
Temperature	Peak of 102°F within 20 hours; stabilizes at less than 100°F after 24 hours.
Radiation[c] near bus 16 and motor control center 1D and 1M.	1190 rad
Radiation[c] at other areas.	Less than 500 rad total
Basement floor - 236-ft. elevation	
Temperature; basement level, West.	Peak of 104°F within 20 hours; stabilizes at less than 100°F after 24 hours.
Temperature; basement level, East near safety injection and containment spray pumps.	Peak of 111°F within 4 hours; stabilizes at less than 100°F after 24 hours.
Radiation[c]; basement level, near containment spray near residual heat removal, and safety injection pumps and piping.	3.7 x 10^6 rad total (at contact); 6.6 x 10^4 rad total 10 feet distance.

Radiation[c] at other areas.	Less than 10^4 rad total

Residual heat removal pump pit

Temperature	Temperature range of 162°F to 142°F from 10 hours to 24 hours after loss-of-coolant accident (LOCA). Peak of 166°F following an assumed 50 gpm residual heat removal (RHR) pump seal leak after 24 hours. Peak temperature lasts less than one hour. Room temperature decreases to 150°F, 40 hours after loss-of-coolant accident (LOCA).
Flooding	8.2 inches

Accident Conditions Based Upon High-Energy Line Breaks or Moderate-Energy Line Breaks:

Temperature (peak)	150°F
Pressure (peak)	0.1 psig
Humidity	≈100%
Radiation	Not applicable
Flooding	0 feet

INTERMEDIATE BUILDING

Normal Operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Less than 6 mrad/hr. (higher near reactor coolant sampling lines).

Accident Conditions Based Upon High-Energy Line Breaks or Moderate-Energy Lines Breaks

Temperature	212°F for 30 minutes; then reducing to 104°F within 3 hours.
Pressure	0.25 psig for 30 minutes; then reducing to 0 psig within 3 hours
Humidity	≈100% indefinitely
Radiation	Not applicable
Flooding	0 feet

Accident Conditions Based Upon LOCA Conditions:

Temperature	115°F indefinitely[d] near large motors and feedwater and steam line piping. 104°F in open areas.
Pressure	0 psig
Humidity	≈100%
Radiation[d]	Negligible
Flooding	None of consequence. (*See Reference 8*)

CABLE TUNNEL

Same as INTERMEDIATE BUILDING

CONTROL BUILDING

Control Room

Normal operation (MODES 1 and 2)

Temperature	50°F to 104°F (usually 70°F to 78°F)
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions

Temperature	Less than 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible
Flooding	Not applicable

Relay Room & Relay Room Annex

Normal operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions

Temperature	Less than 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible
Flooding	Not applicable

Battery Rooms

Normal operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions

Temperature	Less than 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible
Flooding	Not applicable

Mechanical Equipment Room

Normal operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions:

Temperature	Less than 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible
Flooding	3 feet (estimated for a service water line leak).

DIESEL GENERATOR ROOMS

Normal operation (MODES 1 and 2)

Temperature	60°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

<u>Accident Conditions (Maximum Design Temperature Day)</u>

Temperature	Less than 125°F
Pressure	0 psig
Humidity	90% (estimated)
Radiation	Negligible
Spray	Not applicable
Flooding [e]	0 ft

<u>One Ventilation Fan Operating (Maximum Design Temperature Day)</u>

Temperature	Less than 140°F

TURBINE BUILDING

<u>Normal operation (MODES 1 and 2)</u>

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

<u>Accident Conditions (High-Energy Line Break)</u>

Temperature	220°F for 30 minutes, reduce to 100°F within 3 hours
Pressure	1.14 psig on mezzanine and basement levels, 0.7 psig on operating floor for 30 minutes, reduce to ambient 3 hours.
Humidity	100 %
Radiation	Negligible
Flooding	18 inches in basement (circulating water break)

AUXILIARY BUILDING ANNEX

Normal operation (MODES 1 and 2)

Temperature	60°F to 120°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions

Temperature	60°F to 120°F
Pressure	0 psig
Humidity	60% (normal)
Radiation	Negligible
Flooding	Approximately 2 feet

SCREEN HOUSE

Normal operation (MODES 1 and 2)

Temperature	50°F to 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible

Accident Conditions

Temperature	Less than 104°F
Pressure	0 psig
Humidity	60% (nominal)
Radiation	Negligible
Flooding	18 inches (circulation water break)

NOTE:—Temperature considerations for station blackout are contained in Section 8.1.4.5.2

a. Areas where the dose rates are expected to be higher are: (1) Reactor Cavity area. (2) Areas near components that contain reactor coolant, such as RCS loop cubicles and the regenerative heat exchanger area. The appropriate dose rates for these areas are 40 rad/hr. See *Reference 39*.

b. Dose estimates in areas adjacent to the containment recirculation fan cooler charcoal and HEPA filters will be higher than the containment general area doses. For such cases component location specific assessments are utilized as needed. See *References 40, 42 and 43.*

c. Dose estimates are determined for a LOCA with one (1) train of Engineered Safety Feature (ESF) cooling operating. Dose estimates in areas in front of containment penetrations will be higher than that estimated for the zone. For such cases, component location specific assessments are utilized as needed. See *Reference 43*.

d. Estimated (no explicit calculations performed).

e. Service water line crack would affect only one room.

Table 3.11-2
ESTIMATES FOR TOTAL AIRBORNE GAMMA DOSE CONTRIBUTORS IN CONTAINMENT TO A POINT IN THE CONTAINMENT CENTER - GINNA STATION

Time (hr.)	Airborne Iodine Dose (Rem)	Airborne Noble Gas Dose (Rem)	Plateout Iodine Dose (Rem)	Total Dose (Rem)
0.00	---	---	---	---
0.03	5.63E+04	8.86E+04	1.97E+03	1.47E+05
0.06	1.00E+05	1.66E+05	4.65E+03	2.71E+05
0.09	1.27E+05	2.37E+05	8.43E+03	3.72E+05
0.12	1.46E+05	3.00E+05	1.28E+04	4.59E+05
0.15	1.61E+05	3.60E+05	1.77E+04	5.39E+05
0.18	1.71E+05	4.16E+05	2.29E+04	6.10E+05
0.21	1.81E+05	4.69E+05	2.81E+04	6.78E+05
0.25	1.92E+05	5.37E+05	3.54E+04	7.63E+05
0.38	2.19E+05	7.41E+05	5.89E+04	1.02E+06
0.5	2.37E+05	9.11E+05	8.05E+04	1.23E+06
0.75	2.75E+05	1.23E+06	1.24E+05	1.63E+06
1	3.10E+05	1.51E+06	1.63E+05	1.98E+06
2	4.22E+05	2.45E+06	3.05E+05	3.18E+06
5	6.41E+05	4.30E+06	6.30E+05	5.57E+06
8	7.74E+05	5.30E+06	8.71E+05	6.95E+06
24	1.18E+06	7.58E+06	1.69E+06	1.05E+07
60	1.53E+06	8.60E+06	2.45E+06	1.26E+07
96	1.69E+06	9.03E+06	2.78E+06	1.35E+07
192	1.96E+06	9.85E+06	3.33E+06	1.52E+07
298	2.15E+06	1.04E+07	3.71E+06	1.62E+07
394	2.27E+06	1.05E+07	3.97E+06	1.67E+07
560	2.41E+06	1.08E+07	4.24E+06	1.73E+07
720	2.47E+06	1.09E+07	4.37E+06	1.76E+07
888	2.51E+06	1.09E+07	4.45E+06	1.79E+07
1060	2.53E+06	1.09E+07	4.49E+06	1.80E+07
1220	2.54E+06	1.09E+07	4.52E+06	1.80E+07
1390	2.55E+06	1.09E+07	4.53E+06	1.80E+07

Time (hr.)	Airborne Iodine Dose (R)	Airborne Noble Gas Dose (R)	Plateout Iodine Dose (R)	Total Dose (R)
1560	2.55E+06	1.09E+07	4.55E+06	1.81E+07
1730	2.55E+06	1.09E+07	4.55E+06	1.81E+07
1900	2.55E+06	1.10E+07	4.56E+06	1.81E+07
2060	2.55E+06	1.10E+07	4.56E+06	1.81E+07
2230	2.55E+06	1.10E+07	4.56E+06	1.81E+07
2950	2.55E+06	1.10E+07	4.56E+06	1.81E+07
3670	2.55E+06	1.10E+07	4.56E+06	1.81E+07
4390	2.55E+06	1.10E+07	4.56E+06	1.81E+07
5110	2.55E+06	1.10E+07	4.56E+06	1.81E+07
5830	2.55E+06	1.10E+07	4.56E+06	1.81E+07
6550	2.55E+06	1.10E+07	4.56E+06	1.81E+07
7270	2.55E+06	1.10E+07	4.56E+06	1.81E+07
8000	2.55E+06	1.10E+07	4.56E+06	1.81E+07
8710	2.55E+06	1.10E+07	4.56E+06	1.82E+07
			TOTAL	1.82E+07

Table 3.11-3
ESTIMATES FOR TOTAL AIRBORNE BETA DOSE CONTRIBUTORS IN CONTAINMENT TO A POINT IN THE CONTAINMENT CENTER - GINNA STATION

Time (hr)	Airborne Iodine Dose (rads)[a]	Airborne Noble Gas Dose (rads)[a]	Total Dose (rads)[a]
0.00	---	---	---
0.03	1.68E+05	6.34E+05	8.02E+05
0.06	2.99E+05	1.14E+06	1.44E+06
0.09	3.80E+05	1.56E+06	1.94E+06
0.12	4.37E+05	1.91E+06	2.35E+06
0.15	4.79E+05	2.22E+06	2.70E+06
0.18	5.11E+05	2.49E+06	3.00E+06
0.21	5.39E+05	2.73E+06	3.27E+06
0.25	5.69E+05	3.02E+06	3.59E+06
0.38	6.45E+05	3.85E+06	4.49E+06
0.5	7.00E+05	4.51E+06	5.21E+06
0.75	8.11E+05	5.72E+06	6.53E+06
1	9.10E+05	6.81E+06	7.72E+06
2	1.22E+06	1.06E+07	1.18E+07
5	1.80E+06	1.95E+07	2.13E+07
8	2.14E+06	2.61E+07	2.82E+07
24	>3.26E+06	4.85E+07	5.18E+07
60	4.42E+06	7.27E+07	7.71E+07
96	4.96E+06	8.81E+07<	9.30E+07
192	5.83E+06	1.17E+08	1.23E+08
298	6.40E+06	1.37E+08	1.43E+08
394	6.79E+06	1.46E+08	1.53E+08
560	7.19E+06	1.57E+06	1.65E+08
720	7.40E+06	1.64E+08	1.71E+08
888	7.51E+06	1.68E+08	1.75E+08
1060	7.58E+06	1.71E+08	1.78E+08
1220	7.63E+06	1.72E+08	1.80E+08

Time (hr)	Airborne Iodine Dose (rads)[a]	Airborne Noble Gas Dose (rads)[a]	Total Dose (rads)[a]
1390	7.65E+06	1.75E+08	1.83E+08
1560	7.66E+06	1.78E+08	1.86E+08
1730	7.66E+06	1.81E+08	1.89E+08
1900	7.66E+06	1.83E+08	1.90E+08
2060	7.66E+06	1.86E+08	1.93E+08
2230	7.67E+06	1.87E+08	1.95E+08
2950	7.67E+06	1.98E+08	2.06E+08
3670	7.67E+06	2.09E+08	2.17E+08<
4390	7.67E+06	2.20E+08	2.27E+08
5110	>7.67+06	2.31E+08	2.38E+08
5830	7.67E+06	2.40E+08	2.48E+08
6550	7.67E+06	2.51E+08	2.59E+08
7270	7.67+06	2.62E+08	2.70E+08
8000	7.67E+06	2.72E+08	2.80E+08
8710	7.67E+06	2.83E+08	2.91E+08

a. Dose conversion factor is based on absorption by tissue.

Table 3.11-4
ESTIMATES FOR TOTAL AIRBORNE GAMMA DOSE CONTRIBUTORS IN CONTAINMENT TO A POINT IN THE CONTAINMENT CENTER, REGULATORY GUIDE 1.89, REVISION 1

Time (hr)	Airborne Iodine Dose (R)	Airborne Noble Gas Dose (R)	Plateout Iodine Dose (R)	Total Dose (R)
0.00	---	---	---	---
0.03	4.82E+4	7.42E+4	1.69E+3	1.24E+5
0.06	8.57E+4	1.39E+5	3.98E+3	2.29E+5
0.09	1.09E+5	1.98E+5	7.22E+3	3.14E+5
0.12	1.25E+5	2.51E+5	1.10E+4	3.87E+5
0.15	1.38E+5	3.01E+5	1.52E+4	4.54E+5
0.18	1.47E+5	3.48E+5	1.96E+4	5.15E+5
0.21	1.55E+5	3.92E+5	2.41E+4	5.71E+5
0.25	1.64E+5	4.49E+5	3.03E+4	6.43E+5
0.38	1.87E+5	6.19E+5	5.05E+4	8.57E+5
0.50	2.03E+5	7.61E+5	6.90E+4	1.03E+6
0.75	2.36E+5	1.03E+6	1.06E+5	1.37E+6
1.00	2.66E+5	1.26E+6	1.40E+5	1.67E+6
2.00	3.62E+5	2.04E+6	2.61E+5	2.66E+6
5.00	5.50E+5	3.56E+6	5.40E+5	4.65E+6
8.00	6.63E+5	4.38E+6	7.47E+5	5.79E+6
24.0	1.01E+6	6.26E+6	1.45E+6	8.72E+6
60.0	1.31E+6	7.16E+6	2.10E+6	1.06E+7
96.0	1.45E+6	7.56E+6	2.39E+6	1.14E+7
192	1.68E+6	8.29E+6	2.86E+6	1.28E+7
298	1.85E+6	8.76E+6	3.19E+6	1.38E+7
394	1.95E+6	8.85E+6	3.41E+6	1.42E+7
560	2.07E+6	9.06E+6	3.64E+6	1.48E+7
720	2.13E+6	9.15E+6	3.76E+6	1.50E+7
888	2.16E+6	9.19E+6	3.83E+6	1.52E+7
1060	2.18E+6	9.21E+6	3.87E+6	1.53E+7
1220	2.19E+6	9.21E+6	3.89E+6	1.53E+7

Time (hr)	Airborne Iodine Dose (R)	Airborne Noble Gas Dose (R)	Plateout Iodine Dose (R)	Total Dose (R)
1390	2.20E+6	9.21E+6	3.90E+6	1.53E+7
1560	2.20E+6	9.22E+6	3.91E+6	1.53E+7
1730	2.20E+6	9.22E+6	3.91E+6	1.53E+7
1900	2.20E+6	9.22E+6	3.92E+6	1.53E+7
2060	2.20E+6	9.22E+6	3.92E+6	1.53E+7
2230	2.20E+6	9.22E+6	3.92E+6	1.53E+7
2950	2.20E+6	9.23E+6	3.92E+6	1.54E+7
3670	2.20E+6	9.24E+6	3.92E+6	1.54E+7
4390	2.20E+6	9.24E+6	3.92E+6	1.54E+7
5110	2.20E+6	9.25E+6	3.92E+6	1.54E+7
5830	2.20E+6	9.25E+6	3.92E+6	1.54E+7
6550	2.20E+6	9.26E+6	3.92E+6	1.54E+7
7270	2.20E+6	9.27E+6	3.92E+6	1.54E+7
8000	2.20E+6	9.27E+6	3.92E+6	1.54E+7
8710	2.20E+6	9.28E+6	3.92E+6	1.54E+7
			TOTAL	1.54E+7

Table 3.11-5
ESTIMATES FOR TOTAL AIRBORNE BETA DOSE CONTRIBUTORS IN CONTAINMENT TO A POINT IN THE CONTAINMENT CENTER, REGULATORY GUIDE 1.89, REVISION 1

Time (hr)	Airborne Iodine Dose (rads)[a]	Airborne Noble Gas Dose (rads)[a]	Total Dose (rads)[a]
0.00	---	---	---
0.03	1.47E+5	5.48E+5	6.95E+5
0.06	2.62E+5	9.86E+5	1.25E+6
0.09	3.33E+5	1.35E+5	1.68E+6
0.12	3.83E+5	1.65E+6	2.03E+6
0.15	4.20E+5	1.91E+6	2.33E+6
0.18	4.49E+5	2.14E+6	2.59E+6
0.21	4.73E+5	2.35E+6	2.82E+6
0.25	5.00E+5	2.60E+6	3.10E+6
0.38	5.67E+5	3.30E+6	3.87E+6
0.50	6.15E+5	3.86E+6	4.48E+6
0.75	7.13E+5	4.89E+6	5.60E+6
1.00	8.00E+5	5.81E+6	6.61E+6
2.00	1.07E+6	9.02E+6	1.01E+7
5.00	1.58E+6	1.65E+7	1.81E+7
8.00	1.88E+6	2.20E+7	2.39E+7
24.0	2.87E+6	4.08E+7	4.37E+7
60.0	3.89E+6	6.15E+7	6.54E+7
96.0	4.37E+6	7.48E+7	7.92E+7
192	5.14E+6	1.00E+8	1.05E+8
298	5.64E+6	1.17E+8	1.23E+8
394	5.99E+6	1.25E+8	1.31E+8
560	6.34E+6	1.34E+8	1.40E+8
720	6.53E+6	1.39E+8	1.46E+8
888	6.63E+6	1.42E+8	1.49E+8
1060	6.69E+6	1.44E+8	1.51E+8
1220	6.73E+6	1.45E+8	1.52E+8

Time (hr)	Airborne Iodine Dose (rads)[a]	Airborne Noble Gas Dose (rads)[a]	Total Dose (rads)[a]
1390	6.75E+6	1.47E+8	1.54E+8
1560	6.76E+6	1.49E+8	1.56E+8
1730	6.76E+6	1.51E+8	1.58E+8
1900	6.76E+6	1.52E+8	1.59E+8
2060	6.76E+6	1.54E+8	1.61E+8
2230	6.77E+6	1.55E+8	1.62E+8
2950	6.77E+6	1.62E+8	1.69E+8
3670	6.77E+6	1.69E+8	1.76E+8
4390	6.77E+6	1.76E+8	1.83E+8
5110	6.77E+6	1.83E+8	1.90E+8
5830	6.77E+6	1.89E+8	1.96E+8
6550	6.77E+6	1.96E+8	2.03E+8
7270	6.77E+6	2.03E+8	2.10E+8
8000	6.77E+6	2.09E+8	2.16E+8
8710	6.77E+6	2.16E+8	2.23E+8
		TOTAL	2.23E+8

a. Dose conversion factor is based on absorption by tissue.

Table 3.11-6
GINNA STATION/REGULATORY GUIDE 1.89, APPENDIX D, COMPARISON OF POSTACCIDENT RADIATION ENVIRONMENT ASSUMPTIONS

The in-containment post-LOCA radiation environments provided in Appendix D of Regulatory Guide 1.89, Rev. 1 is based on a core power level of 4100 MWt and a 12 month fuel cycle length. The core power level utilized to develop the radiation environment at Ginna is 1811 MWt (includes 2% margin for power measurement uncertainties). The fuel cycle length utilized for Ginna Station is 18 months.

Appendix D Paragraph	Regulatory Guide 1.89	Ginna Station
2.1.1	Release of 50% of the iodine and 100% of the noble gas inventory to the containment atmosphere.	Release of 50% of the iodine and 100% of the noble gas inventory to the containment atmosphere.
2.1.2	Containment free volume of 2.52 x 10^6 ft^3 74% or 1.86 x 10^6 ft^3 directly covered by containment spray.	Containment free volume of 1.00 x 10^6 ft^3. 78% (minimum) or 7.8 x $10^5 ft^3$ covered by containment spray.
2.1.3	Large release uniformly distributed in a relatively open containment.	Large release uniformly distributed in a relatively open containment.
2.1.4	ESF fans with a flow rate of 220,000 cfm. Mixing between all major unsprayed regions and the main spray region.	Four fan coolers produce approximately 132,000 cfm. Thorough mixing is obtained.[a]
2.1.6	Containment spray from two equal capacity trains each rated for 3000 gpm boric acid solution.	Containment spray from two equal capacity trains each bounded by 1200 to 1800 gpm boric acid solution.[b]

a. The Regulatory Guide 1.89 fan cooler flow rate of 220,000 cfm results in a complete recirculation of 2.52 x 10^6 ft^3 of the containment atmosphere every 11.45 min. The Ginna Station fan coolers recirculate the atmosphere once every 7.58 min.

b. The Regulatory Guide 1.89 spray system provides for a spray flow of 1 gpm for every 310 ft^3 of sprayed volume. The Ginna Station spray system provides a spray flow of 1 gpm for every 325 ft^3 of sprayed volume.

APPENDIX 3A

INITIAL EVALUATION OF CAPABILITY TO WITHSTAND TORNADOES

3A.1 INTRODUCTION AND CONCLUSIONS

Ginna Station is located in an area that is relatively tornado free. When the plant design criteria were approved for construction, tornado requirements were not considered necessary. Consequently, Ginna Station was not originally designed in accordance with current tornado requirements.

This appendix contains an analysis of the capability of the plant as built to withstand tornado effects. The adopted criterion is that the plant shall be maintained in a hot shutdown condition during and after tornado passage.

The structures and systems, or parts thereof, required for maintaining the plant in a hot shut-down condition have been checked against the following main tornado characteristics:

1. Tangential wind velocity of 300 mph.
2. External vacuum of 3 psi gauge.

The results of this analysis show that the reactor containment is capable of resisting the tornado loads and the buildings housing critical equipment will not collapse or suffer gross failure. Some of the areas in these buildings might be exposed to the weather because siding, windows, doors, or ventilation openings would blow outward if directly struck by a tornado with the characteristics previously reported. However, redundancy and physical separation give reasonable assurance that critical equipment located in these areas will perform their function. Controls for the critical equipment required for maintaining the plant in a hot shut-down condition are provided locally as well as in the control room.

In summary it is concluded that, although tornado requirements were not included in the design, there is reasonable assurance that public health and safety will not be endangered by a tornado passing through the plant site.

The appendix is organized in sections. Section 3A.1 includes the introduction and conclusions. Section 3A.2 gives a list of the systems required for maintaining the plant in a hot shut-down condition and the buildings in which they are housed. Section 3A.3 gives the status of the various areas of these buildings and indicates the critical components which are located in each. Section 3A.4 contains an analysis of the critical systems, the status of the components insofar as tornado effects are concerned, available redundancy and physical separation, and an overall conclusion on each system. Section 3A.5 deals in particular with the spent fuel pool (SFP) and the loss of pool water.

In drawing conclusions about a system or component vulnerability to tornadoes the following criteria have been adopted. A system or component is considered reasonably protected if:

1. The system or component is located inside a building which will not suffer damage from a tornado.
2. The system or component is located underground.
3. The system or component is located on a building floor that has one or more floors on top of it and is confined by other buildings.

4. The system or component is so designed and installed that no loss of function is anticipated even though the building in which it is housed might suffer damage or might be exposed to the weather. This might result from the fact that this system or component has a redundant system or component physically separated or protected such that failure of both systems or components from the same tornado effect is very unlikely.

The available redundancy gives reasonable assurance that time will be available for performing repairs on the redundant system or component.

3A.2 IDENTIFICATION OF CRITICAL SYSTEMS AND STRUCTURES

In order to maintain the plant in a safe hot shutdown condition, the following two functions must be performed:

1. Decay heat removal.
2. Reactivity control.

These systems are necessary in order to remove decay heat and control the core reactivity:

1. Steam relief system.
2. Auxiliary feedwater system.
3. Service water (SW) system.
4. Boration system.
5. Component cooling system.
6. Ventilation system.
7. Electrical system.
8. Instrumentation system.

The buildings which house the critical systems are

1. Auxiliary building.
2. Intermediate building.
3. Diesel-generator annex.
4. Screen house.
5. Control room.
6. Service building.
7. Cable tunnels.

3A.3 TORNADO EFFECTS ON STRUCTURES

3A.3.1GENERAL

All structures have been designed for wind loads in accordance with the requirements of the State of New York - State Building Construction Code. The wind loads tabulated in this code are based on a design wind velocity of 75 mph at a height of 30 ft above grade level. The stresses resulting from these loads were considered on the basis of a working strength design approach.

For purposes of this study the design of all critical structures has been checked on the basis of a limiting load factor approach wherein the loads utilized to determine the required limiting capacity of any structural element are computed as follows:

$C = (1.00 \pm 0.05)D + 1.0\ W_t + 1.0\ P_t$

Symbols used in this equation are identified as follows:

C =	required load capacity of section
D =	dead load of structure
W_t =	wind loads based upon 300 mph tangential wind velocity
P_t =	pressure load based upon an internal pressure 3 psi higher than the external pressure

3A.3.2REACTOR CONTAINMENT

Although tornado loads were not considered in the original design, this structure is capable of resisting the full strength tornado loads.

3A.3.3AUXILIARY BUILDING

Although tornado loads were not considered in the original design, this structure, up to and including the operating floor (elevation 271 ft 0 in.), is capable of resisting tornado loads. The siding on the superstructure would blow outward, thus relieving the pressure and wind load. Components and systems on the operating floor and above are susceptible to impact by falling debris and potential missiles. The equipment on the auxiliary building operating floor that is required to maintain the plant in a hot shutdown condition is as follows:

1. Boric acid storage tanks, pumps, and filter.
2. 480-V switchgear (bus 14).

The equipment in item 1 is surrounded by a radiological shield wall as shown in Figure 1. This wall offers significant lateral protection against potential missiles. Furthermore, the two tanks and pump are redundant. Therefore, there is reasonable assurance that there will be no loss of boration function. More details are given in Section 3A.4.2.

Damage to bus 14 will not cause loss of power supply since an independent and redundant bus (bus 16) is provided on the intermediate floor of the auxiliary building. This floor, as

previously mentioned, will not be exposed to the weather. More details are given in Section 3A.4.4.

In addition, the spent fuel pool (SFP) has been evaluated. Potential missiles may puncture the spent fuel pool (SFP) liner but will not penetrate through the concrete walls or base causing gross leakage of water.

3A.3.4INTERMEDIATE BUILDING

This structure, as shown in Figure 2, is significantly confined by other buildings, i.e., the service building, turbine building, reactor containment, and auxiliary building. Consequently, a direct exposure to a tornado funnel is extremely remote. Due to the relative vacuum which might be created by a tornado outside of the intermediate building lateral walls may blow outward. This will relieve the pressure differential and prevent gross failure of the structural steel framing, columns, and floors. Therefore, the two floors which house critical equipment, i.e., floors at elevations 253 ft 6 in. and 278 ft 4 in., are afforded significant shielding by the adjoining structures and higher floor/roof elevations.

The critical components in this structure consist of the following:

1. On floor elevation 253 ft 6 in.: two motor-driven and one turbine-driven auxiliary feedwater pumps.
2. On floor elevation 278 ft 4 in.: the cross-connection on main steam and feedwater lines to the two steam generators.

As previously mentioned, no damage is anticipated to the equipment located on these two floors. More details are given in Section 3A.4.1.

3A.3.5DIESEL-GENERATOR ANNEX

The availability of onsite diesel power was reviewed on the basis of the assumption that the tornado could cause a loss of offsite power.

Siding, windows, doors, and ventilation openings would blow outward, thus relieving the pressure loading. Damage to the roof might result if the differential pressure is not relieved in time. Two redundant diesel generators are provided. No physical damage to the diesels is anticipated. Furthermore, the physical separation between them is such that one missile would not be able to impact against both diesel generators, as shown in Figure 3. More details are given in Section 3A.4.4. The conclusion has been drawn that the emergency power supply is reasonably ensured.

3A.3.6SCREEN HOUSE

Siding, windows, doors, and ventilation openings would blow outward, thus relieving the pressure loading. No structural collapse is expected. The critical equipment housed in the screen house is represented by:

1. Four service water (SW) pumps.
2. 480-V switchgear buses 17 and 18.

The four service water (SW) pumps are redundant and sufficient physical separation exists between them to make extremely unlikely the failure of all four pumps from the same tornado effect, as shown in Figure 4.

Service water (SW) pumps 1A and 1C are energized from bus 18 and service water (SW) pumps 1B and 1D are energized from bus 17. Cross-tie between the two buses is available.

The two buses are located in the screen house and are physically separated. Therefore, there is reasonable assurance that at least one service water (SW) pump-bus combination will operate properly. More details are given in Section 3A.4.4.

3A.3.7CONTROL ROOM

No gross failure of this structure is anticipated. The only wall directly exposed is the east wall. The siding of this wall would blow outward relieving the pressure differential and leaving the interior exposed to the weather. The same would be true for windows, doors, and ventilation openings.

Local controls for the equipment required for maintaining the plant in a hot shutdown condition have been provided as a backup to the controls available in the control room. Therefore, there is reasonable assurance that controls for the critical components will be available.

3A.3.8SERVICE BUILDING

The status of this building is similar to that of the auxiliary building. The siding on the superstructure above elevation 271 ft would blow outward, thus relieving the pressure and wind loads. The components which might be affected by a tornado are the two condensate storage tanks (CST). There is reasonable assurance that the feedwater supply will be maintained because of the available redundancy and the fact that two-thirds of the tank volume is below grade.

3A.3.9CABLE TUNNELS

The cable tunnels are located underground and are capable of withstanding tornado loads.

3A.4 TORNADO EFFECTS ON THE SYSTEMS REQUIRED FOR HOT SHUTDOWN

3A.4.1DECAY HEAT REMOVAL

With the plant in a hot shutdown condition, decay heat is removed via the steam generators. In order to achieve this heat transfer, water has to be supplied to the secondary side of the steam generators and steam has to be discharged from them. For this function to be performed, it is necessary to have a source of feedwater, pumps to transfer the feedwater from the tank to the steam-generator secondary side, and steam relief from the steam generators.

3A.4.1.1Steam Relief System

Since a tornado could cause a loss of offsite power, condenser vacuum could not be maintained to allow steam discharge to the condenser. The only available route would be to the atmosphere.

On the steam pipe associated with each steam generator, outside the containment, are four steam relief valves (12.5% of full flow per valve). Figure 5 shows the location of these valves. Significant valve redundancy is available since no more than 2% full flow capacity would be needed a few seconds after shutdown.

The relief valves are located inside the intermediate building and the two sets have a minimum distance between them of 35 ft. Since the valves are relatively heavy steel, they are expected to withstand the effect of falling debris without physical damage.

The centerline of the pipe on which they are installed is at elevation 281 ft 4 in. The bulk of the steam piping is located in the intermediate building, with the exception of a run of the main steam line from steam generator B. This steel pipe being relatively thick-walled, it is also expected to withstand falling debris without sustaining serious damage.

Because of the inherent physical strength of the equipment involved, its redundancy and physical separation, it can be concluded that the steam relief function is ensured.

3A.4.1.2Auxiliary Feedwater System

This system consists of

1. One auxiliary steam-driven feedwater pump.
2. Two auxiliary motor-driven feedwater pumps.
3. Two condensate storage tanks (CST).

The steam-driven pump has the capacity of supplying water to either or both steam generators. This pump is located in the intermediate building on the northwest side at 253 ft 6 in. floor elevation. Local shielding is provided as shown in Figure 6.

The two motor-driven pumps are also located in the intermediate building on the northwest side at 253 ft 6 in. floor elevation. Each pump is sized for the water supply to one steam generator. Piping and valve arrangements allow flow to either of the two steam generators.

The distance between the shafts of the two motor-driven pumps is 8 ft, while the minimum distance between the steam-driven and the two motor-driven pumps is about 36 ft, as shown in Figure 6.

The preferred auxiliary feedwater lines from the condensate storage tank to the suction of the pumps are partly located below grade and partly inside the intermediate building. The preferred auxiliary feedwater lines from the discharge of the pumps to the steam generators are run at elevation 271 ft 0 in. before penetrating the containment.

The power supply to the motors of the two motor-driven pumps is from buses 14 and 16 located in the auxiliary building on the operating floor and on the intermediate floor, respectively.

Pump control is performed from the control room or from a local panel in the intermediate building at 253 ft 6 in. floor elevation.

The main source of water supply is by gravity feed from the condensate storage tank located in the service building on the southwest side at 253 ft 6 in. floor elevation. The feedwater suction is at 254 ft 10 in. floor elevation.

If the condensate storage tank water is not available, feedwater can be delivered to the suction of the preferred auxiliary feedwater pumps by the service water (SW) pumps. This system is described in Section 3A.4.1.3.

Because of the location of the intermediate building, the location of the required pumps, connecting piping, control and electrical cables, and redundancy and physical separation, it is concluded that the preferred auxiliary feedwater supply function is ensured.

3A.4.1.3Service Water System

This system is required for providing cooling to the emergency diesel generators and the containment ventilation system, as well as being an alternate source of preferred auxiliary feedwater. This system consists of the following components:

1. Four service water (SW) pumps.
2. Valves and piping.

Each of the four service water (SW) pumps is capable of carrying the emergency cooling load. These pumps are in the screen house, located about 115 ft north of the turbine building and about 80 ft south of the lake shore (Figure 2). The suction point from the lake water, associated piping, and valves are inside the building, below grade, in a reinforced-concrete structure. The service water (SW) piping which supplies water to the critical components is run underground from the screen house to the area being served.

Two pumps are connected to 480-V bus 18 and two to bus 17. In the event of loss of all outside power, bus 18 is energized by one diesel generator and bus 17 by the other one. Buses 17 and 18 are located inside the screen house. The electrical connections from the diesels to buses 17 and 18 are routed inside a separate underground duct bank from the diesel-generator annex building to the screen house.

Because of redundancy and physical separation, it is concluded that the function of the service water (SW) system is not jeopardized.

Additional redundancy of water supply which can be used instead of the condensate water and the service water (SW) is represented by the domestic water and the fire system.

The pumping station of the domestic water is located 2 miles away from the plant and the piping is routed underground to the plant itself. The two fire pumps, having a capacity of 2000 gpm minimum each, are located in the screen house. The time necessary for these two systems to operate is estimated to be approximately 10 minutes.

3A.4.2REACTIVITY CONTROL

3A.4.2.1Boration System

The reactivity control systems are required to make and hold the core subcritical following a tornado. After control rod insertion, shutdown capability is provided by boric acid injection to compensate for the long-term xenon decay transient. The system required for performing this function is the boration system.

This system is not required to operate immediately but after a period of at least 15 hr, i.e., the time required for the xenon to build up and then decay to the level present before shutdown. Therefore, ample time would be available for repair of the system.

The boration system includes the components listed below:

1. Two boric acid storage tanks.
2. Two boric acid transfer pumps.
3. One boric acid filter.
4. Three charging pumps.
5. Associated piping and cables.
6. Heat tracing.

The boric acid storage tanks, boric acid transfer pumps, and filters are located in the auxiliary building, northeast side, at elevation 271 ft. They are surrounded by a radiological shield wall, as shown in Figure 1. The siding of the auxiliary building above elevation 271 ft is not likely to withstand tornado winds or differential pressure; however, lateral protection is offered by the radiological shield wall. Furthermore, the boric acid transfer pumps and tanks have redundancy and physical separation. The three charging pumps are located on the basement floor of the auxiliary building and only one is needed for delivering the required flow. Connecting piping and control and power cables are all below the grating at 279 ft. Therefore they are protected from falling debris.

3A.4.2.2Boration Using Refueling Water

The reactor coolant system can be borated also by using refueling water. This boration process is slow because of the low boric acid concentration in the refueling water. As a result, a process of feed and bleed is required. For this, the following components are needed:

1. Volume control tank.
2. Associated piping.
3. Nonregenerative heat exchanger.
4. Refueling water storage tank.(RWST)
5. Component cooling system components.
6. Service water (SW) system components.

The volume control tank is located in the auxiliary building on the intermediate floor.

The letdown station, including the nonregenerative heat exchanger, associated piping, and cables, is located below the operating floor of the auxiliary building. The letdown station has a backup in the excess letdown line and excess letdown heat exchanger located inside the containment. The piping which connects the charging pumps to the volume control tank is located in the auxiliary building below elevation 253 ft 6 in. The refueling water storage tank (RWST) has approximately one-half of its volume below the operating floor of the auxiliary building.

Only if boration is performed by using the refueling water is the component cooling system necessary to provide cooling to the nonregenerative heat exchanger. The system includes the following components.

1. Component Cooling Pumps
2. Component cooling heat exchangers.
3. Component cooling surge tanks.
4. Component cooling valves and piping.

Two component cooling pumps are located in the auxiliary building, southeast side, on the operating floor at elevation 271 ft. The distance between the two shafts is about 10 ft. The same distance exists between the other components, as shown in Figure 7, Sheets 1 and 2.

Equipment/system redundancy and separation and the ample time available for repair give reasonable assurance that the boration function can be performed.

3A.4.3CONTAINMENT VENTILATION SYSTEM

In order to guarantee the satisfactory operation of the instrumentation and control systems required for hot shutdown, the containment air temperature must be maintained at a tolerable level. The ventilation system again requires operation of the service water (SW) and electrical systems. Electrical power is supplied separately from bus 14 to two fans, and from bus 16 to the other two fans via an underground tunnel. Local control is performed by four transfer switches and pushbuttons located in the intermediate building at 253 ft 6 in. floor elevation.

Redundancy and physical separation ensure the containment ventilation function.

3A.4.4 EMERGENCY POWER SUPPLY SYSTEM

The emergency power supply system includes the following components:

1. Switchgear.
2. Emergency buses.
3. Two diesel generators.
4. Two diesel fuel-oil storage tanks.

Switchgear and emergency buses at 480 V are needed to carry the power for previously mentioned components. (Two redundant emergency buses are provided.) Bus 14 is located in the auxiliary building, on the operating floor, and bus 16 is on the intermediate floor of the same building. Buses 17 and 18 are located in the screen house, as mentioned in Section 3A.4.1.3.

Each of the two diesel generators is able to supply the emergency power through an independent system of buses (buses 14 and 18 to one diesel generator, and buses 16 and 17 to the other diesel generator).

Credit is not taken for redundancy from offsite power supply since total loss of offsite power has been assumed. This is an extremely conservative assumption since the switchyard is located 2500 ft south of the plant and the connecting cables are run underground.

The two diesel fuel-oil storage tanks are 6 ft below grade and the pipelines run underneath the hydrogen storage building.

Redundancy and physical separation give reasonable assurance that emergency power will be supplied.

3A.4.5 CONTROL SYSTEM

3A.4.5.1 Control Room

The shutdown operations are directed from the control room located on the southeast side of the turbine building. A redundant means of maintaining the plant in hot shutdown condition is provided by local control of the vital components and a local panel that displays the steam generator and pressurizer level and pressure, as described in Section 3A.4.5.3.

3A.4.5.2 Systems of Batteries

Each of the two separate systems is able to carry its expected shutdown load. Their location is two floors below the control room.

Cables feeding the dc loads are protected since one cable unit runs in an underground duct bank.

3A.4.5.3 Steam-Generator Level and Pressure Indicators, Pressurizer Pressure and Level Control

A local panel in the intermediate building gives indication of the above instruments. Cables from the containment to the panel are run in an underground tunnel.

Proper functioning is also required for the following components which are protected from tornado loads:

1. Communication network between the local panel and the components which have to be operated (boric acid transfer pumps and charging pumps) is provided by a paging system.
2. Pressurizer heaters are required to maintain heat removal by natural circulation. Their location is inside the containment; thus, no additional protection is required. The power supply is from buses 14 and 16 in the auxiliary building. Local control is provided on the intermediate floor of the auxiliary building.

Redundancy, physical separation, and proper location give reasonable assurance that the plant will be under control during and after a tornado.

3A.5 TORNADO EFFECT ON SPENT FUEL POOL

The spent fuel storage pool is located in a structure attached to the end of the auxiliary building and adjacent to the containment, as shown in Figures 8 and 9. The pool itself is 43 ft long, 22 ft 3 in. wide, and 40 ft 3 in. deep. The superstructure could blow outward as a result of a direct hit by a tornado. No damage would result to the pool itself, however, because of the thick concrete biological shield walls and the location of the pool at a low elevation.

The only conceivable means for water loss would be due to action of tornado winds on the pool during the incident time interval.

It is possible for tornado action to cause a partial water loss. It is somewhat speculative, however, that tornado action could empty a pool of the depth and restricted dimensions of the spent fuel pool (SFP). Because of the depth of the water in the pool and friction on the walls of the pool and the spent fuel assemblies and their storage racks, it is not anticipated that a tornado could completely uncover the fuel assemblies. Approximately 68% of the pool water could be removed without uncovering the top of the fuel assemblies. Assuming water remains only at the top level of the fuel assemblies and that the pit holds one-third of a core which has decayed 1 week after refueling, it would take over 3 hr to heat the water from its normal temperature to 212°F.

The assumed heat load would cause the water to boil off at a rate of 3.2 in./hr. Thus it would take approximately 18 hr before the level could reach the midplane of the assemblies.

The two fire pumps can be arranged to reflood the pool at a rate of 6 in./min.

APPENDIX 3B

DESIGN OF LARGE OPENING REINFORCEMENTS FOR CONTAINMENT VESSEL

SEPTEMBER 6, 1968   GAI REPORT NO. 1683

R. E. Ginna Nuclear Power Station

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Including

3B Appendix A	EFFECT OF CONCRETE CREEP AND THE SUSTAINED OPERATING STRESSES ON STRESS DISTRIBUTION AROUND OPENINGS IN A RAPIDLY PRESSURIZED REINFORCED CONCRETE VESSEL
3B Appendix B	EARTHQUAKE ANALYSIS
3B Addendum	ADDENDUM TO THE REPORT ON: DESIGN OF LARGE OPENING REINFORCEMENTS FOR CONTAINMENT VESSEL

TABLE OF CONTENTS

SUMMARY

I. DESIGN BASES

The large openings consisting of a 14 ft 0 in. diameter opening (equipment access hatch) and a 9 ft 6 in. diameter opening (personnel lock) are designed generally in accordance with the criteria outlined in Section 5.1.2.3 of the Final Description and Safety Analysis Report. The only substantive difference relates to the fact that the principal mild steel reinforcement used in the vicinity of the openings has a 60,000 psi, in lieu of 40,000 psi, yield stress. There is no mix of rebar types for principal reinforcement in the high stress area.

II. GENERAL DESCRIPTION

The vertical prestressing tendons are draped around the holes and are continuous (i.e., no tendons are terminated at the hole). Elliptical rebar rings are located around the hole as principal reinforcement. Where practical, hoop rebars are draped around the opening. Radial rebars are provided to the extent required by calculated radial (to the hole) stresses. Normal shear forces, due to pressure on the hatch, at the interface between penetration barrel and concrete are resisted by steel shear rings.

III. STRESS DISTRIBUTION AROUND A CIRCULAR HOLE IN A CIRCULAR CYLINDRICAL SHELL

As described in more detail in Section 3 of GAI Report No. 1683 (hereafter called the report), a survey was made of the available theoretical solutions and experimental techniques for determining stress distributions around circular holes in a shell structure. This survey indicated that an available programmed finite-element solution was superior for this application.

IV. ANALYSIS OF STRESSES AROUND LARGE OPENINGS

A. Method of Analysis

The adequacy of the design of the openings was verified by the use of Computer Program FELAP developed at Franklin Institute Research Laboratories (FIRL). This program provides for the representation of the shell by flat rectangular panels with multiple layers and is used on the assumption that the perturbation on the shell introduced by the presence of the opening is local. The basis for the derivation of this program is described in Reference 13 to the report.

B. Verification of Method Accuracy

In order to evaluate the accuracy of the solution method, a test problem was solved to develop stresses which could be directly compared with other theoretical solutions and experimental results. Very satisfactory results were obtained as evidenced by the data presented in Section 4.1 of the report. This study further verified the adequacy of the grid used on the analysis of the openings for the R. E. Ginna Containment Vessel.

C. Basis for Analytical Model

For purposes of the analysis the shell was idealized by representing (1) the liner as an isotropic steel layer, (2) the horizontal reinforcement as an orthotropic layer with no Poisson's ratio effect and no shear or meridional stiffness, and (3) the elliptical ring

reinforcement with zero shear stiffness, the hoop stiffness varies from zero along the horizontal axis of the opening to the maximum value along the vertical axis and the meridional stiffness varies in the converse manner. A more complete description of the model idealization is contained in Section 4.3.1 of the report.

The effect of concrete cracking was established by setting the stiffness coefficient normal to the crack equal to zero. This procedure was followed for the factored accident pressure of 90 psi and no thermal effect which is the most unfavorable condition. A more detailed description of the cracking criterion and its application is contained in Section 4.3.3 of the report. Sufficient steps in crack development were evaluated to confirm reasonable convergence requirements. The significant conclusion was reached that the distribution of stress-resultants and stress-couples was rather insensitive to such variations as changes in shear modulus of cracked concrete as well as the degree of cracking.

D. Load Combinations

The basic loads including dead, internal pressure, earthquake, prestress, thermal (operating and accident) loads were combined in accordance with the basic criteria and as more fully tabulated in Table 4-1 of the report. An additional possible loading condition investigated involved the stress redistribution resulting from the hypothesized loss of bond along the horizontal rebars terminated near the edge of the opening. All loading conditions are more fully described in Section 4.3.2 of the report.

E. Non-Linear Effects

The effect of load redistribution due to concrete shrinkage and creep was investigated as described in Section 4.3.4 of the report and found to be negligible.

F. Summary of Results

The stress-resultants and stress-couples for panels which were found to be of interest in evaluating the adequacy of the design are summarized in Tables 4-2 and 4-3. The correctness of the values computed by the finite-element method was verified in several ways, as described in Sections 4.1 (c) and 4.3.3 of the report.

V. VERIFICATION OF REINFORCEMENT ADEQUACY

A. General Method

Principal stress-resultants and stress-couples were computed and found to be colinear or essentially so for all panels which were significant in the design check. Likewise the orientation of stress-resultants and stress-couples was found to essentially coincide with the mild steel reinforcement for all significant panels. Interaction diagrams were prepared based upon procedures for ultimate strength design of ACI 318-63. Interaction diagrams for those panels found to be significant are included as Figures 19 through 24 of the report indicating the stress state for all pertinent loading combinations. The interaction diagrams show that sufficient reinforcement has been provided to carry all loads, including the full thermal stress-resultants and stress-couples.

B. Additional Considerations

Additional studies were performed to evaluate the acceptability of liner stresses (assuming total composite action), penetration barrel, reinforcement for normal shears, mechanical

anchorages for terminated rebar and tendon friction losses all of which are described in Sections 5.4 through 5.9 of the report.

C. Design Drawings

Design drawings providing details of the opening reinforcement are included at the end of the report.

1. DESIGN BASES

1.1GENERAL

The large openings in the Containment Vessel are designed generally in accordance with the criteria outlined in Section 5.1.2.3 of the Robert E. Ginna Nuclear Power Plant Final Description and Safety Analysis Report (FSAR), and as more completely detailed hereafter. The large openings consist of the opening for the equipment access hatch with a diameter of 14 ft 0 in. and the opening for the personnel lock with a diameter of 9 ft 6 in. Although these criteria and the analytical and design methods described hereafter apply to both openings, detailed results are provided only for the larger opening.

1.2DESIGN LOADS

The following loads were considered in the structural design of the Containment Vessel:

a. Test Pressure - 69 psig (1.15 times design pressure)

b. Accident Pressure - Design Pressure is 60 psig

c. Thermal Loads

   1. Accident

   2. Operating

   3. Test

d. Seismic Ground Accelerations

e. Dead Load

f. Prestressing Load

The thermal loads on the Containment Vessel and their variation with time are developed on the basis of the blow-down transients shown in Figures 14.3.4-2 and 14.3.4-3 of the FSAR. The openings as a portion of a Class I structure are designed on the basis of a ground acceleration of 0.08g acting in the horizontal and vertical planes simultaneously as the design earthquake and of 0.20g acting in the horizontal and vertical planes simultaneously as the maximum hypothetical earthquake.

The equipment access hatch consists of a single door located outboard of the containment shell with a personnel lock inset in the door. The barrel (penetration sleeve) for the hatch is consequently subjected to the accident pressure. The isolated personnel lock consists of double doors, one located inboard and the other outboard of the containment shell. The design considers the consequence of both doors being closed during the accident as well as the extremely remote possibility that either one of the doors is opened during the accident.

1.3LOAD COMBINATIONS

The design is based upon limiting load factors which are used as the ratio by which dead, accident, and earthquake loads are multiplied for design purposes to ensure that the load

deformation behavior is essentially elastic. The loads utilized to determine the required limiting capacity of any structural element are computed as follows:

a. $C = (1.00 \pm 0.5)D + 1.5P + 1.0T$

b. $C = (1.00 \pm 0.5)D + 1.25P + 1.0T' + 1.25E$

c. $C = (1.00 \pm 0.5)D + 1.0P + 1.0\,\underline{T} + 1.0E'$

Symbols used in the above equations are defined as follows:

C: Required load capacity of section

D: Dead load of structure

P: Accident pressure load - 60 psig

T: Thermal loads based upon temperature transient associated with 1.5 times accident

T ′: Thermal loads based upon temperature transient associated with 1.25 times accident pressure.

$\underline{T}$: Thermal loads based upon temperature transient associated with accident pressure.

E: Seismic load based upon 0.08g ground acceleration.

E ′: Seismic load based upon 0.20g ground acceleration.

Refer to Section 5 1.2.4 of the FSAR for acceleration response spectra and structural damping.

The maximum temperature within the Containment Vessel under normal operating conditions will be 125°F.

The equipment access hatch is housed outside of the containment shell within an unheated concrete enclosure provided for biological shielding. It is assumed that the minimum ambient temperature within this enclosure is -10°F. The isolated personnel lock is housed outside of the containment shell within the Intermediate Building. It is assumed that the minimum ambient temperature within this building is 50°F.

For Load Combination "a" (i.e., $C = 0.95D + 1.5P + 1.0T$), the maximum temperature of the inner surface of the liner (inner face of the metal sheet covering for the insulation where the liner is insulated) is calculated to be 312°F. For this load combination the ΔT of the liner where it is insulated at the time of maximum accident pressure is calculated to be 2°F, although the design is based upon a ΔT of 10°F.

1.4MATERIAL STRESS/STRAIN CRITERIA

a. Concrete Reinforcement

The deformed bars used for concrete reinforcement in the Containment Vessel are normally intermediate grade billet steel conforming to either ASTM A15-64 or A408-62T, depending upon the bar size, with a guaranteed minimum yield strength of 40,000 psi. Where required due to stress concentrations in the vicinity of the large openings, the deformed bars used for

concrete reinforcement are made from new billet steel, conforming to ASTM A432-66 with a guaranteed minimum yield strength of 60,000 psi.

The design limit for tension members (i.e., the capacity required for the factored loads) is based upon the yield stress of the reinforcing steel. No mild steel reinforcement is designed to experience strains beyond the yield point at the factored loads. The load capacity so determined is reduced by a capacity reduction factor "ϕ" which provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in an under-capacity. The coefficient "ϕ" is 0.95 for tension, 0.90 for flexure, and 0.85 for diagonal tension, bond and anchorage.

b. Prestressing Tendons

The steel tendons for prestressing consist of 90 1/4-in. diameter wires using a BBRV anchorage system and high tensile, bright, cold drawn and stress-relieved steel wires conforming to ASTM A421-59T, Type BA, "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete" with a minimum ultimate tensile stress of 240,000 psi.

The steel tendons are stressed during the post-tensioning operation to a maximum of 80 percent of ultimate strength and locked-off for an initial stress of 70 percent of ultimate strength. The maximum effective prestress is determined taking into consideration allowances for the following losses, which are deducted from the transfer prestress:

1. Elastic shortening of concrete
2. Creep of concrete
3. Shrinkage of concrete
4. Steel relaxation
5. Frictional loss due to intended or unintended curvature of the tendons

In no event does the effective prestress exceed 60 percent of the ultimate strength of the prestressing steel or 80 percent of the nominal yield point stress of the prestressing steel, whichever is smaller. The design is based upon the steel tendons not being stressed beyond the yield point as defined by ACI 318-63 when subjected to the factored loads.

c. Structural Concrete

The structural concrete will have a minimum compressive strength of 5,000 psi in 28 days. Under operating conditions the allowable concrete stresses are in accordance with ACI 318-63 Part 1V-A, "Structural Analysis and Proportioning of Members-Working Stress Design," and Part V, Chapter 26, "Prestressed Concrete."

The Containment Vessel, including the large openings, is checked for the factored load combinations and compared with what is generally defined as the yield strength of the structure. For concrete, the yield strength is defined except as described hereafter by the allowable stresses given in ACI 318-63, Part IV-B, "Structural Analysis and Proportioning of Members - Ultimate Strength Design." Concrete cracking is assumed when the principal tensile stress based upon all loads including thermal loads exceeds $6\sqrt{f'_c}$ or 424 psi.

d. Liner

The liner is designed as participating with the concrete shell in carrying membrane forces (See 5.1). The stress limits established for the liner are consistent with those limits for stress intensity (i.e., the difference between the algebraically largest principal stress and the algebraically smallest principal stress at a given point) defined in Section III of the ASME Boiler and Pressure Vessel Code and based upon a working strength design consistent with that code.

The liner is carbon steel plate conforming to ASTM A442-60T Grade 60 modified to ASTM A300. This material has a minimum yield stress of 32,000 psi and a minimum ultimate tensile strength of 60,000 psi. The liner plate is normally 3/8 in. thick in the cylinder portion of the structure. In the immediate vicinity of the large openings, this liner plate is reinforced with a doubler plate. The barrel and doubler plate are carbon steel conforming to ASTM A516 Grade 60 Firebox Quality modified to ASTM A300. This material has a minimum yield stress of 32,000 psi and a minimum ultimate tensile strength of 60,000 psi.

1.5TEST CONDITION

No specific stress or strain limits are established for the test condition. The factored load combinations previously described have been established so as to ensure that the response due to design loads is essentially elastic. A check is made to ensure that no significant permanent deformation of the structure occurs during the test. This means that following the test there should be no visible permanent distortion of the liner and only small hairline cracks should exist in the concrete.

1.6OPERATING CONDITION

The load combinations relevant to operating conditions are determined as reflected in Section 4.3.2 of this report. Allowable stresses are based upon those stipulated in ACI 318-63.

2. GENERAL DESCRIPTION OF OPENING REINFORCEMENT

2.1INTRODUCTION

The normal flow of stresses in the hoop and meridional direction is obstructed by the large access holes in the Containment Vessel. During normal operation of the plant, vertical prestress forces are the single largest stress contributor. These forces do not create any difficulty in transferring vertical compressive stresses around the opening, but give the necessary reserve compressive stresses to counterbalance the vertical strains resulting from the accident pressure load. The horizontal stresses in the hoop reinforcing steel are very small during normal operation, but will theoretically increase up to almost yield stress due to the factored pressure load. These hoop forces will be transferred around the opening by draped continuous hoop reinforcing steel, and elliptical ring reinforcing steel.

2.2REBAR FOR DISCONTINUITY STRESSES

As seen from Drawing No. D-421-023 (Figure Drawing 2), some of the hoop reinforcing bars are terminated at the opening. The tensile load in these bars is transferred to the concrete by bond, or through the end anchor plates, or a combination of both. The effect of these different conditions was carefully investigated.

The elliptical ring reinforcing steel will be spliced by use of a Cadweld Rebar Splice. These splices are located at points of low rebar stresses, and no more than 1/3 of the ring steel will be spliced in one section. This should eliminate any slip between Cadweld and rebar that could occur at high rebar stresses.

Radial tensile stresses out from the center of the access opening are created due to the pressure load. Radial reinforcement is provided to carry these stresses out from the opening and will be terminated where "pure" membrane stresses exist in the wall.

2.3NORMAL SHEAR AT EDGE OF OPENING

The peripheral or normal shear reinforcement in the concrete around the Penetration Barrel is designed for the computed shear at equal distance or greater than, d/2 = 33.0 in., from the edge of the opening per ACI 318-63, or for twice the normal shear due to internal pressure on the hatch, whichever is the larger of the two values.

2.4PRESTRESSING

The vertical tendons will be curved around the opening, using a minimum radius equal to 20.0 feet. The total angular change of the tendons was laid out to keep the frictional losses within a satisfactory margin, and to satisfy a practical execution of the job in the field. However, it will be required to retension all the curved tendons around the large access openings approximately 1000 hours after the initial stressing. The minimum effective prestress after 40 years, i.e., the lifetime of the plant, will then be met.

3. STRESS DISTRIBUTION AROUND A CIRCULAR HOLE IN A CIRCULAR CYLINDRICAL SHELL

3.1INTRODUCTION

The first theoretical treatment of the problem presented by openings in thin shell structures is commonly attributed to Lur′e[1], who obtained an approximate solution for the stress distribution around a very small circular cut-out in a thin circular cylindrical shell subjected to a homogeneous biaxial stress state σ_x′ σ_θ. Lur′e derived the following expression for the edge stress:

$$\sigma = (\sigma_x + \sigma_\theta) - 2(\sigma_x - \sigma_\theta)\cos 2\phi$$

$$+\sqrt{3(1-\nu^2)}\ \frac{\pi d^2}{16Rt}\ [2\sigma_x + (\sigma_\theta - 3\sigma_x)\cos 2\phi]$$

(Equation 3.1)

in which:

σ_x = meridional stress in the shell without the opening

σ_θ = hoop stress in the shell without the opening

ϕ = angular coordinate (see Fig. 2)

R = shell radius

t = shell thickness

d = diameter of cut-out = 2r

Equation (3.l) does not include bending effects, which can be quite important in thin shells. Therefore, Lur′e's solution represented a relatively minor the flat plate solution[2]. In fact, for many years the design of reinforcement around shell openings was based on the classical stress concentration factors of flat plate theory. In this area solutions for openings reinforced by means of a symmetrical circular doubler plate were obtained by Sezawa and Kubo[3], Gurney[4], and Beskin[5]. Solutions for the stress distribution around unreinforced and ring-reinforced holes in flat plates can be found in Savin's[6] extensive treatment of the subject.

The plane stress approach is not satisfactory, however, for large openings. In this case large stress-couples may appear around the edge of the opening even when the shell elsewhere is in a pure membrane state of stress. Fortunately, very valuable results have recently become available. Withum[7] investigated the stress distribution in a cylinder weakened by a hole, subjected to torsion, by using a perturbation scheme. This technique was extended by Kline 'et al'[8] to determine the stresses around a circular cut-out in a pressurized circular cylindrical vessel. This work, carried out at the General Technology Corporation with the support of the Bureau of Ships of the U. S. Navy, was part of a systematic theoretical investigation of two problems:[9,10]

a. determination of the state of stress in the vicinity of a circular hole on a circular cylindrical shell subject to internal pressure

b. determination of the stress distribution in two normally intersecting cylindrical shells

The solution of problem (a), for internal pressure as well as other practical loading conditions, has been presented by Naghdi and Eringen[10]. Lekkerkerker[11], extending Lur′e's approach, solved the same problem for axial tension and torsion. They found excellent correlation between their solution and experimentally determined stresses (using electric resistance strain gages) in a mild steel tube subject to torsion. Stress concentration factors from Ref (8) to (11) are presented in Figures 1 and 2. The coefficients given may be directly applied to the calculation of maximum stresses at the edge of the opening.

Unfortunately, no theoretical solution is available for reinforced openings or for non-isotropic shells (e.g., orthotropic reinforced concrete shells). The complexity of these problems is such that they can only be dealt with by means of numerical or experimental methods. Among the many possible approaches, the finite-element method and the stress-freezing technique of three-dimensional photoelasticity have become especially attractive in recent years and are briefly evaluated in Sections 3.2 and 3.3.

3.2FINITE ELEMENT METHOD

Steady progress in the finite element approach has led to the possibility of determining the stress distribution around reinforced openings in shell structures with good accuracy. The most important advantage of this method is its generality: reinforcing rings, variable shell thickness and material orthotropicity, for example, may be incorporated into the analysis without difficulty. Unfortunately, the accuracy of the solution must still be evaluated by test: for instance, results obtained using two different grid may provide an indication of convergence. Alternatively, results obtained using a prescribed grid size and pattern may be checked against some of the theoretical solutions of Section 3.1 or against experimental values. The latter approach was followed in connection with the analysis of the stresses around the openings for the R. E. Ginna Containment Vessel.

The designs of the reinforcement around the openings in the R. E. Ginna Containment Vessel were verified by using Computer Program FELAP[12], developed at Franklin Institute Research Laboratories (FIRL) The solution is based on a representation of the shell as an assembly of flat rectangular panels. In the first order shell theory described by Zudans[13], which formed the basis for the FIRL finite element solution, the rotation about the normal to the shell middle surface is taken equal to zero. Consequently, this leaves only five degrees of freedom associated with each nodal point. It must be noted that the model with five degrees of freedom at a corner point, while "compatible" for plate problems, is unbalanced for shell problems in the third rotation[14]. Although usually this unbalance does not affect the accuracy of the solution, it can lead to unrealistic results[14]. Therefore, in applications to nuclear power plants, a verification of the results becomes desirable.

Connor and Brebbia[15] developed a stiffness matrix for a thin shell element of rectangular plan and also noted that good results can be obtained with the finite element method using

noncompatible displacement expansions, which do not include all the rigid body displacements[15]. According to Connor and Brebbia[15], it appears that curved elements lead to better results, for the same element size and shape, and are therefore more efficient than flat elements. However, comparison of results obtained using two different types of curved triangular elements[16,35] with flat elements[12,14] for the test problem discussed in Section 4.l did not substantiate that belief (See Figure 9). In fact, all finite-element results were close to Eringen, Naghdi, and Thiel's[9] solution, which can be considered, within the limitations of shallow shell theory, an "exact" solution.

3.3APPLICATIONS OF THREE-DIMENSIONAL PHOTOELASTICITY

Until very recently, experimental methods constituted the only feasible approach to the stress analysis of geometrically complicated reinforced openings in shell structures. In this area, the stress-freezing technique of three-dimensional photoelasticity appears to be the most suitable experimental method. Outstanding studies of stresses around reinforced openings in pressure vessels were carried out by Leven[17], Taylor and Lind[18], and Takahashi and Mark[22]. In the latter, comparison of the photoelastic results with a finite element analysis of the axisymmetric thick-walled reinforced concrete vessel showed very close agreement between the two solutions. Durelli, del Rio, Parks, and Feng[19] carried out an experimental evaluation of the stress around an opening in a thin shell by means of brittle coatings, electrical and mechanical strain gages, micrometers, and photoelasticity with the objective of comparing the accuracy and advantages of each technique.

Photoelasticity was concluded to be the most effective experimental approach in this type of problem[19]. In the fabrication of the model Durelli 'et al'[19] used a Hysol 4290 epoxy resin which was found to give poor performance in recent tests[20]. Mark and Riera[20] believe that the use of improved model materials will lead to a considerable reduction in the dispersion of photoelastic readings, which was large in the past[25], and which still is the most important argument against this experimental approach. In spite of the difficulties associated with the model material, the photoelastically determined stresses in Reference (19) show good correlation with Eringen and Naghdi's[9] theoretical solution. Stress concentration factors determined photoelastically by Durelli 'et al' and by Houghton and A. Rothwell[21] by means of electric resistance strain gages mounted on circular cylindrical aluminum shells are given in Figure 3C. Figure 3C also shows the stress concentration factors computed by Eringen and Naghdi[9].

The photoelastic approach presents several advantages[24]: 1) full field observations give clear understanding of overall behavior and permits recognition of unsuspected critical regions, 2) measurements are made with very small effective gage lengths so that high gradients can be studied in small models, and 3) basic instrumentation is simple and inexpensive. Traditionally, models have been machine finished, but improving casting techniques have already permitted the fabrication of complicated models without any noticeable edge effect. This represents two important steps forward: 1) model fabrication cost can be drastically reduced and 2) the technique may now be applied to complicated models that cannot be readily machined.

Ducts for prestressing "tendons" have successfully been incorporated into epoxy models by using nylon piano cords, which are set in the model prior to casting as if they were rebars in a conventional reinforced concrete element[23]. After the epoxy has hardened, they can be easily pulled out, leaving a perfect duct without residual stresses around the walls. The photoelastic method, therefore, offers the possibility of determining in one single study the stresses around openings in prestressed vessels in "the large", as well as in "the small". The latter, which includes stresses around curved tendons, corners, possible non-linear distributions through the wall thickness, etc., may demand separate analyses when a solution based on thin shell theory is used as the basis for design. On the other hand, in applications for design purposes, the photoelastic method presents some drawbacks: 1) a procedure to include the liner and steel rebars into the model has not yet been proposed. 2) the approach is not adequate for vessels that may be in a cracked condition, such as the R. E. Ginna Containment Structure, which has only vertical prestress (See Section 4.2). Strictly speaking, the method is applicable only to fully prestressed vessels in which virtually no cracking due to high concrete tensile stresses is expected.

Further progress in theoretical or numerical analysis of stresses around shell openings, which must account for non-linear distributions through the shell thickness near the opening and for other effects that cannot be predicted by thin-shell theory, can be fostered by adequate experimental verification of the results, or by purely experimental studies that may help define the areas that require additional investigation. This is illustrated by a photoelastic investigation of stresses around reinforced openings in plates due to Lerchenthal[27], which indicates that the departure from a plane stress distribution near the openings may be significant. For this purpose, the stress-freezing technique may yet prove to be an invaluable tool.

4. ANALYSIS OF THE STRESSES AROUND LARGE OPENINGS IN THE R. E. GINNA SECONDARY CONTAINMENT VESSEL

4.1VERIFICATION OF FINITE-ELEMENT METHOD OF ANALYSIS

As outlined in Section 3.1, a finite element solution was chosen to determine stresses around large openings in the containment vessel. The selection of this approach was based on an overriding consideration: it constitutes the only method by which the steel reinforcement around the opening as well as the orthotropic character of the shell in the cracked condition (see Section 4.2) can be taken into account. To evaluate the accuracy of the solution method (FIRL Program FELAP, as described in Section 3.2), a test problem was solved with the same grid used in the finite-element analysis of openings for the R. E. Ginna Containment Vessel. The grid is shown in Figure 4. In the finite element analysis the following assumptions were made:

a. The perturbation introduced by the presence of the opening on the shell state of stress is localized.

b. According with (a), stresses and displacements some distance away from the region surrounding the opening are not affected by the opening.

c. A panel of rectangular plan, which is centered around the opening, is removed from the shell (Figure 4). The displacements corresponding to the shell without the opening along the boundary lines of this panel constitute the boundary conditions for the finite-element analysis. The analysis is correct if the panel boundaries are sufficiently far from the opening so that (a) and (b) apply. This assumption can be verified 'a posteriori' by comparing stresses along the boundary lines with those existing in the shell without the opening.

d. Because of symmetry, only one quadrant need be analyzed.

Since reliable experimental or theoretical results for reinforced openings in shells similar to that under consideration were not available, it was decided to check the solution method against the shell tested by Durelli 'et al'[19], for which other theoretical solutions were also available. The problem is defined by:

R =	430.00 in.
r =	85.70 in.
t =	18.04 in.
ν =	0.30
p =	100.00 psi

The computed tangential membrane stresses around the opening edge are compared in Figure 5 with those given in Ref. (9) and with the experimental stresses determined by Durelli 'et al'[19] Similarly, Figure 6 shows the tangential surface stresses around the edge. Additional comparisons between the finite element solution and the experimental values are given in Figures 7 and 8. Finally, Figure 9 shows the variation along the symmetry axes of the stress resultant N_θ determined by different approaches, including another finite-element solution

using triangular shell elements based on Prato's work[16]. The correlation of the FIRL finite element solution with the other results is good. It must be noted that the finite-element results are particularly close to the solution of Eringen 'et al', which was regarded as the most accurate. It should also be pointed out that the correlation is good in spite of the fact that the panel boundaries were not sufficiently removed from the opening, as revealed by Figure 8 which

shows the existence of a small stress couple M_θ at $\frac{s}{r} = 3.5$ that had not been predicted by the model study. This result was verified by another finite-element analysis of the test problem using Prato's triangular shell elements[16].

It was pointed out in Section 3.1 that the stress concentration factors under loading conditions such as internal pressure, axial tension or torsion can be computed at different locations in terms of the nondimensional parameter (See Figure 2). As $\nu \rightarrow 0$ we approach the plane stress solution and the convergence problem for the finite-element solution disappears. It can be concluded, therefore, that satisfactory results in the test problem ($\nu = 1.17$) constitute adequate verification of the solution method for the large openings in the R. E. Ginna Containment Vessel for which $\nu = 0.62$ (equipment hatch, based on typical shell thickness).

4.2GENERAL CONSIDERATIONS CONCERNING METHODS OF ANALYSIS OF REINFORCED CONCRETE STRUCTURES IN THE CRACKED CONDITION

The stress distribution in reinforced or prestressed concrete shell structures subject to cracking may be approximately determined by analyzing the shell with appropriately reduced stiffness coefficients. Milekykovsky and Hedgren and Billington[29] used the method to obtain approximate solutions for reinforced concrete cylindrical shells in the post-linear range. In order to eliminate the uncertainty related to the somewhat arbitrary reduction of stiffness coefficients in the above analyses[28,29] Riera and Billington[30] proposed to assume that the concrete-reinforcing steel composite material is non-linearly elastic, which is shown to be an adequate idealization for concrete shell roofs under sustained loads.

Following a different but essentially parallel approach, Rashid[31] proposed to treat the influence of a crack on a continuous element as a mechanism that changes the element's behavior from isotropic to orthotropic. In other words, concrete is assumed to be isotropic whenever stresses are contained inside the failure surface. It becomes orthotropic when a crack develops normal to a principal stress direction. The stiffness coefficients are then set equal to zero in the direction of the normal to the crack. Once a cracking criterion is established, practical solutions can be obtained using a discrete representation, or a finite-element formulation of the problem. An essentially identical procedure used by Watson to determine stresses around a circular tunnel through a rock material with a stringent cracking criterion is briefly described by Zienkiewicz[32].

Several areas of concern associated with the approach may be mentioned: First, the failure surface for concrete and its time and temperature dependence have not yet been well defined. Disagreement exists concerning the short-time failure envelope of concrete in biaxial compression and to a greater degree concerning general stress states involving at least one principal tensile stress. Second, the question of existence of solutions and convergence of

iterative or other numerical techniques used to determine those solutions have not been explored in depth.

Zienkiewicz[32] notes, in connection with direct iterative solutions for non-linear elastic materials, that three or four iteration cycles are usually sufficient to obtain satisfactory results. Riera and Billington[30] also obtained good correlation between experimental and numerical results for a cylindrical reinforced mortar shell after three iteration cycles, but they note that the process is not necessarily convergent and that there is no rigorous justification for using the values obtained after a few iteration cycles, disregarding what may happen afterwards. Since the convergence of the iterative procedure depends upon the degree of nonlinearity of the constitutive equations, the problem may be circumvented by applying the loads in small increments[30]. The question of how small these increments should be still remains unresolved. Rashid[31] determined the load-carrying capacity of axially symmetrical prestressed concrete primary pressure vessels by iterating until no further cracks develop after each small load increment, thereby improving the likelihood of obtaining a correct solution.

In spite of the aforementioned areas of concern, the stress analysis of reinforced or prestressed concrete shells in the cracked condition on the basis of reduced or modified stiffness coefficients has been quite successful. The method also led to valuable results in the stress analysis of thick (axially-symmetrical) pressure vessels. The correlation between experimental and theoretical results reported in Ref. 28 to 31 is good and should encourage additional basic research in this area. In the meantime, the technique appears to be sufficiently developed to be used in the solution of technical problems, such as the stress distribution around large openings in the R. E. Ginna Containment Vessel. Because of the computational effort demanded by the solution of this particular problem, elaborate approaches such as application of the load in small increments, combined with iteration, could not be employed. Instead, a direct iterative technique on the stiffness distribution (with the fully loaded structure) was used. Furthermore, the application of the former method would require the specification of a complete history of external loads and temperatures, which are not known nor easily predictable. In other words, too many loading conditions need be investigated to make the approach feasible or even meaningful.

Consequently, areas of concern such as convergence requirements, influence of cracking criterion, and effect of different loading histories on the theoretical results were given careful consideration in the verification of structural adequacy.

4.3STRESS ANALYSIS IN CRACKED AND UNCRACKED CONDITIONS UNDER OPERATING AND ACCIDENT LOADS

4.3.1Representation of the Shell Around the Opening

The finite-element grid used to solve the test problem (Section 4.1) was also employed to determine the stress distribution around the large openings in the Containment Vessel. The shell was idealized as follows:

a. The liner was represented as an isotropic steel layer (E_{st} = 30,000 ksi, ν = 0.3). Composite action was assumed in the determination of the stress resultants and stress couples.

b. The horizontal steel reinforcement (hoop rebars) was represented as a layer of an orthotropic material having no Poisson's ratio effect, no shear stiffness ($G_{12} = 0$) and no meridional stiffness ($E_1 = 0$). This layer was located at the center of gravity of the hoop reinforcement.

c. The ring steel rebars, providing additional reinforcement around the opening, were represented as a layer located at the center of gravity of the ring reinforcement. The shear stiffness of this layer was set to equal zero ($G_{12} = 0$). The hoop (E_2) and meridional (E_1) stiffnesses vary from $E_1 = E_{st}$, $E_2 = 0$ along the horizontal axis of the opening, to $E_1 = 0$, $E_2 = E_{st}$ along the vertical axis of the opening. In Computer Program FELAP the axes of orthotropicity are oriented in the hoop and meridional directions. Consequently, since the axes of orthotropicity of the ring reinforcement only coincide with those directions in the regions around the opening's vertical and horizontal axes, the following approximation was made: at every panel type the hoop and meridional stiffnesses were directly proportioned to the projected steel area.

d. The concrete layers were idealized as follows:

1. Uncracked concrete
   - $E_1 = 4000$ ksi
   - $E_2 = 4000$ ksi
   - $G_{12} = 1740$ ksi.
   - $\nu_{12} = 0.15$
2. vertically cracked concrete
   - $E_1 = 4000$ ksi
   - $E_2 = 0$
   - $G_{12} = 800$ ksi
   - $\nu_{12} = 0$
3. horizontally cracked concrete
   - $E_1 = 0$
   - $E_2 = 4000$ ksi
   - $G_{12} = 800$ ksi
   - $\nu_{12} = 0$
4. fully cracked concrete
   - $E_1 = 0$
   - $E_2 = 0$
   - $G_{12} = 800$ ksi
   - $\nu_{12} = 0$

The shear stiffness G_{12} of cracked concrete was computed taking into consideration the shear deformation of concrete between cracks, as well as the deformation of the rebars

subjected to dowel action. The contribution of the latter was obtained from "Nelson Manual No. 21", Figure 13, p. 12. The following expression for the shear stiffness of concrete (that would lead to a correct in-place shear stiffness of the entire section) was derived: $G_{12} \cong \dfrac{G}{1 + \dfrac{1}{pL}}$

in which

G = shear stiffness of uncracked concrete

ρ = ratio of area of steel rebars in dowel action to concrete area.

L = distance between cracks (in.)

Taking

ρ = 0.034 and L = 25 in.

G = $\dfrac{G}{2.17}$ = 803,000 psi

Analyses based on G_{12} equal to zero, 800 ksi and 1740 ksi indicated that the in-plane stress-resultants and stress-couples are not sensitive to variations in G_{12}.

e. The barrel (penetration sleeve) was represented on contributing, with 50 percent of its area, to the stiffness of the elements adjacent to the openings. In other words, a stiffener of 3/8 in. thickness was included in the model along the periphery of the opening.

Note that although the penetration barrel was incorporated into the finite-element model as explained above, it was not regarded as contributing to the load-carrying capacity of the shell (See Section 5.2).

4.3.2Basic Loading Conditions

The stress distribution around the opening was determined for the loading conditions described in Section 1 and those loading combinations more completely described hereafter. The specific loads are defined as follows:

a. Dead Load

The stress distribution around the opening due to dead weight was calculated assuming a uniform meridional compression in the cylinder equal to the stress resultant at the elevation of the opening axis (in the shell without the opening). The weights of the equipment hatch and personnel lock were neglected. Since dead weight stresses in the typical shell wall at the elevation of the equipment and personnel access are low (less than 100 psi), the above simplifications will not significantly influence the final stresses in the pressurized vessel.

b. Internal Pressure

Internal pressure was assumed to act on the interior of the shell as well as on the barrel (penetration sleeve) of the equipment access hatch. The internal pressure on the hatch was

assumed transferred to the shell by a uniform normal shear Q_ρ around the opening. The effect of non-uniform distributions of Q_ρ were analyzed separately.

The personnel opening was analyzed with and without internal pressure acting on the barrel (penetration sleeve).

c. Earthquake

Seismic stresses around the openings were investigated for two directions of the horizontal component of motion:

1. earthquake motion oriented in the direction normal to the openings
2. earthquake motion oriented at 90° with the direction normal to the openings

Seismic loads were evaluated as described in Appendix B.

d. Prestress

Prestressing loads are represented as two independent loading conditions by a uniform meridional compression in the shell (away from the opening) equal to the stress resultant corresponding to initial and final prestress. Curved tendons around the opening were considered in the analysis as line loads q = T/r where T is the total prestressing force per tendon and r is the radius of curvature at the location under consideration. These line loads were integrated within each panel and applied as nodal forces on the shell, as shown in Figure 11.

e. Thermal Loads

1. Operating Temperature

The steady-state temperature distribution in the reinforced area around the opening was obtained using a modified Gauss-Seidel iteration technique.

The structure was broken up into 2257 elements (38 x 62 nodes) and the temperature at each node was determined by the temperature at the four nodes surrounding it using the formula:

$$T = \frac{\sum_{i=1}^{4} T_i K_i \frac{f_1}{X_i}}{\sum_{i=1}^{4} K_i \frac{f_i}{X_i}}$$

The skin temperatures of the structure were determined by a parabolic curve fit using the formula:

$$h(T_{AMP} - T_{skin}) = -\frac{k}{2\Delta X}(3T_{skin} - 4T_{skin-1} + T_{skin-2})$$

or

$$T_{skin} = \frac{T_{AMB} - \frac{k}{2\Delta Xh}[T_{skin-2} - 4XT_{skin-1}]}{1 + \frac{3k}{2\Delta Xh}}$$

The following values were used for the thermal conductivity:

INSULATION	$k = 0.0208 \ \frac{BTU}{hr\ ft\ °F}$
STEEL	$k = 26.0 \ \frac{BTU}{hr\ ft\ °F}$
CONCRETE	$k = 0.8333 \ \frac{BTU}{hr\ ft\ °F}$

The following values were used for the boundaries:

INSIDE	$h = 2.0 \ \frac{BTU}{hr\ ft^2\ °F}$
TOP	$h = 1.0 \ \frac{BTU}{hr\ ft^2\ °F}$
OUTSIDE	$h = 1.0 \ \frac{BTU}{hr\ ft^2\ °F}$
BOTTOM	Adiabatic

The concrete was considered to be 4 percent steel reinforcement and its conductivity was determined from the formula:

$$k_{effective} = .96k_{concrete} + .04k_{steel}$$

The iteration was performed on the computer to a tolerance of 0.005°F.

Typical results of the thermal analysis for operating conditions are given in Figure 12, which shows the temperature distribution around the opening for the equipment access hatch with an interior air temperature of 120°F and an exterior air temperature of -10°F. Linearized thermal gradients at 8, 32, and 96 in. from the edge are shown in the same figure.

The input for the finite-element stress analysis was prepared on the basis of the above results. The steady-state (winter) temperature distribution around the opening used in the analysis is indicated in Figure 13. To simplify the input, a temperature of 0°F was used in the analysis for the outside face of the typical wall rather than -3°F. Consequently, all temperatures were shifted by +3°F.

Summer thermal stresses are conservatively computed as equal to 40 percent of the peak winter thermal stresses.

2. Accident Temperature

Since the finite-element solution used in the analysis is restricted to linear temperature gradients through the wall thickness, accident thermal stresses, which result in a highly non-linear gradient, were computed as described below:

a. Effect on typical wall: The 10°F temperature increase in the liner was conservatively represented by an equivalent internal pressure equal to the pressure that the heated liner would exert against a rigid confining cylinder.

b. Effect on the barrel (penetration sleeve): The effect of increasing the liner temperature to 312°F (net increase: 312 - 120 = 192°F) was represented by internal pressure acting on the barrel. The magnitude of this pressure was determined on the basis of a two dimensional analysis, which assumed that the barrel (t = 0.75 in.) plus 0.55 in. of concrete were suddenly heated to 192°F. The equivalent pressure on the concrete was found to be equal to 160 psi.

f. Bond Failure Along Rebars Anchored Near Opening Edge

It is expected that the horizontal rebars terminated near the edge of the opening will transfer their load by bond to the surrounding concrete and thus to the ring reinforcement and adjacent (uninterrupted) rebars. However, since concrete in the area involved may present vertical cracks, it appears unsafe to rely only on bond stress transfer. Therefore, the stress redistribution that would occur in case of a complete bond failure along all horizontal rebars terminated at the opening was investigated as described below.

Figure 14a shows one such rebar with the rebar load before bond failure (as determined by the finite element analysis of the pressurized shell in the cracked condition) indicated in Figure 14b. The assumed bond stress distribution is also shown in Figure 14b. Note that the shaded area times the perimeter of the rebar equals the load in the rebar away from the opening. The applied loads in the finite element study of the stress redistribution due to bond failure are schematically shown in Figure 14c. The stress resultants and stress couples from the present analysis were superimposed with those corresponding to internal pressure to obtain the stress state expected under internal pressure in the case when all loads of the rebars are transferred to the mechanical anchorage at the end of the rebars. These stresses are treated in the evaluation of the results as an independent loading condition. The load combinations considered in the analysis are given in Table 4-1. The stress-resultants, stress-couples and normal shears due to the 76 load combinations described in Table 4-1 were calculated using a computer program for sixteen elements located along both symmetry axes and along a 45° line.

4.3.3 Effect of Concrete Cracking

The effect of concrete cracking on the stress distribution around the opening was determined, as outlined in Section 4.2, by assuming that concrete principal tensile stresses in excess of 424 psi produce cracks in the direction normal to that principal stress direction. In the finite

element analysis this is accomplished by setting a stiffness coefficient at that location equal to zero. Since this procedure cannot be performed for every load combination, the iteration was carried on for accident conditions with 90 psi internal pressure and no thermal effect, which appears to be the most unfavorable condition as far as the extent of cracking around the opening is concerned. The shell was first analyzed in the uncracked state, and the stresses corresponding to the following load combination determined:

$$1.0 \text{ DL} + 1.0 \text{ VP}_f + 1.5 \text{ IP}$$

In successive runs the average principal tensile stresses within each of the concrete layers of the model (See Figure 10) were inspected. When this average principal stress at any layer of an element exceeded 424 psi, the elastic properties of that particular layer were changed as follows:

a. If the average principal tensile stress direction is sensibly horizontal, then E_2 is set equal to zero.

b. If the average principal tensile stress direction is sensibly vertical, then E_1 is set equal to zero.

c. If the average principal tensile stress direction is approximately 30°, 45° or 60°, then:

   $E_1 = E_0 \sin 30°$ $E_2 = E_0 \cos 30°$, or

   $E_1 = E_0 \sin 45°$ $E_2 = E_0 \cos 45°$, or

   $E_1 = E_0 \sin 60°$ $E_2 = E_0 \cos 60°$

   respectively. For cracks oriented other than horizontally or vertically, the above stiffness coefficient reduction constitutes an approximation Figure 15 gives the stress-resultant distribution under internal pressure along both symmetry axes for the uncracked shell, as well as results of several analyses for the cracked shell. It should be noted that the difference between stress resultant distributions based on different degrees of cracking is not excessive and, furthermore, that all distributions are close to the distribution corresponding to an uncracked, unreinforced shell computed on the basis of Eringen's 'et al' theory[9]. Whenever possible, the results presented in References (*9*) to (*11*) were compared with the finite-element solutions to provide additional evidence on the correctness of the computed values. Displacements for vertical prestress and internal pressure are shown in Figures 16 and 17.

   Stress-resultants and stress-couples for the uncracked shell can be found in Table 4-2. The results corresponding to the last computer analysis are summarized in Table 4-3. Tables 4-2 and 4-3 give the stresses at the center of the elements located along both symmetry axes and along an approximately 45° line (See Figure 4). Inspection of the data indicates that these elements represent the significant areas which could conceivably control the design.

   These results are considered to represent the state of stress in the shell with the cracking pattern expected under 90 psi internal pressure. Note that these distributions for the basic loading conditions were obtained with the same model. Also note that earthquake stresses were not computed by means of the finite-element technique. (See Appendix B)

4.3.4Effect of Creep and Shrinkage

It appears that shrinkage of concrete can only introduce compressive stresses into the steel rebars. These stresses will largely disappear after cracking due to the internal pressure in the shell takes place, and need not be considered in the stress analysis of the opening.

The load redistribution due to concrete creep (i.e., the redistribution of N_ϕ, $N_{\phi\theta}$, N_θ, M_ϕ, M_θ, etc.) is expected to be small. This conclusion is sustained by a finite-element plane stress analysis, which indicates that the stress concentration factor for uniaxial compression changes by only 5 percent when the hoop 'in-plane' stiffness is reduced by 100 percent. If the 'effective modulus' approach is used to determine the final stress distribution under operating stresses, a final modulus equal to 40 percent of its initial value would lead to a final ratio between vertical and hoop 'in-plane' stiffness equal to:

$$\frac{42 \times 1600}{40.6 \times 1600 + 1.4 \times 30,000} = \frac{42}{66.8} = 0.63$$

the initial ratio is:

$$\frac{42 \times 4000}{40.6 \times 4000 + 1.4 \times 30,000} = \frac{42}{51.1} = 0.82$$

Therefore, a change in the ratio between vertical and hoop in-plane stiffnesses of about 23 percent is not expected to have any significant effect on the stress-resultant and stress-couple distributions.

The effect of creep and shrinkage on prestress losses is taken into account as indicated in Section 5.8. Likewise, the transfer of load from concrete to steel in any given cross-section is considered in the verification of rebar stresses when applicable.

With reference to the discussion of Appendix A, the following conclusions can be stated:

a. The load redistribution due to concrete creep in the unpressurized vessel will not affect in any significant degree the load distribution in the structure under test or accident pressure.

b. The knowledge of the stress-resultant and stress-couple distributions after creep in the unpressurized vessel cannot be of direct use in the evaluation of the corresponding distributions under internal pressure, should the internal pressure be applied late in the life of the structure, since as concrete cracking takes place, the distribution before the internal pressure is applied changes according to the cracking pattern.

5. VERIFICATION OF DESIGN CRITERIA

5.1BASIS FOR VERIFICATION OF SHELL LOADING CAPACITY DUE TO PRIMARY LOADS (PRINCIPAL STRESS-RESULTANTS AND PRINCIPAL STRESS-COUPLES)

The loading capacity at any point of the shell was verified according to the following procedure:

a. The principal stress-resultants N^1 and N^2 were computed in terms of N_ϕ, N_θ, and $N_{\phi\theta}$

b. The principal stress-couples M^1 and M^2 were computed in terms of M_ϕ, M_θ, and $M_{\phi\theta}$

c. Considering that throughout the critical regions of the shell (both axes of symmetry and along the edge of the opening) the orientations of N^1, N^2, and M^1, M^2 coincide with the orientation of the reinforcement and that in the rest of the shell they nearly coincide with each other and with the orientation of the steel rebars, systems N^1, M^1 and N^2, M^2 are treated independently. Since in the regions in which the directions of principal stress-resultants and stress-couples are not colinear or do not coincide with the orientation of the steel rebars, stresses are low, the error introduced by assuming them colinear will not affect the conclusion concerning the load-carrying capacity of the shell. In the latter case, steel rebars not oriented in the direction of principal stress-resultants or stress-couples were conservatively neglected in the computation of the strength of the section. In other words, it is proposed that forces it the 1-direction will not affect the strength of the section in the 2-direction and vice verse. This is in full agreement with a square failure criterion for concrete in biaxial compression, which appears to very conservative. Stresses in rebars oriented along a principal stress direction obviously will not be affected by forces in the other principal direction.

d. The computed ultimate capacity of any section of the shell satisfies the requirements of ACI 318-63, Sections 1600, 1700, 1800, and 1900. 0nly deformed bars as defined in ACI 318-63, Section 301 are used. Deformed bars ensure higher bond strength and a more uniform crack distribution.

e. Composite action between the shell and the liner is neglected in the computation of ultimate moments. The liner is regarded as carrying only its share of the principal stress-resultants.

Interaction diagrams were prepared as described in Section 5.2 for elements located along both symmetry axes and along a 45° line. Principal stress-resultants and stress-couples corresponding to all critical load combinations are show in Figures 19 through 24 on the interaction diagram corresponding to elements 55, 66, 77, 49, 73, 97, and 101. The position of a point representing a stress state within the diagram gives a clear indication of the 'local safety factor' at that location (i.e., at the center of the element). It must be emphasized that even if a point representing a stress state fell outside the diagram, that would not indicate a critical or nearly critical condition for two reasons:

a. The interaction diagrams were determined on the basis of conservative assumptions and it is expected that the 'true' failure envelopes lie a certain distance away from the computed envelopes.

b. A point outside the interaction diagram would merely indicate local yielding of one or more rebars at that location, which would cause a load redistribution towards less highly stressed regions. A point representing a stress state contained withIn an interaction diagram indicates that stresses in all steel rebars at that section are below yield stress, and that concrete stresses, where applicable, are below code allowable stresses.

5.2INTERACTION DIAGRAM

a. Axial Compression and Bending

(See Figure 18)

1. Concentric Compression

$$P_0 = \phi[0.85\, f'_c\, A_c + A_{st}\, f_y],$$

$$A_{st} = A_{s1} + A_{s2} + A_{s3} \quad \text{(Equation 5.1)}$$

2. Simple Bending

$$a = \frac{A_{s1}\, f_y + A_{s2}\, f_{s2} - A'_{s3}\, f_{s3}}{0.85\, f'_c\, b} \quad \text{(Equation 5.2)}$$

$$M_0 = \phi\left[T_1 d_1 + T_2 d_2 + C_{s3}\, d_3' + C_c\left(C - \frac{a}{2}\right)\right]$$

$$\frac{C}{\varepsilon_c} = \frac{d}{\varepsilon_{sl} + \varepsilon_c}$$

$$C_c = 0.85\, f_c'\; a\; b$$

3. Bending and Axial Compression

$$C_b = \frac{d(87000)}{87000 + f_y}$$

$$a_b = k_1\, a . b \quad \text{(Equation 5.3)}$$

$$P_b = \phi\,[0.85\, f_c'\,(a_b - a)\, b]$$

$$M_b = M_0 + P_b\, d_b$$

4. Determine P_a and M_a

$e_a = 0.1\ t;$ ACI 318-63, Section 1901 (a)

$$M_a = P_a\, e_a$$

By similar triangles, $$P_a = \frac{P_0\ M_b}{e_a\,(P_0 - P_b) + M_b}$$ (Equation 5.4)

b. Axial Tension and Bending

1. Concentric Tension

$$P_0 = \phi\,[A_{st}\ f_y + t_L\ f_{yL}]$$ (Equation 5.5)

Note: For purpose of clarity, only three steel layers were included in the preceeding equations.

5.3REINFORCING STEEL

a. Hoop, draped and elliptical steel reinforcement will be as shown on Drawing Nos. D-421-023 (Figure Drawing 2), and D-421-024 (Figure Drawing 1).

b. The amount of reinforcement which includes regular hoop steel draped around the opening and elliptical reinforcement, equals or exceeds that shown to be required by calculations. In no event is the liner assumed to contribute more than its yield strength.

c. The clear distance between parallel bars is not less than 2 times the maximum size of coarse aggregate, 2 times the bar diameter, nor less than 2 in.

d. Vertical shrinkage or temperature reinforcement is placed at outside face of wall. The minimum amount of such reinforcement on the outside concrete face wall is greater than 0.0015 of the gross cross-sectional area of concrete.

5.4MAXIMUM LINER STRESSES

The maximum liner stresses are given in Table 5-1.

5.5PENETRATION BARREL

The portions of the Equipment Access Hatch and Personnel Lock extending beyond the concrete shell were designed and fabricated by Chicago Bridge and Iron Company in accordance

with the ASME Nuclear Vessels Code. The barrel of each of these penetrations within the limit of the concrete thickness was investigated for the following loads:

a. Meridional membrane stresses in the barrel due to internal pressure on the hatch.

b. Hoop membrane stresses in the barrel due to the in-plane deformation of the opening.

c. Meridional bending stresses in the barrel caused by meridional shear transfer from the barrel to the concrete.

d. Thermal Stresses

The objective of this investigation was to verify that the stresses in the barrel are within allowable Units. Refer to ASME Nuclear Vessels Code, Article 4, Par. N-414. It should be noted that the allowable stresses referred to are based on working strength design.

5.6NORMAL SHEAR

a. Normal shears in the concrete shell surrounding the.Penetration Barrel have been computed by the Finite Element Method of analysis. The computed normal shear stress resultant, at a distance $\frac{d}{2}$ = 33.0 in. from the edge of the opening or twice the normal shear transferred by the barrel, whichever was the larger of the two, was used in the design.

b. Two modes of shear transfer are considered. First, sheer transfer through concrete without shear reinforcement. Second, disregarding the shear capacity of concrete, enough reinforcing steel is provided to carry the normal shears by steel alone.

Ultimate peripheral or normal shear stress carried by concrete is computed by:

$$v_u = \frac{Q}{d}$$

$$v_c = 4\phi (f_c')^{1/2}$$ (Equation 5.6)

c.

(ACI 318-63, Section 1707)

d.

where:	Q =	normal shear stress-resultant at the critical section
	v_u =	nominal ultimate shear stress as a measure of diagonal tension
	v_c =	allowable ultimate shear stress to be carried by concrete
	d =	distance from extreme compression fiber to centroid of tension reinforcement
	f'_c =	28 days compressive strength of concrete

$\phi =$ 0.85

Shear reinforcement requirements

The ultimate shear capacity of the reinforcing steel alone is computed by:

$$v_u = \phi \left[A_{sv} f_y + A_{sD} \frac{f_y - f_r}{2} \right]$$ (Equation 5.7)

where: $A_{sv} =$ cross-sectional area of reinforcing steel acting in tension across a potential diagonal tension crack

$A_{sD} =$ cross-sectional area of reinforcing steel acting in dowel action across a potential diagonal tension crack

$f_y =$ yield strength of reinforcement

$f_r =$ existing stress due to N, M in rebar also acting as a dowel

$\phi =$ 0.85

5.7REBAR ANCHORAGE

a. The #18S regular hoop steel that is terminated at the penetration barrel will transfer the tensile load from the reinforcing bar to concrete by bond. As a redundancy, a mechanical anchor is provided at the end of each bar to transfer the tensile load from the reinforcing bar to the concrete in bearing. For details of anchor plates see Drawing No. D-421-024 (Figure Drawing 1).

b. Ultimate bearing stress on the concrete is computed by:

$$f_c = \frac{A_s f_y}{A_c}$$

$$F_{cu} = 0.6 f_{c\,i}' \sqrt[3]{\frac{A_b'}{A_b}}$$ (Equation 5.8)

Ultimate bending stress in anchor plates will not exceed 0.90 yield strength of the anchor plate.

The transfer tensile reinforcement through the wall thickness will be determined by[33]:

$$A_{st} = \frac{\alpha A_s f_y}{0.95 \; f_y} = 0.105 \;\; A_s$$ (Equation 5.9)

where α = splitting force/axial force which for this design is equal to 0.1.

5.8 TENDON LOSSES

The vertical post-tensioning tendons are curved around the large access openings as shown on Drawing No. D-421-023 (Figure Drawing 2).

Tendon friction losses are determined according to ACI 318-63, Chapter 26.

$$T_0 = T_x e^{(KI + \mu\alpha)}$$ (Equation 5.10)

where:	K =	0.0003 (curved length only)
	μ =	0.16

Tendon losses due to elastic shortening, shrinkage, creep, and steel relaxation have been determined. These losses, combined with the additional frictional losses, will require retensioning of the curved tendons after 1000 hours.

Tendon losses excluding friction after 40 years without retensioning will be approximately 16.0%.

Tendon losses excluding friction after 40 years, retensioned 1000 hours after the initial stressing, will be 11.75%.

Steel stress in any curved tendon around the large opening was determined by using the following formula:

$$T_0^{\,i} (0.8825) = T_x^{\,i} e^{(KI + \mu\alpha)}$$ (ACI 318-63 Eq. 26-2)

$$F_{s_i} + \frac{T_x^{\,i}}{A_s}$$

(Equation 5.11)

where:	f_{si} =	steel stress in the i-th tendon at point x
	i =	i-th curved tendon
	A_s	area of prestressed tendons

It should be noted that the average steel stress of a group of curved tendons will be:

$$f_{s_{avg.}} = \frac{\sum_{i=1}^{n} T_x^{\ i}}{N} \geq 0.6\ f_s'$$

(Equation 5.12)

where: f'_s = ultimate strength of tendon steel

N = number of curved tendons

5.9SUMMARY OF DESIGN AND CONCLUSIONS

In selecting the analytical method used for determining the stress-resultant and stress-couple distributions in the shell under all critical loading conditions an extensive bibliographic search was conducted and available methods were evaluated. In our judgment the analysis of the stresses around the openings for the R. E. Ginna Containment Vessel has been based on the most satisfactory of available methods.

The design was guided by the basic proposition that the best reinforcement is in fact the least reinforcement that will satisfy the requirement for carrying the shell loads around the opening and the normal shear applied along the opening edge into the shell out to a distance from the opening until a membrane state of stress is reached. Although not directly applicable the IITRI studies on steel containment structures conclusively showed that a stiff reinforcement around openings substantially reduces the burst strength of circular plates[34]. In our judgment this design as evidenced by the data included in Section 5 provides the required reinforcement strength and further conservatism in determining reinforcement requirements is not prudent in that the ultimate load capacity might be thereby reduced.

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Table 4-1
Load Combinations

LOAD COMBINATIONS		FUNDAMENTAL LOADS							
		DL(1)	VP(2)	OT(3)	AT(4)	IP(5)	Bo(6)	E1(7)	E2(8)
Normal* Operation	1	1.0	1.17	0.4					
	2	1.0	1.0	0.4					
	3	1.0	1.17	1.0					
	4	1.0	1.0	1.0					
Test Pressure	5	1.0	1.17	0.4		1.0			
	6	1.0	1.0	0.4		1.0			
	7	1.0	1.17	0.4		1.0	1.0		
	8	1.0	1.0	0.4		1.0	1.0		
	9	1.0	1.17	1.0		1.0			
	10	1.0	1.0	1.0		1.0			
	11	1.0	1.17	1.0		1.0	1.0		
	12	1.0	1.0	1.0		1.0	1.0		
Accident Pressure Loadings	13	1.0	1.17	1.0	1.0	1.304			
	14	1.0	1.0	1.0	1.0	1.304			
	15	1.0	1.17	0.4	1.0	1.304			
	16	1.0	1.0	0.4	1.0	1.304			
	17	1.0	1.17	1.0	1.0	1.304	1.304		
	18	1.0	1.0	1.0	1.0	1.304	1.304		
	19	1.0	1.17	0.4	1.0	1.304	1.304		
	20	1.0	1.0	0.4	1.0	1.304	1.304		
	21	1.0	1.17	1.0	0.92	1.087			0.404
	22	1.0	1.0	1.0	0.92	1.087			0.404
	23	1.0	1.17	0.4	0.92	1.087			0.404
	24	1.0	1.0	0.4	0.92	1.087			0.404
	25	1.0	1.17	1.0	0.92	1.087	1.087		0.404
	26	1.0	1.0	1.0	0.92	1.087	1.087		0.404
	27	1.0	1.17	0.4	0.92	1.087	1.087		0.404
	28	1.0	1.0	0.4	0.92	1.087	1.087		0.404
	29	1.0	1.17	1.0	0.92	1.087		0.404	
	30	1.0	1.0	1.0	0.92	1.087		0.404	
	31	1.0	1.17	0.4	0.92	1.087		0.404	
	32	1.0	1.0	0.4	0.92	1.087		0.404	
	33	1.0	1.17	1.0	0.92	1.087	1.087	0.404	
	34	1.0	1.0	1.0	0.92	1.087	1.087	0.404	
	35	1.0	1.17	0.4	0.92	1.087	1.087	0.404	
	36	1.0	1.0	0.4	0.92	1.087	1.087	0.404	
	37	1.0	1.17	1.0	0.92	1.087		-0.404	
	38	1.0	1.0	1.0	0.92	1.087		-0.404	
	39	1.0	1.17	0.4	0.92	1.087		-0.404	
	40	1.0	1.0	0.4	0.92	1.087		-0.404	
	41	1.0	1.17	1.0	0.92	1.087	1.087	-0.404	
	42	1.0	1.0	1.0	0.92	1.087	1.087	-0.404	
	43	1.0	1.17	0.4	0.92	1.087	1.087	-0.404	
	44	1.0	1.0	0.4	0.92	1.087	1.087	-0.404	
	45	1.0	1.17	1.0	0.92	1.087			-0.404
	46	1.0	1.0	1.0	0.92	1.087			-0.404
	47	1.0	1.17	0.4	0.92	1.087			-0.404
	48	1.0	1.0	0.4	0.92	1.087			-0.404
	49	1.0	1.17	1.0	0.92	1.087	1.087		-0.404
	50	1.0	1.0	1.0	0.92	1.087	1.087		-0.404
	51	1.0	1.17	0.4	0.92	1.087	1.087		-0.404
	52	1.0	1.0	0.4	0.92	1.087	1.087		-0.404
	53	1.0	1.17	1.0	0.85	0.87		1.0	
	54	1.0	1.0	1.0	0.85	0.87		1.0	
	55	1.0	1.17	0.4	0.85	0.87		1.0	
	56	1.0	1.0	0.4	0.85	0.87		1.0	
	57	1.0	1.17	1.0	0.85	0.87	0.87	1.0	
	58	1.0	1.0	1.0	0.85	0.87	0.87	1.0	
	59	1.0	1.17	0.4	0.85	0.87	0.87	1.0	
	60	1.0	1.0	0.4	0.85	0.87	0.87	1.0	
	61	1.0	1.17	1.0	0.85	0.87		-1.0	
	62	1.0	1.0	1.0	0.85	0.87		-1.0	
	63	1.0	1.17	0.4	0.85	0.87		-1.0	
	64	1.0	1.0	0.4	0.85	0.87		-1.0	
	65	1.0	1.17	1.0	0.85	0.87	0.87	-1.0	
	66	1.0	1.0	1.0	0.85	0.87	0.87	-1.0	
	67	1.0	1.17	0.4	0.85	0.87	0.87	-1.0	
	68	1.0	1.0	0.4	0.85	0.87	0.87	-1.0	
	69	1.0	1.17	1.0	0.85	0.87			-1.0
	70	1.0	1.0	1.0	0.85	0.87			-1.0
	71	1.0	1.17	0.4	0.85	0.87			-1.0
	72	1.0	1.0	0.4	0.85	0.87			-1.0
	73	1.0	1.17	1.0	0.85	0.87	0.87		-1.0
	74	1.0	1.0	1.0	0.85	0.87	0.87		-1.0
	75	1.0	1.17	0.4	0.85	0.87	0.87		-1.0
	76	1.0	1.0	0.4	0.85	0.87	0.87		-1.0

DL = Dead Load

VP = Vertical Prestress

OT = Operating Temp.

AT = Accident Temp.

IP = Internal Pressure

B_o = Loss of Bond

E_1 = Earthquake 1

E_2 = Earthquake 2

Note: Coefficients are based on: Test pressure, Winter temp., Final Prestress, Max. Hypothetical Earthquake

*Distributions for normal operation based on uncracked model.

Initial prestress: $\frac{.7}{.6} = \underline{\underline{1.17}}$

Pressure Load:

$\frac{1.5}{1.15} = 1.3043$ $\frac{1.25}{1.15} = 1.0870$ $\frac{1.0}{1.15} = 0.8696$

Earthquake: 0.2g - 2% damp → 0.47; 0.08g 2% damp → 0.19; $r = \frac{0.19}{0.47} = 0.404$

Accident temp:
90 psig 1.5p→ T = 312°F r = 1.0
75 psig 1.25p→T = 285°F r = 285/312 = 0.92
60 psig 1.0p→ T = 265°F r = 265/312 = 0.85

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Table 4-2
Stress Around Equipment Hatch-Loading (Uncracked Shell)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 2
Vertical Prestress (Final)

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ
11	-22.410	- 18.050	1.428	- 3.797	0.166	1.101	0.001
22	-19.470	- 72.220	2.573	23.639	1.065	0.418	0.039
33	-15.232	- 87.135	2.975	75.887	1.382	-0.900	0.184
44	-10.148	- 63.587	3.005	95.206	2.070	-1.271	0.297
55	- 5.290	- 22.978	4.664	122.587	2.243	-0.227	-0.219
66	- 2.056	1.545	8.666	170.009	2.252	0.316	-0.841
77	0.064	31.434	17.139	273.855	.384	5.411	0.324
25	-32.907	-212.489	4.786	61.086	0.355	2.654	-0.844
49	-34.615	-307.571	6.387	78.725	2.061	1.699	-0.930
73	-38.819	-377.591	4.024	63.011	4.300	0.585	-1.245
74	-31.907	-308.348	-2.301	79.870	7.999	-0.190	3.254
94	-23.312	31.380	3.933	- 42.611	-0.127	-0.006	-0.551
97	-38.051	-405.931	1.403	- 5.139	-0.077	0.073	0.345
99	-43.620	33.491	-3.261	421.929	0.409	0.366	0.962
100	-52.537	-478.211	-6.196	- 48.723	1.671	1.480	0.738
101	-62.902	-612.005	-6.349	- 60.802	3.436	-3.406	-7.782

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ and M_θ in Kips.

Values computed by finite-element analysis of uncracked shell.

Sheet 1 of 2

GILBERT ASSOCIATES, INC.

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 4
69 psi Internal Pressure

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ
11	22.350	138.494	43.708	14.810	- 0.028	1.438	0.009
22	22.208	74.061	49.821	230.145	- 0.004	0.895	-0.035
33	21.764	40.226	63.614	582.624	- 0.339	- 0.063	-0.135
44	19.749	43.602	67.554	602.660	- 1.021	- 0.740	-0.084
55	16.294	82.115	75.487	651.797	- 2.272	5.219	-1.298
66	12.291	107.555	86.917	746.735	- 4.078	8.460	-2.052
77	4.827	129.865	100.974	942.268	6.999	18.022	5.028
25	24.344	48.384	49.209	273.717	- 1.193	- 0.974	-2.980
49	25.229	86.106	45.915	347.055	- 8.114	- 2.250	-2.614
73	28.609	178.390	36.473	332.698	-17.349	- 4.008	-0.998
74	26.820	64.056	38.803	351.177	-24.430	-13.956	-3.537
94	20.988	- 21.114	35.201	63.859	- 0.342	- 0.032	-1.770
97	35.276	374.791	26.405	164.953	- 1.747	- 0.706	0.393
99	37.272	385.443	15.239	131.236	- 3.218	- 0.984	1.455
100	38.959	388.064	7.885	106.644	- 4.163	- 1.473	2.016
101	36.600	404.216	0.533	73.195	- 3.367	3.706	6.202

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ and M_θ in Kips.

Values computed by finite-element analysis of uncracked shell.

Sheet 2 of 2

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Table 4-3
Stress Around Equipment Hatch-Loading (Cracked Shell)

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 1
Dead Load

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	- 2.09	- 11.58	-0.06	0.01	0.07	0.02	0.	-0.09
22	- 1.86	- 13.95	-0.17	-0.01	0.10	0.03	-0.04	0.71
33	- 1.24	- 3.30	-0.17	-1.73	0.15	-0.24	0.03	1.52
44	- 0.72	1.74	0.23	1.36	0.20	-0.22	0.02	1.55
55	- 0.27	3.16	0.66	4.85	0.19	0.01	-0.03	0.66
66	0.	2.58	1.14	8.91	0.18	0.11	-0.09	0.46
77	0.16	0.56	2.05	16.35	-0.10	0.51	-0.17	-1.90
25	- 5.00	- 32.56	0.15	3.58	-0.11	0.21	-0.10	-7.68
49	- 5.19	- 40.21	0.23	4.04	-0.16	0.25	-0.05	-5.60
73	- 6.53	- 63.65	0.23	5.70	-0.02	0.32	0.14	-7.29
74	- 4.49	- 29.16	0.14	0.91	0.63	-0.15	0.12	-3.22
94	- 3.49	4.93	0.12	-3.94	-0.01	0.	-0.16	-0.06
97	- 4.96	- 40.64	-0.32	3.10	-0.06	0.01	-0.02	-1.01
99	- 5.82	- 46.02	-0.62	2.36	-0.13	-0.07	0.05	-2.17
100	- 7.40	- 59.79	-0.87	1.11	-0.16	0.33	0.20	-2.67
101	-11.74	-102.48	-0.86	2.91	0.66	1.35	0.27	3.58

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.
Values computed by finite-element analysis of cracked shell.

Sheet 1 of 8

GILBERT ASSOCIATES, INC.

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 2
Vertical Prestress (FINAL)

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	-18.19	-105.15	1.66	1.57	-0.08	0.26	-0.0	0.50
22	-16.72	-128.36	1.70	4.67	1.14	0.48	-0.27	3.36
33	-11.49	- 54.12	1.29	17.71	1.21	-1.75	0.14	10.10
44	- 7.34	- 12.24	0.64	13.36	1.48	-1.74	0.14	9.32
55	- 4.09	2.50	0.98	18.31	1.39	-0.67	0.09	3.26
66	- 2.10	- 5.56	2.14	29.98	1.55	0.03	-0.68	3.01
77	- 0.51	- 14.23	5.59	59.5	0.24	1.61	-1.28	- 6.66
25	-31.80	-207.2	1.40	19.38	0.39	1.57	-0.49	-39.14
49	-32.93	-258.52	2.07	16.50	1.60	1.82	-0.33	-22.81
73	-40.68	-389.59	2.44	34.18	3.97	0.38	0.84	-25.94
74	-29.51	-172.09	-3.03	- 8.08	6.09	-0.91	1.0	-21.27
94	-22.78	29.11	3.26	-15.84	0.05	0.02	-0.56	- 0.22
97	-35.03	-322.31	2.50	10.80	0.20	0.28	-0.21	- 1.75
99	-39.93	-311.45	-0.33	4.15	0.60	-0.52	-0.53	- 1.67
100	-48.64	-352.39	-3.76	8.52	0.83	1.95	-0.05	- 2.73
101	-69.91	-529.52	-5.85	22.70	5.28	5.88	-1.11	22.15

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Values computed by finite-element analysis of cracked shell.

Sheet 2 of 8

GILBERT ASSOCIATES, INC.

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 3
Operating Temperature (WINTER)

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	- 8.70	123.86	1.73	51.13	- .01	-1.49	0.02	- 0.36
22	- 8.12	180.47	- .80	50.39	.18	-1.69	.09	- 3.02
33	- 7.54	197.31	- 1.01	85.14	.31	-1.57	0.06	- 7.46
44	- 6.35	190.36	- 8.21	37.54	0.55	-1.45	0.04	-11.47
55	- 4.86	167.34	-13.01	9.12	0.89	-1.08	-0.40	-11.14
66	- 2.75	172.01	-19.78	- 35.85	1.20	1.23	2.60	-20.60
77	- 0.20	128.64	-32.56	-123.57	2.67	-2.87	3.75	-23.16
25	- 3.78	363.61	- 3.82	36.28	-3.50	-2.56	0.21	-44.20
49	- 5.61	454.02	- 3.94	39.53	-4.51	-1.65	-0.32	-92.31
73	-24.92	357.52	- 8.59	- 1.10	0.79	2.37	0.13	-80.09
74	-30.11	-233.75	-13.83	- 45.07	9.72	0.08	-0.48	-74.01
94	5.45	162.68	- 2.75	33.64	-0.07	0.03	-0.53	- 0.80
97	- 2.17	622.03	- 3.68	27.20	-0.26	-0.69	-0.86	- 8.25
99	-18.67	540.17	- 5.02	34.79	-0.36	-0.32	-2.04	-22.36
100	-38.57	422.50	- 5.27	37.52	0.30	0.95	-2.95	-24.41
101	-80.37	94.07	- 2.78	25.19	5.77	3.53	4.62	-24.39

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Values computed by finite-element analysis of cracked shell.

Sheet 3 of 8

GILBERT ASSOCIATES, INC.

3B-49

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 4
Accident Temperature

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	-1.45	-13.56	.76	1.57	- .06	.15	- .01	0.25
22	-1.73	8.88	.94	1.21	-0.11	.12	- .09	0.05
33	-2.11	7.21	2.55	20.71	- .16	.05	- .10	- 0.86
44	-2.86	8.72	3.38	27.85	- .34	- .18	- .14	- 3.27
55	-3.65	10.24	4.80	37.39	- .53	-0.05	-0.40	- 6.38
66	-4.60	15.51	6.02	49.72	- .82	0.11	-0.30	- 8.04
77	-5.84	15.94	7.93	68.66	-1.28	1.95	0.47	-13.56
25	.09	3.95	.29	1.76	- .55	- .16	- .05	- 2.29
49	.21	8.12	.05	3.71	-1.43	- .06	- .13	- 2.22
73	1.27	11.80	- .70	- .10	-3.00	.64	.03	- 2.24
74	.14	24.26	- .43	-10.75	-4.81	-2.02	- .36	- 5.93
94	.49	- .28	- .51	- 2.76	- .03	0.	- .18	- .02
97	1.91	15.73	-1.37	3.63	- .20	- .08	.17	- .54
99	3.06	- 2.51	-2.48	5.98	- .36	.13	.66	1.20
100	4.78	-24.63	-3.56	4.04	- .90	0.43	1.61	4.07
101	5.53	-71.13	-5.03	- 5.38	-2.09	-0.88	3.53	6.61

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Values computed by finite-element analysis of cracked shell.

Sheet 4 of 8

GILBERT ASSOCIATES, INC.

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 5
69 psi Internal Pressure

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	20.02	264.15	39.59	22.93	0.52	6.21	- 0.20	5.63
22	25.52	96.70	34.32	- 12.86	1.34	3.30	- 1.00	15.99
33	27.82	10.82	69.99	481.80	0.33	1.96	- 1.08	22.40
44	26.42	30.86	76.32	517.73	- 1.13	- 0.33	- 0.64	11.04
55	22.91	50.91	87.83	596.92	- 2.14	5.20	- 3.33	1.93
66	20.79	154.48	100.23	685.26	- 3.81	6.34	- 0.69	- 1.64
77	14.63	170.28	121.52	848.44	-11.12	19.25	1.79	- 73.84
25	24.54	94.49	44.14	33.84	0.29	- 2.69	0.43	100.47
49	20.91	33.51	42.34	46.48	- 4.90	3.68	- 4.76	22.94
73	25.42	8.15	48.55	396.22	-22.52	2.75	- 1.95	-157.81
74	29.33	43.58	53.02	567.11	-28.25	-15.01	- 2.21	-154.40
94	20.53	- 86.57	42 07	19.85	- 0.05	0.05	- 0.66	- 0.37
97	35.56	223.25	29.19	- 29.24	- 2.29	- 3.11	1.61	- 16.85
99	41.39	193.13	15.10	4.38	- 3.36	- 1.31	1.11	- 17.92
100	45.71	182.31	8.48	7.61	- 4.57	- 1.98	2.81	- 32.13
101	39.72	59.79	0.04	2.66	- 5.25	4.79	11.59	- 19.30

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Values computed by finite-element analysis of cracked shell.

Sheet 5 of 8

GILBERT ASSOCIATES, INC.

TABLE 4.5

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 6
Effect of Bond Loss Associated With 69 psi

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	3.92	62.18	- 0.25	0.91	0.04	0.69	- 0.03	0.56
22	4.34	51.42	- 0.21	- 3.85	- 0.01	0.09	- 0.09	2.61
33	3.99	42.07	- 0.56	-17.27	- 0.05	0.46	0.	3.22
44	3.69	39.12	0.44	-12.62	- 0.19	0.52	- 0.14	1.56
55	3.08	36.96	1.67	- 5.30	- 0.39	0.69	- 0.45	2.02
66	2.69	51.66	3.12	4.48	- 0.77	0.65	0.62	- 1.10
77	1.85	48.44	4.19	12.59	- 0.79	0.78	1.45	- 6.88
25	- 1.02	14.36	- 0.69	- 6.02	- 1.02	0.77	0.09	2.82
49	- 4.21	- 55.67	- 1.94	8.79	- 1.39	0.79	0.13	- 7.98
73	- 6.74	-111.31	- 5.27	- 3.53	- 3.62	- 1.24	- 1.55	9.91
74	7.24	67.19	0.38	-15.12	- 1.29	2.25	- 3.59	53.00
94	- 1.97	- 10.08	3.83	34.02	0.13	- 0.97	1.39	0.44
97	7.09	- 0.94	-13.70	- 0.87	6.16	- 0.31	- 0.72	65.16
99	14.07	164.57	-29.94	19.74	0.97	- 1.18	0.44	3.32
100	11.03	82.57	-45.63	26.25	-15.91	-21.56	4.62	-150.49
101	-25.20	-602.32	15.18	12.79	-30.42	44.29	23.33	-126.64

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Values computed by finite-element analysis of cracked shell.

Sheet 6 of 8

GINNA/UFSAR
TABLE 4.3

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 7
Earthquake #1

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	- 3.80	- 21.08	- .109	.018	0.127	0.036	0.	- 0.18
22	- 3.39	- 25.39	-0.309	- 0.018	0.182	0.055	-0.073	1.36
33	- 2.26	- 6.01	-0.309	- 3.15	0.273	-0.437	0.055	2.90
44	- 1.31	3.17	0.419	2.48	0.364	-0.400	0.036	2.98
55	- 0.491	5.75	1.20	8.83	0.346	0.018	-0.055	1.27
66	0.	4.70	2.07	16.22	0.328	0.200	-0.164	0.88
77	0.291	1.02	3.73	29.76	-0.182	0.928	-0.309	- 3.65
25	- 9.10	- 59.26	0.273	6.52	-0.200	0.382	-0.182	-14.70
49	- 9.45	- 73.18	0.419	7.35	-0.291	0.455	-0.091	-10.70
73	-11.88	-115.84	0.419	10.37	-0.036	0.582	0.255	-14.00
74	- 8.17	- 53.07	0.255	1.66	1.15	-0.273	0.218	- 6.17
94	- 6.35	8.97	0.218	- 7.17	-0.018	0.	-0.291	- 0.12
97	- 9.03	- 73.96	-0.582	5.64	-0.109	0.018	-0.036	- 1.93
99	-10.59	- 83.76	-1.13	4.30	-0.237	-0.127	0.091	- 4.18
100	-13.47	-108.81	-1.58	2.02	-0.291	0.601	0.364	- 5.14
101	-21.37	-186.51	-1.57	5.30	1.20	2.46	0.491	6.85

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Sheet 7 of 8

TABLE 4.3

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 8
Earthquake #2

	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears	Twisting Moment
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ	$M_{\phi\theta}$
11	- 0.49	- 2.52	0.	0.	5.0	NEGLIGIBLE	NEGLIGIBLE	NEGLIBIBLE
22	- 0.43	- 3.26	- 0.04	0.	5.0			
33	- 0.28	- 0.77	- 0.04	-0.4	4.0			
44	- 0.17	0.41	0.05	0.3	3.0			
55	- 0.06	0.74	0.2	1.1	2.0			
66	0.	0.61	0.3	2.1	1.0			
77	0.	0.14	0.5	3.8	0.			
25	- 1.2	- 7.6	0.	0.8	0.			
49	- 4.2	- 9.2	- 3.	1.	3.0			
73	- 6.5	-14.8	- 6.	1.3	6.0			
74	-11.7	- 6.8	-10.6	0.2	10.9			
94	- 0.8	1.1	0.	-0.9	5.			
97	- 1.16	- 9.4	- 0.1	0.7	4.			
99	- 1.37	-10.8	- 0.2	0.5	3.			
100	- 1.73	-14.	- 0.2	0.2	2.			
101	- 2.74	-24.	- 0.2	0.7	0.			

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, M_θ, and $M_{\phi\theta}$ in Kips.

Sheet 8 of 8

GILBERT ASSOCIATES, INC.

Table 5-1
Maximum Liner Stresses Stress tangent to the edge in Ksi

Element	Load#	σ_t [1]	σ_t [2]
77	19	+34*	+55*
74	20	+25	+36*
101	25	-23	-7

Note:

[1] Composite action neglected

[2] Composite action included

* As a measure of strain

APPENDIX A TO APPENDIX 3B

EFFECT OF CONCRETE CREEP AND THE SUSTAINED OPERATING STRESSES ON STRESS DISTRIBUTION AROUND OPENINGS IN A RAPIDLY PRESSURIZED REINFORCED CONCRETE VESSEL

3B.A EFFECT OF CONCRETE CREEP AND THE SUSTAINED OPERATING STRESSES ON STRESS DISTRIBUTION AROUND OPENINGS IN A RAPIDLY PRESSURIZED REINFORCED CONCRETE VESSEL

Consider the simple structure shown below:

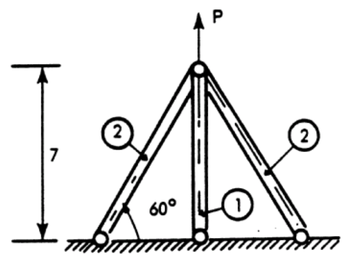

Figure A-1

Column (1) is a reinforced concrete column with net concrete area A_c^1 and longitudinal steel Area A_{st}^1 . A_c^2 and A_{st}^2 denote the net concrete area and the longitudinal steel area, respectively, of reinforced concrete columns (2). The system is loaded at time t = 0 with a vertical load P. Let us determine the initial load distribution:

$$P = T_1 + 2T_2 \sin 60° = T_1 + \sqrt{3}\ \ T_2 \quad \text{(Equation 1)}$$

$$\Delta L_2 = \Delta L_1 \sin 60°; \quad \Delta L_2 = \frac{\sqrt{3}}{2}\ \Delta L_1 \quad \text{(Equation 2)}$$

in which $$\frac{\Delta L_1}{L} = \frac{T_1}{A_c^1\ E_c + A_{st}\ E_{st}^1} \quad \text{(Equation 3)}$$

$$\frac{\sqrt{3}}{2}\ \frac{\Delta L_2}{L} = \frac{T_2}{A_c^2 E_c + A_{st}^2\ E_{st}} \quad \text{(Equation 4)}$$

E_c and E_{st} denote the "effective" modules of elasticity of concrete and steel, T_1 and T_2 the loads carried by columns (1) and (2), respectively. From equations (1) to (4) we obtain:

$$T_1 + \sqrt{3}\ T_2 = P$$

$$\frac{\sqrt{3}}{2}\ \frac{\Delta L\ T_1}{A_c^1 E_c + A_{st}^1 E_{st}} - \frac{2}{\sqrt{3}}\ \frac{L\ T_2}{A_c^2 E_c + A_{st}^2 E_{st}} = 0$$

or $$\begin{Bmatrix} T_1 \\ T_2 \end{Bmatrix} = \begin{Bmatrix} \dfrac{\beta}{1.5\alpha + \beta} \\ \dfrac{\sqrt{3}}{2}\ \dfrac{\alpha}{1.5\alpha + \beta} \end{Bmatrix} P$$ (Equation 5)

In which

$$\alpha = \frac{L}{A_c^1 E_c + A_{st}^1 E_{st}}$$

$$\beta = \frac{2}{\sqrt{3}}\ \frac{L}{A_c^2 E_c + A_{st}^2 E_{st}}$$ (Equation 6)

Equation (5) gives the loads acting on columns (1) and (2) in terms of the "effective" modulae E_c and E_{st}.

In general, T_1 and T_2 will change with time (load redistribution) due to concrete creep. Under the assumption that concrete behaves like a Kelvin-type material, the load distribution may be calculated exactly for $t \to \infty$ by resorting to the creep-limit modulus E_{cu}. (Time-dependent behavior of steel is neglected.)

Note that even when there is no load redistribution (for example, when $\alpha = \beta$) there will be some stress redistribution. In other words, T_1 and T_2 may remain constant, but the percentage of both carried by the steel reinforcement will increase as concrete creeps. As a result, steel stresses will increase and concrete stresses will decrease to final values which may be easily computed.

Let us now assume that at time t_1 a load P_1 is superimposed on the existing load P_0 giving a total load $(P_0 + P_1)$. (See Figure A-1). Let T_1 and T_2 be the column loads immediately before the load P_1 is applied. To compute the actual column loads T_1 and T_2 after P_1 is applied, we would be tempted to determine the column loads T_1* and T_2* corresponding to P_1 acting alone on the structure (On the basis of the initial modulus of elasticity) and then add them to T_1^1 and T_2^1:

$$T_1^2 = T_1^1 + T_1^*$$ (Equation 7)

$$T_2^{\,2} = T_2^{\,1} + T_2^{\,*} \quad \text{(Equation 8)}$$

The approach is valid if there is no concrete cracking. If either column (1) or (2) (or both) cracks due to the resluting tensile stresses, the results obtained by resorting to equation (7) will be incorrect. The situation will be best illustrated by an example. Let:

E_c^0 = initial modulus of elasticity of concrete = 4000 ksi

E_c^u = "final" effective modulus of elasticity of concrete = 2000 ksi

E_{st} = 7.5 E_c^0 = 3000 ksi

A_c^1 = 100 in.^{2}

A_c^2 = 100 in.^{2}

A_{st}^1 = 2 in.^{2}

A_{st}^2 = 4 in.^{2}

L = 1000 in.

A load P_0 = -200 kips is applied at t = 0 and kept constant.

At time t = t_1 a second load P_1 = +300 kips is applied.

Initial Load Distribution under P = P_0

$$\alpha_i = \frac{1000}{100 + 2 \times 7.5}\,\frac{1}{E_c^{\,0}} = \frac{8.69}{E_c^{\,0}} \quad \text{(Equation 9)}$$

$$\beta_i = \frac{2}{\sqrt{3}}\,\frac{1000}{100 + 4 \times 7.5}\,\frac{1}{E_c^{\,0}} = \frac{8.875}{E_c^{\,0}} \quad \text{(Equation 10)}$$

$$T_1^{\,0} = \frac{8.875}{1.5 \times 8.69 + 8.875}\,P_0 = 0.404P_0$$

$$T_2^{\,0} = \frac{8.69 \times 0.866}{1.5 \times 8.69 + 8.875}\,P_0 = 0.343P_0 \quad \text{(Equation 11)}$$

Load Distribution under P = P_0 Immediately Before Application of P_1 ($t_1 \to \infty$)

$$\alpha_f = \frac{1000}{0.5 \times 100 + 2 \times 7.5} \; \frac{1}{E_c{}^0} = \frac{15.4}{E_c{}^0}$$

$$\beta_f = \frac{1.155 \times 1000}{0.5 \times 100 + 4 \times 7.5} \; \frac{1}{E_c{}^0} = \frac{14.45}{E_c{}^0}$$ (Equation 12)

$$T_{11}^1 = \frac{14.45}{1.5 \times 15.4 + 14.45} \; P_0 = 0.385 P$$

$$T_2^1 = \frac{15.4 \times 0.866}{1.5 \times 15.4 + 14.45} \; P_0 = 0.355 P_0$$ (Equation 13)

Concrete Stresses Due to P = P_0

Initial Stresses
$$\begin{cases} f_{c_1}{}^0 = \dfrac{\alpha_i T_1 E_c{}^0}{L} = -704 \; psi \\ f_{c_1}{}^0 = \dfrac{\beta_i T_2 E_c{}^0}{L} = -608 \; psi \end{cases}$$ (Equation 14)

Final Stresses ($t_1 \to \infty$)
$$\begin{cases} f_{c_1}{}^{-1} = \dfrac{\alpha_f T_1 E_c{}^u}{L} = -598 \; psi \\ f_{c_2}{}^{-1} = \dfrac{\beta_f T_2 E_c{}^u}{L} = -514 \; psi \end{cases}$$ (Equation 15)

Consequently, if t_1 is large, concrete stresses immediately after P_1 is applied would be:

$$f_{c_1}{}^{1+} = -598 + 1055 = +457 \; psi$$

$$f_{c_2}^{1+} = -514 + 912 = +398 \; psi$$

(Equation 16)

If the tensile strength of concrete is f_t = 420 psi, then column (1) should be expected to crack, i.e., the entire load T_1 would be carried by the steel reinforcement. Under such condtions, a load redistribution would occur, resulting in a large increase in T_2, with susequent cracking of columns (2) as well. The final load distribution will therefore depend on A_{st}^1 and A_{st}^2 only. That is to day, if both columns crack it is irrelevant whether t_1 is small or large. Moreover, A_c^1 and A_c^2 will no longer play any role in the problem. In fact:

$$\alpha^{1+} = \frac{1000}{2 \times 30000} = 0.01667$$

$$\beta^{1+} = \frac{1.155 \times 1000}{4 \times 30000} = 0.00962$$

(Equation 17)

$$T_1^{1+} = \frac{0.00962 \; P}{1.5 \times 0.01667 + 0.00962} = 0.278 P$$

$$T_2^{1+} = \frac{0.01667 \times 0.866}{1.5 \times 0.01667 + 0.00962} = 0.416 P$$

(Equation 18)

It has been shown that if our sample structure is fully cracked, then creep has no influence whatsoever on the final load distribution. It may be hypothesized, however, that the concrete tensile strength in Column (2) is higher, say ft = 1500 psi. The question is then asked, what is now the load distribution. The problem may be solved by computing the total load "C" carried by concrete in Column (1) at time shortly before t = t_1, and assuming that, as Column (1) cracks, load will be transferred simultaneously to joint and to the steel of column (1).

Initial Load Distribution with Concrete in Column (1) Fully Cracked

$$\alpha_c = \frac{1000}{2 \times 7.5} \; \frac{1}{E_c^0} = \frac{66.6}{E_c^0}$$

$$\beta_c = \frac{1.155 \times 1000}{100 + 4 \times 7.5} \frac{8.875}{E_c{}^0}$$ (Equation 19)

$$T_1 = \frac{8.875}{1.5 \times 66.6 + 8.875} P = 0.082\ P$$

$$T_2 = \frac{0.866 \times 66.6}{1.5 \times 66.6 + 8.875} P = 0.531\ P$$ (Equation 20)

$$T_1 = 0.385\ P_0 + 0.082\ (P_1 + C) - C$$

$$T_2 = 0.355\ P_0 + 0.531\ (P_1 + C)$$ (Equation 21)

with C = -.598 x 100 = -59.8 kips we get:

$$\left.\begin{aligned} T_1 &= +2.6\ kips \\ T_2 &= +56.9\ kips \end{aligned}\right\} \quad t_1 \rightarrow \infty$$ (Equation 22)

If t_1 is sufficiently small, it may be assumed that P_0 and P_1 are applied simultaneously at t = 0, in which case:

$$\left.\begin{aligned} T_1 &= 0.082 \times 100 = +8.2\ kips \\ T_2 &= 0.531 \times 100 = +53.1\ kips \end{aligned}\right\} \quad t_1 = 0$$ (Equation 23)

The difference between (22) and (23) is not large. Note that superimposing the load distribution after creep due to P_0 (equation 13) with the load distribution corresponding to the cracked structure under P_1 leads to:

$$T_1 = -0.385 \times 200 + 0.082 \times 300 = -42.4\ kips$$

$$T_2 = -0.355 \times 200 + 0.531 \times 300 = 88.3\ kips$$ (Equation 24)

which are entirely unrealistic figures. Note also that an "exact" stress analysis for the case when t_1 is large (equations (19) and (20)) would not be feasible for moderately complex structures.

APPENDIX B TO APPENDIX 3B

EARTHQUAKE ANALYSIS

3B.B EARTHQUAKE ANALYSIS

The computation of seismic stresses was carried out on the basis of the fundamental mode of the containment structure associated with maximum response. The peak of the response curve (=0.47g) for 2 percent critical damping and 0.2g peak ground acceleration was used to determine:

1. The stress-resultant N_{ϕ} at the center of the opening (in the shell without the opening) for the horizontal component of earthquake action oriented in the direction normal to the openings.
2. The in-plane shear stress-resultant at the center of the opening (in the shell without the opening) for the horizontal component of earthquake notion oriented at 90° with the direction normal to the opening.
3. The stress-resultant $N_{\phi\theta}$ at the center of the opening (in the shell without the opening) for the vertical component of motion associated with 0.2g peak ground acceleration.

The influence of the opening on the above seismic loads was evaluated as follows:

a. The stress-resultant and stress-couple distributions and, therefore, the stress-concentration factors corresponding to (1) and (3) were conservatively computed on the basis of the finite-element results for dead load.
b. The stress concentration factors corresponding to (2) were determined on the basis of Lakerkerker's solution[11]. (See Figure 3) for a shell with a hole subjected to torsion, i.e., to a pure membrane shear $N_{\phi\theta}$ at the location of the opening, (in the shell without the opening). Note that the stress concentration factors are slightly larger than those corresponding to the plate solution. In computing stresses at elements away from the edge of the opening, however, the stress concentration factor was assumed to decrease as in the plane solution.

In determining the values shown in Table 4-3, the absolute value of the contribution of the vertical component of motion (3) was added to the absolute value of the contributions of the two horizontal components [(1) and (2)].

ADDENDUM TO APPENDIX 3B

ADDENDUM TO THE REPORT ON: DESIGN OF LARGE OPENING REINFORCEMENTS FOR CONTAINMENT VESSEL

OCTOBER 16, 1968 ADDENDUM TO GAI REPORT NO. 1683

Robert E. Ginna Nuclear Power Station

J. D. Riera, Ph.D.
D. K. Croneberger
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3B.C INTRODUCTION

The analysis and design of reinforcement for the large openings in the containment vessel for the Robert Emmett Ginna Nuclear Plant were described in the Third Supplement to the Final Facility Description and Safety Analysis Report (FSAR). This addendum to the aforementioned report provides supplemental information, including certain construction procedures, and additions or corrections to the basis report. The final design is reflected on the attached revised Drawings D-421-023 (Figure Drawing 2), and D-421-024 (Figure Drawing 1).

1 DESIGN

1.1CONCRETE SHEAR

Splitting planes were hypothesized parallel to the surface of the shell through the various 1ayers of concrete reinforcement and tendon conduit. The in plane shear stresses are produced by the interrupted horizontal reinforcing bars as well as by radial forces produced by elliptical rebar rings and draped tendons. Sufficient steel has now been provided in the form of straight or hooked radial bars and ties to develop the total shear stress across the hypothesized planes. The shear stresses are conservatively assumed to be the summation of the loads resisted by the elliptical bars on the vertical axis due to the factored pressure load. That is to say, the shearing force exerted across a plane through Layer 6 (see attached Drawing No. D-421-024, Figure Drawing 1) is equal to the summation of rebar forces on Layers 6 and 7 on the vertical axis. The maximum shear stress on the dowels due to the aforementioned load does not exceed the yield stress of the dowels. The dowels are anchored by mechanical anchorage (180° or 90° hooks) and/or sufficient bond development length which is determined on the basis of Ultimate Strength Design provisions of ACI 318-63. All rebars provided to resist the aforementioned loads consist of A15 material with a 40 ksi minimum guaranteed yield strength.

1.2INTERACTION DIAGRAMS

For derivation of Equations (5.1) through (5.5) refer to:

a. ACI 318-63 chapters 16 and 19.

b. "Design of Concrete Structures" by 0. Winter et. al., chapter 5.

It should be pointed out that all points on the interaction diagram are computed with respect to shell reference surface as shown in Figures 19 through 23. That is, under compression and bending, "N" and "H" were transferred from the plastic centroid to the shell reference surface. Under tension, "N" was transferred from the center of gravity of the reinforcing steel to the shell reference surface.

1.3EARTHQUAKE DESIGN

The stresses due to earthquake motions, as described in section 4.3.2c on Page 29 and in Appendix B of the report, were determined on the basis of the acceleration response spectra for 0.20g maximum ground acceleration (Figure 5.1.2-8 of the FSAR) and the resultant load diagram in the form of a triangular distribution with the base of the triangle at the top of the structure.

1.4THERMAL GRADIENTS

The thermal gradients described in section 1.2 on Page 2 and Figure 12 of the report are based upon steady stare (operating) conditions.

1.5PENETRATION MATERIAL

As stated in section l.4d on Page 7 of the report the penetration materials (steel plate) conform to ASTM A516 Grade 60 Firebox Quality modified to ASTM A300. The steel plate has a nil ductility transition temperature, as measured by a Charpy V-notch specimen of at least 30° F below the minimum service metal temperature. This requirement on NDTT applied to all penetration materials as well as the liner plate.

1.6WORKING STRENGTH DESIGN

The load combinations listed in section 1.3 on page 3 and in Table 4-1 of the report are based upon an ultimate strength design approach. In addition, the load combinations were considered in a working strength design approach (i.e., load factors equal 1.0) when the stress/strain criteria is established by the ASME Nuclear Vessels Code and Chapter 26 - Prestressed Concrete of ACI 318-63. For the design of the opening reinforcement, the items for which working strength design therefore applied included:

a. Liner plate and penetration barrel.

b. Anchorage of interrupted horizontal bars (concrete bearing stresses).

1.7ANCHORAGE PLATE BEARING STRESS

The concrete bearing stress, as defined by Equation (5.8) on page 58 of the report, shall not exceed the compressive strength of the concrete when the load is applied, which for purposes of this design is assumed to be the 28 day compressive strength. The calculated bearing stress is 3640 psi which is less than the allowable value of 3670 psi determined by Equation (5.8).

It should be noted that the calculated bearing stress is based upon the factored pressure load while the allowable stress is on the basis of a working strength design.

1.8INSULATED LINER TEMPERATURE INCREASE

The change in liner temperature of 2°F, as referred to in section 1.3 on page 4 of the report, represents the mean temperature rise from normal operating conditions to the time associated with the maximum pressure as shown on the transients for the factored pressure (90 psig). Verification of the capability of the insulation to restrict the liner temperature change to this specified value is described in Appendix 5B of the FSAR.

1.9HIGH STRENGTH REBAR

The use of 60 ksi rebars was basically restricted to the immediate vicinity of the stress concentration (i.e., the elliptical bars and the horizontal bars draped around the hole). Specific requirements for 60 ksi material (A432) are shown on attached Drawing No. D-421-023 (Figure Drawing 2).

1.10PROOF TEST INSTRUMENTATION

Measurements of displacements, strains, and cracking about the opening for the equipment access hatch will be obtained as follows:

a. Displacement Measurements

Horizontal and vertical displacements of the reinforced area around the opening will be obtained with linear variable differential transformers (LVDT) mounted on a structure not affected by the test. On the horizontal axis, on one side only, six horizontal and vertical displacements will be obtained at equally spaced locations extending from a location two feet from the edge of the opening to a location twenty-one feet from the edge of the opening.

On the vertical axis, on the top side only, the quantity and locations of measurements to be obtained will be identical to that described herebefore. On the horizontal axis, on the opposite side previously mentioned, two horizontal and vertical displacements will be obtained at one location two feet from the edge of the opening and another location seven feet from the edge of the opening.

b. Strain Measurements

Horizontal and vertical strains will be measured on the rebar nearest the exterior concrete face at those locations described for displacement measurements.

c. Concrete Crack Measurements

One upper quadrant of the area around the opening extending from the edge of the hale to a line 3 ft - 6 in. outboard of the thickened concrete portion of the shell will be coated and detailed measurements made of spacing and width of cracks.

1.11 OPERATING CONDITIONS

Refer to Table 4-1 on page 35 of the report. The first four load combinations reflecting normal operating conditions are based on an uncracked shell.

The computer output for Element No. 77 resulted in:

$N_\theta =$ + 6.5 K/in. for Load Comb. (2)

$N_\theta =$ - 15.6 K/in. for Load Comb. (4)

Maximum tensile stresses will be:

$\sigma =$ 155 psi in the concrete for uncracked concrete, and

$\sigma =$ 1625 psi in the rebar for cracked concrete.

1.12SHEAR - DIAGONAL TENSION

The ACI 318-63 recognizes the punching shear to be critical at a distance $\frac{"d"}{2}$ out from the periphery for slabs and footings (See Section 1207 and 1707). A beam type of diagonal tension failure is impossible due to the geometry and two directional stresses in the shell. We believe that a punching mode of failure is the type of failure that should be and has been investigated. These shear stresses included in the computer output take into account the effect of the pressure on the door plus the reinforced area.

1.13NORMAL SHEARS

The average normal shears at the edge of the thickened (reinforced) area are:

Top horizontal edge:v = 38 psi

Vertical edge: v = 29 psi

1.14RADIAL SHEAR AT THE PERIPHERY OF THE OPENING

The assumption of a uniform radial shear at the periphery of the opening was made solely for the finite element method of stress analysis. This assumption was judged to be reasonable, as the radial displacements about the periphery of the hole are essentially constant. Furthermore, a variation of the radial shears was made and was found to have little effect on the resulting stress resultants and stress couples. Nevertheless components including the shear ring and reinforcement for diagonal tension were designed for a radial shear twice the computed value based upon a uniform distribution.

1.15ACCIDENT TEMPERATURE EFFECTS

Item 2(b), page 32 of the report, indicates that the concrete around the opening was considered to be uniformly heated to 192°F, to a depth of 0.55 in. This is an approximation, arrived at by using the area under a step gradient equal to the area under the actual gradient remote from the boundaries (i.e., the inside or outside faces of the wall).

1.16ANALYTICAL MODEL FOR DIFFERENT LOAD COMBINATIONS

The coefficients listed for the various load combinations are developed on the basis of the absolute values used in the stress analysis, which were dead load, final prestress (0.60 f_s), test pressure (p = 69 psig), accident temperature based on factored pressure load (T = 312°F), and 0.20g ground acceleration. All load combinations representing operating conditions are based on the uncracked model. All other load combinations are based on the same cracked model. An inspection of the stress resultants and stress couples for various cracked models indicated that this approach is valid, in that changes in the cracking pattern did not significantly alter the stress resultants and stress couples.

1.17SHEAR REINFORCEMENT

$$v_u = \phi\left(A_{sv}f_y + A_{sd}\frac{f_y - f_r}{2}\right)$$ (Equation 5.7)

The first term within the bracket is the "stirrup" effect to resist diagonal tension. The cross-sectional area of "A_{sv}" must be properly anchored in order to be considered effective. In the actual design the effect of the first term ii found to be negligible.

The second term is intended to represent the dowel action of reinforcing steel intersecting a potential crack. Tests on studs [a] have indicated that the ultimate shear capacity is equal to the

a. Nelson Stud Welding Manual No. 21, August 1, 1961, Gregory Industries, Inc.

ultimate tensile capacity of the steel. Further tests on rebar and discussion of the shear-friction hypothesis[a] indicate that the steel normal to a crack will act in tension and that for shear across a rough concrete to concrete interface:

$$A_{sd} = \frac{v_u}{1.4 f_y}$$

Therefore, the second term in equation (5.7) is only 35 percent of the value predicted by the shear friction hypothesis. This conservatism was employed to minimize the amount of ship-page required to develop the capacity at the hypothesized crack.

The out-of-plane shear stresses in the meridional direction and in the hoop direction t_ϕ were combined as a resultant shear stress acting in the plane of the shell. That is:

$$\tau_{in\ plane} = (\tau_\phi^2 + \tau_\theta^2)^{1/2}$$

Tee's and straight bars were provided to carry this shear by steel alone, that is, assuming the concrete to be cracked. It should be noted that the concrete stress, so computed, $\tau_{\text{in plane}}$ is less than $\sqrt{c} = 4\,\phi\sqrt{f_c'}$.

The meridional shear along the vertical axis can be resisted by unreinforced concrete. However, sufficient reinforcement has been provided to carry this shear by steel alone.

The hoop shear along the horizontal axis has been investigated and sufficient reinforcement provided to carry this shear by steel alone.

1.18EQUATION (5.11)

Equation (5.11) on page 60 should be as follows:

$$f_{si} = \frac{f_x i}{A_s}$$

1.19REBAR LOCATED AWAY FROM THE BARREL

The average clearance between elliptical steel and barrel is only 11 in. or about 17 percent of the shell thickness. The elliptical reinforcing steel arrangement around the access barrel will:

a. R. F. Mast: "auxiliary Reinforcement in Concrete Connections," Journal ASCE, Vol. 94, No. ST6, June 1968, pp. 1485-1504.

a. Punish enough reinforcing steel close to the barrel on the top (Element No. 77) to resist the high hoop tension wider pressure load, and
b. Provide enough space between the barrel and the first ring for anchorage of the terminated hoop steel.

The maximum distance from the barrel to the first elliptical ring is at the horizontal axis, 18 in. The stresses at this point (Element No. 101) are all compressive. See interaction diagram Figure 21 of the report.

For stresses on the top (Element No. 77), see Figure 22 of the report. For stresses on the 45 degree axis from the horizontal (Element 73 and 74), see Figure 19 of the report.

1.20VERIFICATION OF ANALYSIS

Hansen, Holley and Biggs (H.H.&B.) as consultants to Rochester Gas & Electric Corporation did make limited independent checks on the GAl analysis described in the report. The scope of the H.H.&B. analysis was as follows, with all models based upon uncracked plain concrete:

a. Using two separate programs, H.H.&B. analyzed the GAl test problem (refer to Section 4.1 of the report); Prato's program [reference 16 of the report, (Mixed Finite Element Method)] and Rodriguez's program (Displacement Method) gave results in good agreement with each other and with the other results included in the report.
b. Prato's program was thereafter used for the solution of the following canes based upon the actual containment cylinder and opening radii and a Poisson's ratio of 0.15:
    1. An internal pressure of 90 psig with loading on the perimeter of the opening limited to the component along the opening axis (i.e., no component normal to the opening was considered). The shell was considered to be of constant thickness (42 in.), with a modulus of elasticity of 4.0×10^6 psi.
    2. Same as (1) except that the shell was thickened to 66 in. in the vicinity of the opening within a boundary which varied from 119 in. to 144 in. from the edge of the opening chosen to follow the boundary of selected elements.
    3. Same as (1) above except that the constant thickness was reduced to 4.2 in. and the modulus of elasticity was increased to 40×10^6 psi. These changes were intended to indicate the sensitivity of the analysis to a greatly reduced bending stiffness with no change in the membrane stiffness.
    4. Same as (1) above, except that internal pressure is not considered and the loading parallel to the axis of the opening was defined as "pr/4 cos 2θ" where "p" equals 90 psig and "r" equals the opening radius of 85.7 in.
    5. Same as (4) above, except that the loading parallel to the axis of the opening was defined as "pr/4 sin 2θ

Stress resultants "N_θ" along the horizontal and vertical axes of symmetry are compared on the attached Figure I with similar results obtained by the GAl analysis. This comparison is based on the models described in (1) and (2) above. The correlation is excellent.

Even in the extreme case investigated to determine sensitivity to reduced bending stiffness (Case No. 3), sufficient rebar is available without yielding to resist the calculated stress resultants (N_θ) due to the pressure load commencing at a distance 10 in. from the edge of the opening. Sufficient rebar is available even considering the high stress resultant at the edge of the opening if the stress resultants are integrated for a distance approximately 24 inches from the opening edge. Cases Nos. 4 and 5 were reported to indicate that the solution is not sensitive to variations in the radial load distribution at the edge of the opening.

1.21 TEST PROBLEM

On page 21 of the report, the coefficient "ν" for the test problem is 0.62 instead of 1.17. The coefficient "ν" for the R. E. Ginna Containment Vessel is $\mu = 0.34$ (equipment hatch, based on typical thickness). The conclusion is still valid.

1.22 ACCIDENT TEMPERATURE

Refer to Item 2 on page 32 of the report. Further verification of the numerical results given in the report revealed that the equivalent pressure applied by the barrel (penetration sleeve) on the concrete is 320 instead of 160 psi, when the barrel is heated due to accident temperature. This error affects the stress-resultants and stress-couples for loading condition No. 4 (accident temperature). Therefore, the values indicated in Table 4-3, page 43, should be replaced by those given in the attached Table I. It can readily be seen that at the most critical location, i.e., in element No. 77 and in the hoop direction, the difference is:

$\Delta N_\theta =$ 14.09 - 7.93 = 6.56 K/in.

$\Delta M_\theta =$ 149.08 - 68.66 = 70.42 Kips-in./in.

Inspection of the interaction diagram for element No. 77, Figure 22 of the report shows that displacing the points closer to the edge of the diagram by the amounts computed above, still leaves them well inside the interaction diagram. The proposed design is therefore not affected by the computational error referred to above.

Note, that even if the stress-resultants and stress-couples due to accident temperatures given in the attached Table were doubled, the resulting stress-resultants and stress-couples for all load combinations would still fail within the interaction diagrams.

2 CONSTRUCTION

2.1CONSTRUCTION SCHEDULE

The schedule for placement of concrete is shown on the attached Drawing SS-400-659 (Figure Drawing 3). This drawing also reflects the location of construction joints.

2.2CONCRETE REMOVAL

a. Condition of Old Concrete

Prior to placing concrete which will abut a joint produced by concrete removal, the joint will be thoroughly cleaned with filtered air and water spray to remove all loose material, and an inspection will be made for cracks. Any visible cracks will be removed using hand tools, and patched if necessary. Special attention will be given to detect possible cracks oriented parallel to reinforcing bars which might produce a "splitting type" bond failure when the structure is loaded. The reinforcing bars that were exposed by the concrete removal operation have been or will be thoroughly cleaned to remove bonded concrete or mortar to ensure the bond is achieved with the new concrete. It should be pointed out that the air hammers employed could only remove small particles of concrete and it was impossible to produce significant cracking to ease the operation.

b. Construction Joint Preparation

Horizontal and vertical construction joints were, or are, to be prepared for receiving the next pour by either sandblasting, air/water jet, bush hammering, or other means to remove all coatings, stains, debris, or other foreign material.

On construction joint surfaces in the Containment Vessel, including all vertical joints in the cylindrical shell and all joints in the dome, an epoxy resin (Colma Bonding Compound as manufactured by Sika Chemical Corporation) was, or is, being used. This applies to the vertical joints in the vicinity of the large openings.

The horizontal joints will be dampened (but not saturated), and then thoroughly covered with a coat of neat cement mortar of similar proportions to the mortar in the concrete.

The mortar will be approximately 1/2 inch thick and fresh concrete will be placed before the mortar has attained its initial set.

2.3CONCRETE WORK

a. Pour Limits

Construction joints will be located as shown on the attached Drawing SS-400-659 (Figure Drawing 3). A review was made of the shear stresses at construction joints and it was found that sufficient rebars exist to resist the shears without contribution from the concrete. The maximum concrete lift height is 10 ft - 0 in. and maximum concrete quantity per pour is approximately 63 cubic yards.

b. Concrete Mix and Curing

The basic concrete mix was initially developed to minimize shrinkage. This involved selection of a course aggregate (dolomite) and careful selection of additives (water reducer/ retarder). Special attention has been given to orientation of construction joints to permit adequate venting, thereby making possible a sound interface between new and old concrete. Initial and final concrete curing will be the wet method as specified in ACI 301-66.

2.4RETENSIONING TENDONS

Relaxation losses which account for approximately two-thirds of the total losses in a tendon were estimated, based on "A Study of Stress Relaxation in Prestressing Reinforcement," by D. D. Mogura et. al., PCI Journal, April 1964. In order to provide the required prestress force at the base of the structure at the end of plant life (40 years), it will be necessary to retension those tendons which are draped around the openings for the equipment hatch and the personnel lack and therefore experience higher than typical friction losses. According to the foregoing procedure the minimum time delay for retensioning to ensure required prestress at the base of the cylinder at the end of plant life is 1,000 hours.

The total increase in prestress force at the top of the cylinder due to retensioning the tendons at the equipment access hatch is approximately 750 kips. This force produces negligible changes in concrete stress at the opening.

2.5REBAR SPLICES

The normal procedure for locating bar-to-bar splices in the containment structure was to have no more than one-third of the splices in one plane with the minimum dimension between planes being 3 ft - 0 in. In the vicinity of the opening the splices are staggered to the maximum extent practical with no more than one half of the splices in one plane. The distance between two planes approximating the location of splices will be 20 to 24 inches. Splices on bars in one layer and in bars from layer to layer are staggered to the maximum extent permitted by existing conditions. The minimum spacing between splices on one bar will be 8 ft - o in., except for limited locations which consist primarily of locations where in-place splices were, or will be, removed for mechanical testing, and of locations on the inner band of horizontal bars where limited access precluded splice removal.

2.6TENDON CONDUIT

The tendon conduit, including that in the vicinity of the large openings, consists of six inch nominal diameter Schedule 40 pipe conforming to ASTM A53. The splices on this conduit consist of either standard threaded or welded couplings. The former were used only in the form of a half coupling connecting the tendon coupling enclosure to the conduit.

Table I
STRESS AROUND EQUIPMENT HATCH LOADING CONDITION NO. 4 - Accident Temperature

STRESS AROUND EQUIPMENT HATCH
LOADING CONDITION NO. 4
Accident Temperature

Element	Axial Direction		Hoop Direction		Membrane Shear	Normal	Shears
	N_ϕ	M_ϕ	N_θ	M_θ	$N_{\phi\theta}$	Q_ϕ	Q_θ
11	- 3.46	16.08	0.93	3.07	-0.14	-0.14	-0.02
22	- 4.18	18.08	1.36	4.78	-0.15	-0.04	0.02
33	- 5.00	17.38	4.31	43.32	-0.35	-0.27	-0.26
44	- 6.57	21.37	5.90	60.11	-0.71	-0.93	-0.40
55	- 8.19	26.80	8.15	82.63	-1.09	-1.12	-0.96
66	-10.11	43.34	10.63	108.75	-1.85	-1.28	-0.54
77	-12.54	48.87	14.49	149.08	-2.47	2.59	1.85
25	- 0.30	6.68	- 0.18	2.78	-1.16	-0.23	-0.10
49	0.04	16.41	- 0.30	5.03	-2.97	-0.10	-0.17
73	2.29	26.74	- 3.14	-15.27	-6.04	2.15	0.08
74	- 0.19	66.77	- 2.35	-47.19	-9.40	-4.19	-0.88
94	0.66	1.19	- 1.86	- 4.47	-0.07	-0.01	-0.32
97	3.36	19.21	- 3.48	7.58	-0.37	-0.09	0.42
99	5.75	- 30.87	- 5.48	10.01	-0.70	0.39	1.66
100	9.42	- 91.38	- 7.65	3.14	-1.85	1.47	4.05
101	10.95	-208.35	-10.68	-20.82	-4.48	-3.04	8.35

Note: N_ϕ, N_θ, $N_{\phi\theta}$, Q_ϕ and Q_θ in Kips/in. M_ϕ, and M_θ in Kips.

Values computed by finite-element analysis of cracked shell.

Figure 1 *Boration System*

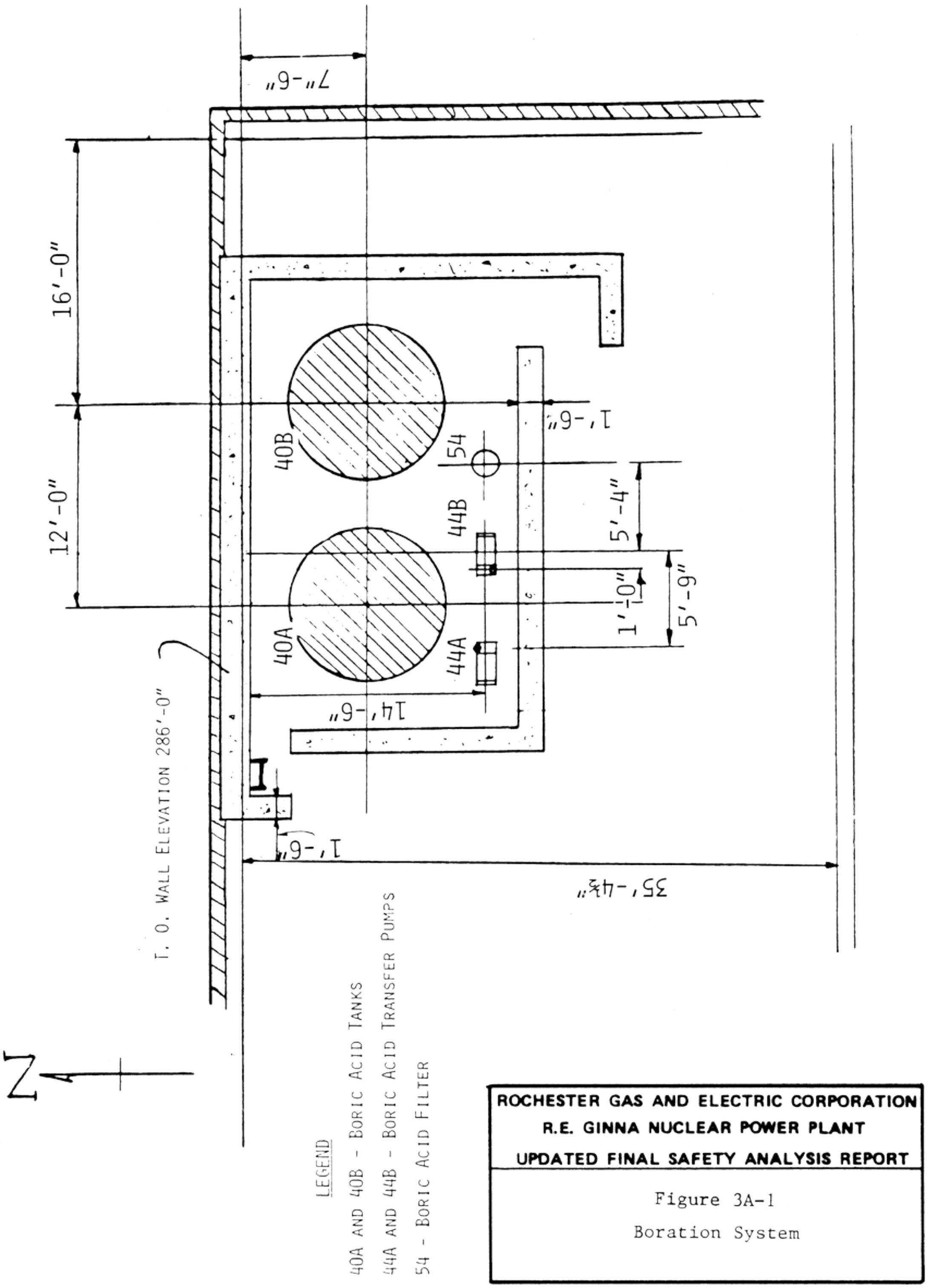
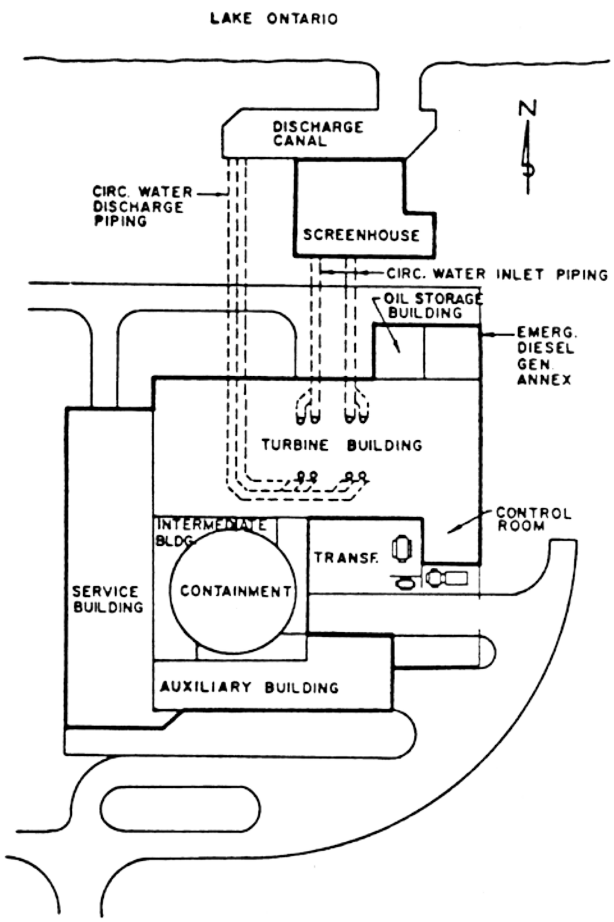
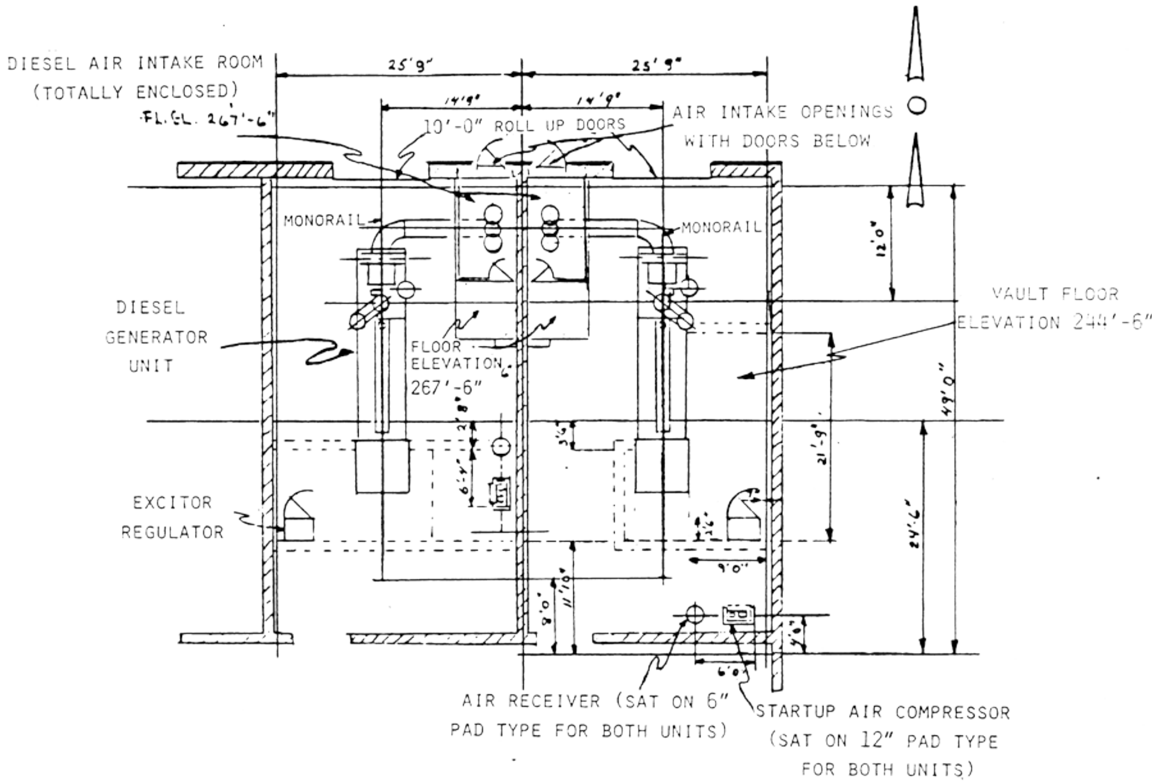


Figure 2 Site Plot Plan



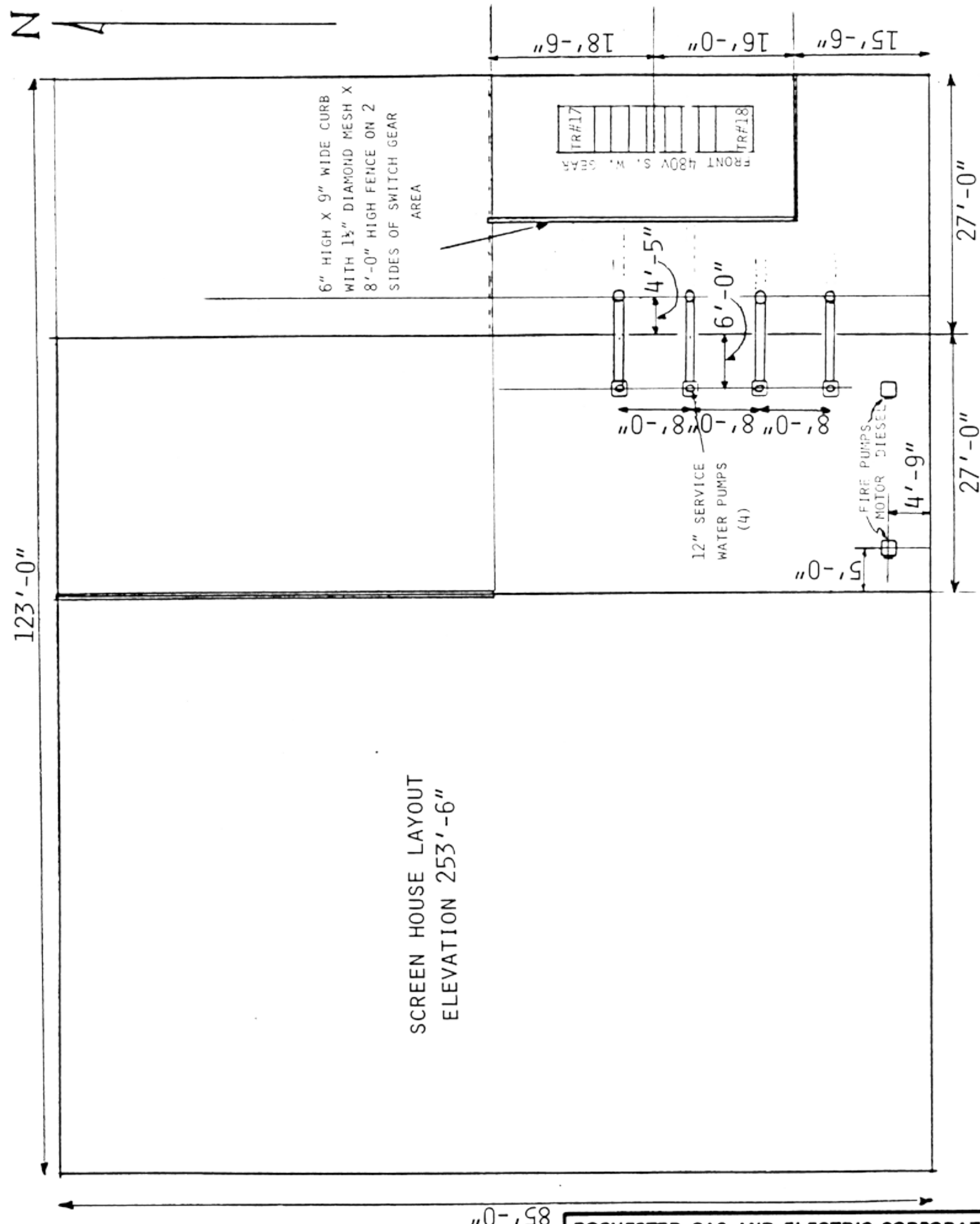
ROCHESTER GAS AND ELECTRIC CORPORATION **R.E. GINNA NUCLEAR POWER PLANT** **UPDATED FINAL SAFETY ANALYSIS REPORT**
Figure 3A-2 Site Plot Plan

Figure 3 Diesel Generator Annex - Elevation 253 ft 6 in.



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 3A-3 Diesel Generator Annex - Elevation 253 ft 6 in.

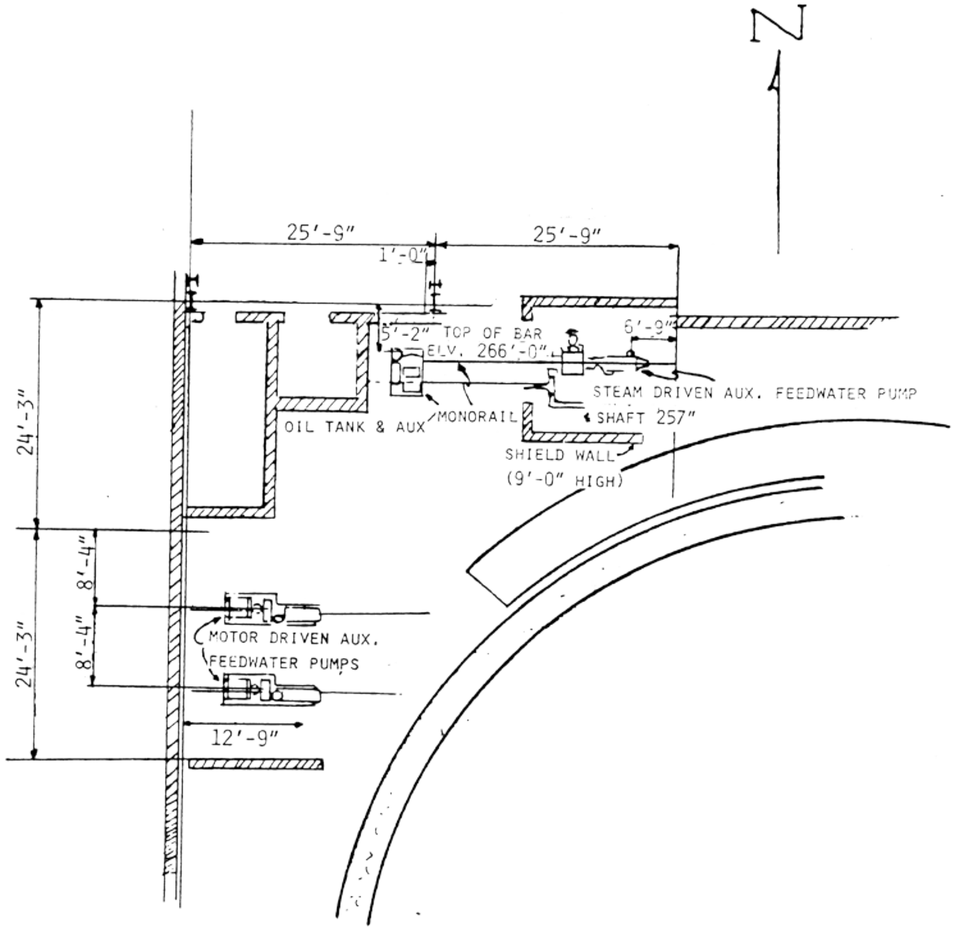
Figure 4 Screen House Layout



ROCHESTER GAS AND ELECTRIC CORPORATION
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

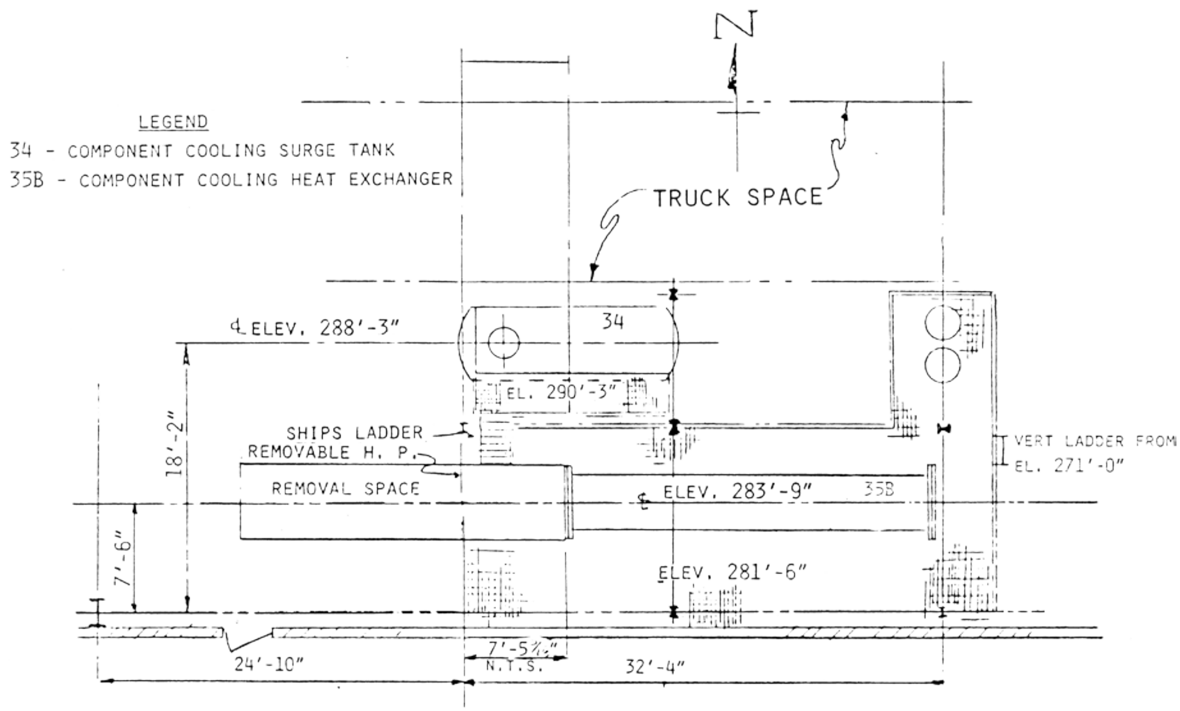
Figure 3A-4
Screen House Layout

Figure 6 Auxiliary FeedwaterB00019 Pumps



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 3A-6 Auxiliary Feedwater Pumps

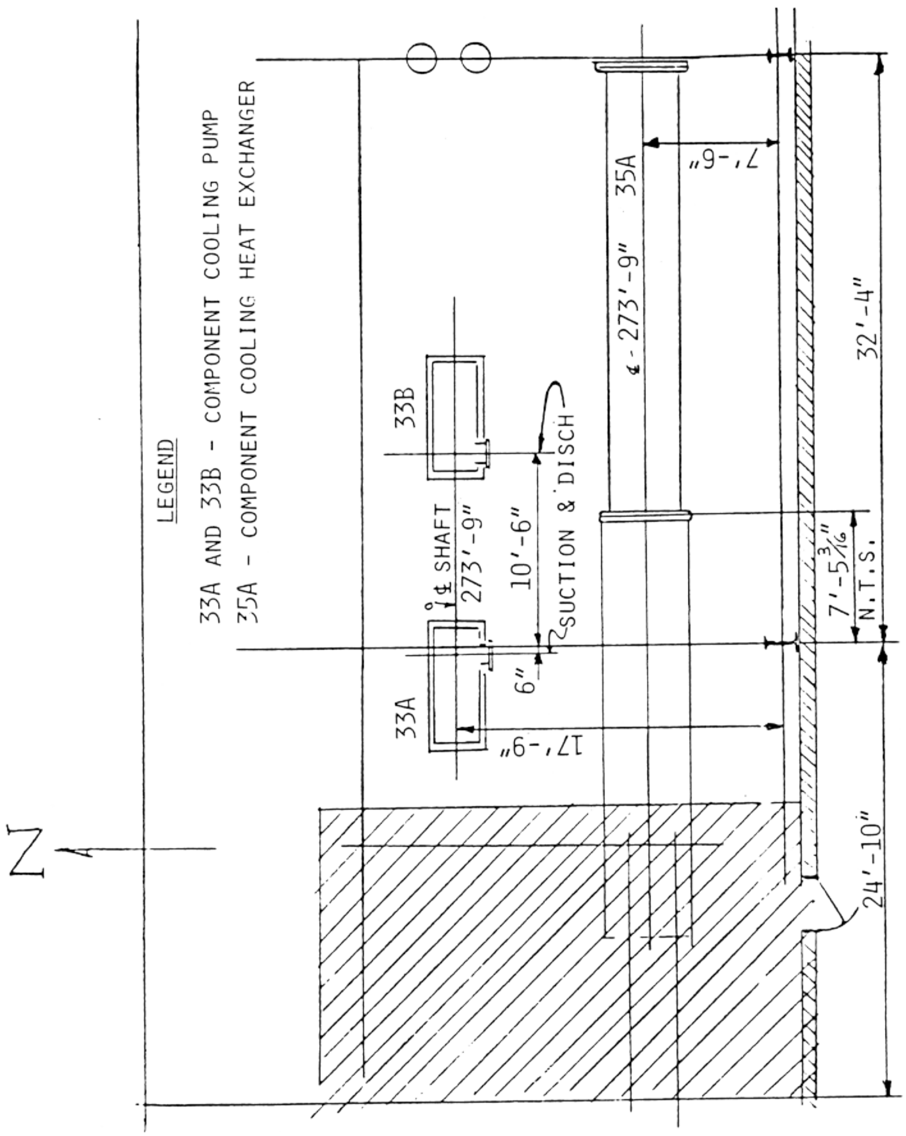
Figure 7 Component Cooling System



ROCHESTER GAS AND ELECTRIC CORPORATION
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3A-7, Sheet 1
Component Cooling System

Sheet 2 of Figure 7



ROCHESTER GAS AND ELECTRIC CORPORATION
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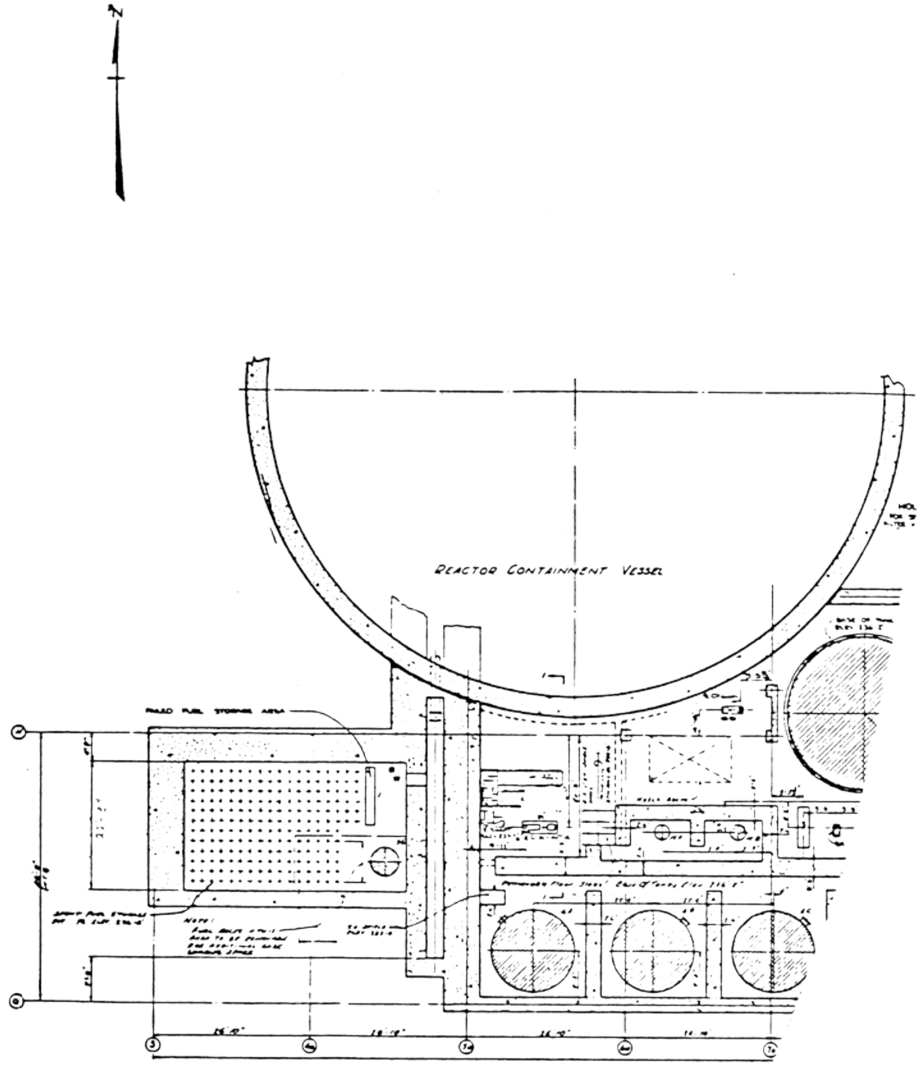
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3A-7, Sheet 2
Component Cooling System

Figure 8 *Spent Fuel Storage Pool, Plan View*



ROCHESTER GAS AND ELECTRIC CORPORATION
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UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3A-8
Spent Fuel Storage Pool, Plan View

Figure 9 Spent Fuel Storage Pool, Section View

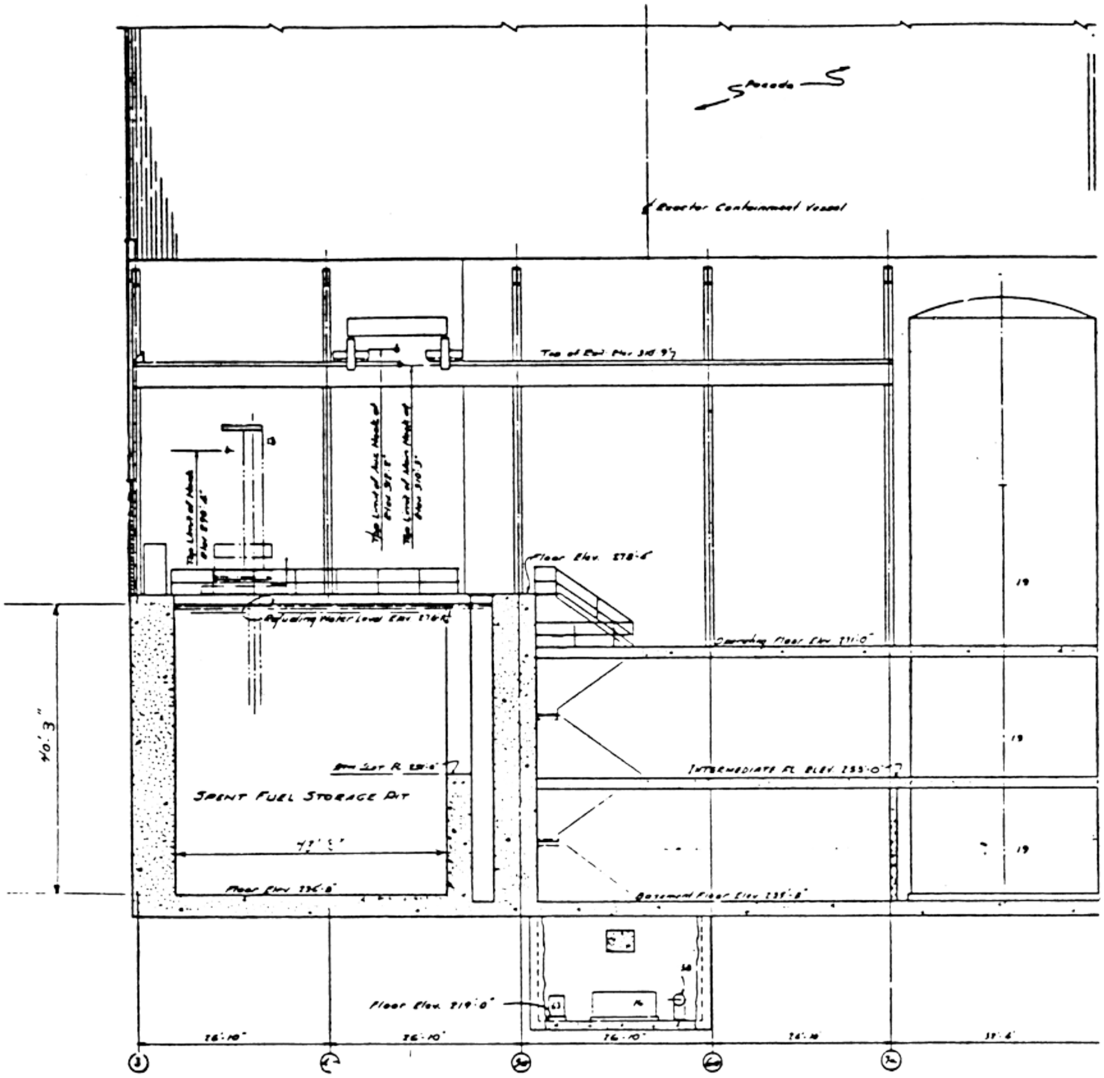
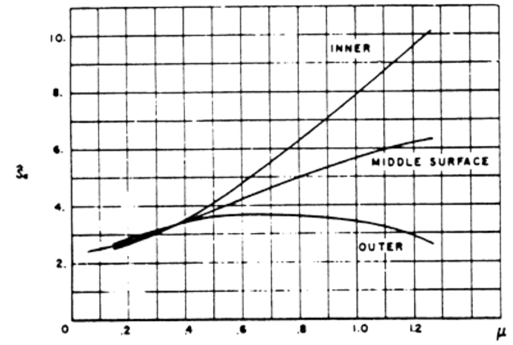
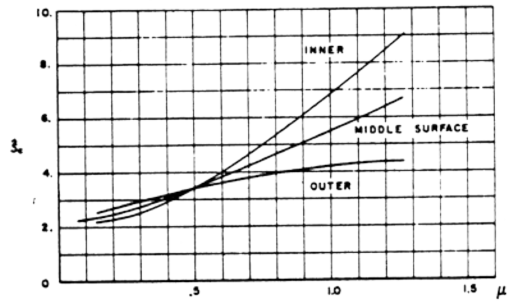


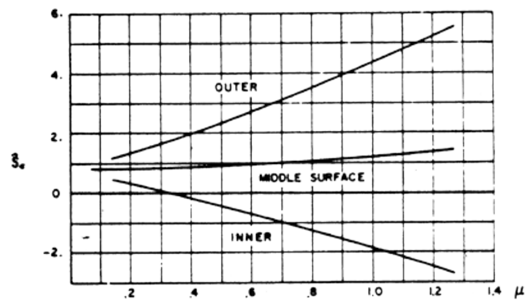
Figure 1



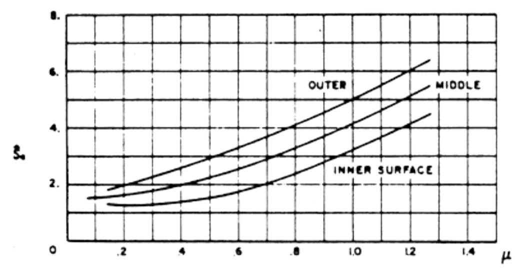
A—$\hat{S}_c$ for $\phi = 0$ vs. $\beta\rho_0$ (capped cylinder), $\nu = 0.3$



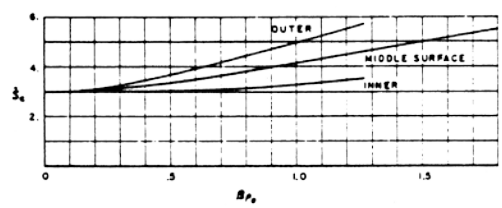
B—$\hat{S}_c$ for $\phi = \pi/8$ vs. $\beta\rho_0$ (capped cylinder), $\nu = 0.3$



D—$\hat{S}_c$ for $\phi = 3\pi/8$ vs. $\beta\rho_0$ (capped cylinder), $\nu = 0.3$

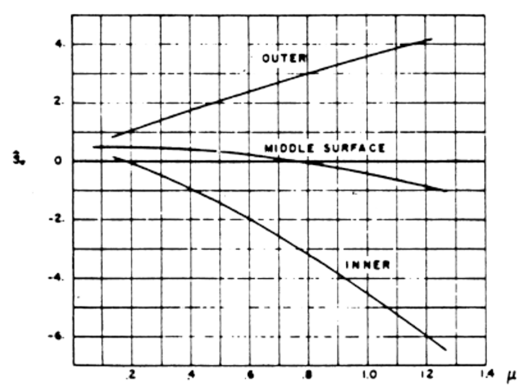


C—$\hat{S}_c$ for $\phi = \pi/4$ vs. $\beta\rho_0$ (capped cylinder), $\nu = 0.3$

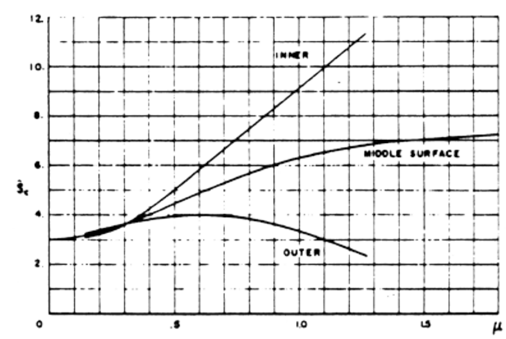


F—$\hat{S}_c$ at $\phi = \pi/2$ for Case II (extension case) vs. $\beta\rho_0$, $\nu=0.3$

NOTE
REFERENCE: "STATE OF STRESS IN A CIRCULAR CYLINDRICAL SHELL WITH A CIRCULAR HOLE," WELDING RESEARCH COUNCIL BULLETIN 102, 1965.



E—$\hat{S}_c$ for $\phi = \pi/2$ vs. $\beta\rho_0$ (capped cylinder), $\nu = 0.3$



G—$\hat{S}_c$ at $\phi = 0$ for Case III (internal pressure), $\nu = 0.3$

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FIGURE 1

Figure 2

Table 3—Stress Concentration Factor Variations for Different $\beta_{\rho e}$ at Various Angles—Capped Cylinder

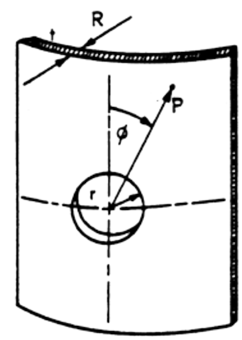
$\nu = 0.30$

$\beta_{\rho e}$	$(\hat{S}_c)$upper	S_c	$(\hat{S}_c)$lower
		$\phi = 0$	
0.14142	2.75054	2.64709	2.54365
0.21213	2.94174	2.81900	2.69625
0.28284	3.13290	3.03670	2.94050
0.35355	3.30842	3.28570	3.26297
0.42426	3.45933	3.55467	3.65001
0.49497	3.58032	3.83486	4.08940
0.56568	3.66795	4.11911	4.57027
0.63639	3.72044	4.40201	5.08357
0.70710	3.73656	4.67874	5.62091
0.84852	3.66013	5.20121	6.74228
0.98994	3.44158	5.66463	7.88769
1.13137	3.09080	6.05690	9.02300
1.27279	2.61996	6.37120	10.12243
		$\phi = \pi/8$	
0.14142	2.46986	2.33179	2.19372
0.21213	2.66925	2.47978	2.29030
0.28284	2.87265	2.67011	2.46758
0.35355	3.06813	2.89212	2.71611
0.42426	3.25013	3.13783	3.02553
0.49497	3.41623	3.40126	3.38629
0.56568	3.56589	3.67806	3.79024
0.63639	3.69959	3.96509	4.23059
0.70710	3.81810	4.25958	4.70107
0.84852	4.01561	4.86482	5.71403
0.98994	4.16952	5.48212	6.79472
1.13137	4.29120	6.10374	7.91627
1.27279	4.39120	6.72431	9.05741
		$\phi = \pi/4$	
0.14142	1.79394	1.56884	1.34374
0.21213	2.01190	1.65385	1.29580
0.28284	2.23906	1.76710	1.29514
0.35355	2.46809	1.90438	1.33966
0.42426	2.70142	2.06304	1.42466
0.49497	2.93776	2.24160	1.54544
0.56568	3.18071	2.43944	1.69816
0.63639	3.43312	2.65648	1.87983
0.70710	3.69754	2.89277	2.08800
0.84852	4.27107	3.42439	2.57771
0.98994	4.91382	4.03512	3.15642
1.13137	5.63074	4.72359	3.81643
1.27279	6.42060	5.48575	4.55090
		$\phi = 3\pi/8$	
0.14142	1.12040	0.80340	0.48640
0.21213	1.35527	0.81807	0.28087
0.28284	1.59750	0.83819	0.07888
0.35355	1.84227	0.86252	−0.11723
0.42426	2.08946	0.88996	−0.30954
0.49497	2.34032	0.91983	−0.50065
0.56568	2.59629	0.95182	−0.69263
0.63639	2.85862	0.98593	−0.88675
0.70710	3.12823	1.02243	−1.08337
0.84852	3.69078	1.10414	−1.48249
0.98994	4.28578	1.20142	−1.88293
1.13137	4.91210	1.31959	−2.27291
1.27279	5.56731	1.46418	−2.63894
		$\phi = \pi/2$	
0.21213	1.08342	0.46892	−0.14557
0.28284	1.32929	0.44570	−0.43787
0.35355	1.57420	0.41483	−0.74454
0.42426	1.81691	0.37473	−1.06744
0.49497	2.05720	0.32403	−1.40912
0.56568	2.29480	0.26158	−1.77164
0.63639	2.52926	0.18654	−2.15617
0.70710	2.75997	0.09861	−2.56273
0.84852	3.20626	−0.11635	−3.43897
0.98994	3.62829	−0.38079	−4.38988
1.13137	4.02201	−0.68968	−5.40137
1.27279	4.38777	−1.03524	−6.45827

Table 3-Continued

$\hat{S}_c$ (*Middle Surface only*)

$\beta_{\rho e}$	$\phi = 0$	$\pi/10$	$\pi/5$	$3\pi/10$	$2\pi/5$	$\pi/2$
1.4142	6.6163	7.1556	7.2307	4.7269	0.6424	−1.4030
1.5909	6.7936	7.7349	8.3542	5.6384	0.6623	−1.8994
1.7677	6.9059	8.4207	9.5285	6.6467	0.7164	−2.4293



$$\mu = \frac{1}{2}\sqrt[4]{3(1-\nu^2)}\quad\frac{r}{\sqrt{Rt}}$$

The membrane stress concentration factor S_c and the total stress concentration factor $\hat{S}_c$ are, respectively, defined by

$$S_c = \frac{\text{largest of } (N_1, N_2)}{\text{largest of } (N_1{}^0, N_2{}^0)} = \hat{S}_c \quad \text{(middle surface)}$$

$$\hat{S}_c = \frac{\text{largest of } (\sigma 1, \sigma 2)}{\text{largest of } (\sigma 1^0, \sigma 2^0)} \quad \text{(for fixed } r, \phi)$$

where $N_1{}^0$, $N_2{}^0$ are the nominal principal stress resultants and $\sigma 1^0$, $\sigma 2^0$ are the nominal flexural stresses for the shell under the same loading but without the hole. N_1 and N_2 denote the principal stress resultants, σ_1 and σ_2 the principal stresses respectively. The stress concentration factor is calculated as a function of ϕ.

NOTE
REFERENCE: "STATE OF STRESS IN A CIRCULAR CYLINDRICAL SHELL WITH A CIRCULAR HOLE." WELDING RESEARCH COUNCIL BULLETIN 102, 1965.

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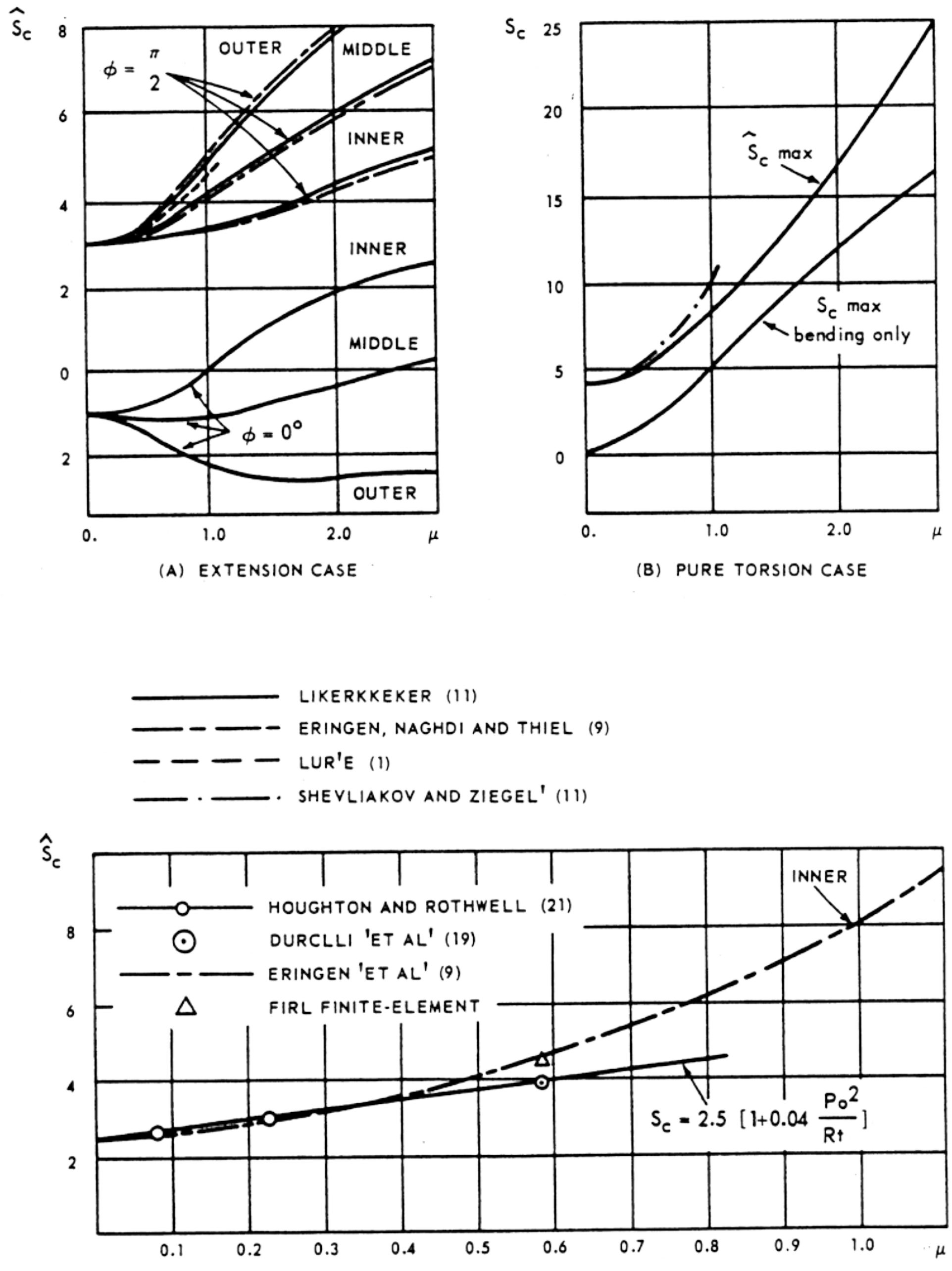
FIGURE 2

Figure 3

STRESS DISTRIBUTION AROUND OPENINGS IN CYLINDRICAL SHELLS



(C) CAPPED CYLINDER UNDER INTERNAL PRESSURE

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FIGURE 3

Figure 4

GRID FOR FINITE ELEMENT ANALYSIS
OF THE STRESSES AROUND OPENINGS

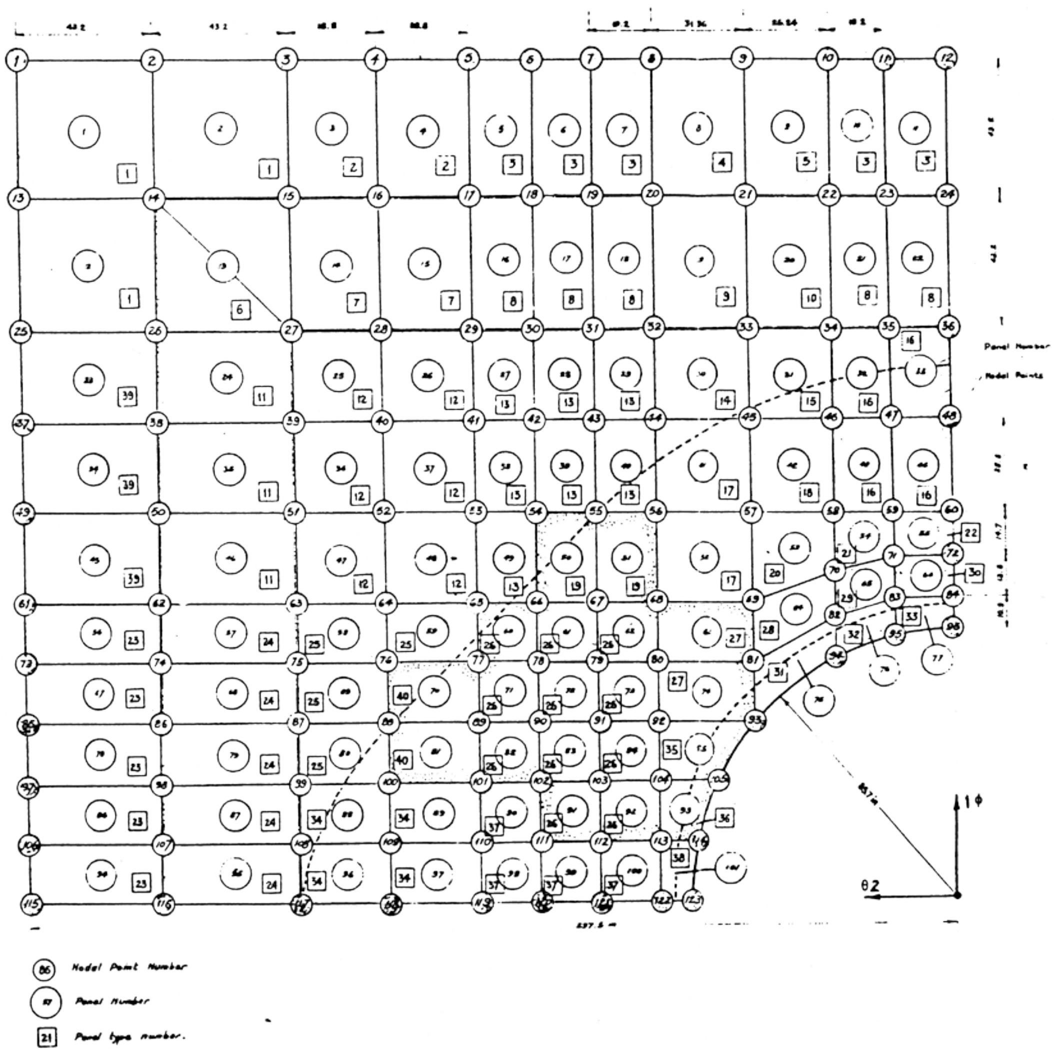

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FIGURE 4

Figure 5

MEMBRANE STRESS AROUND OPENING EDGE
(VESSEL SUBJECT TO INTERNAL PRESSURE)

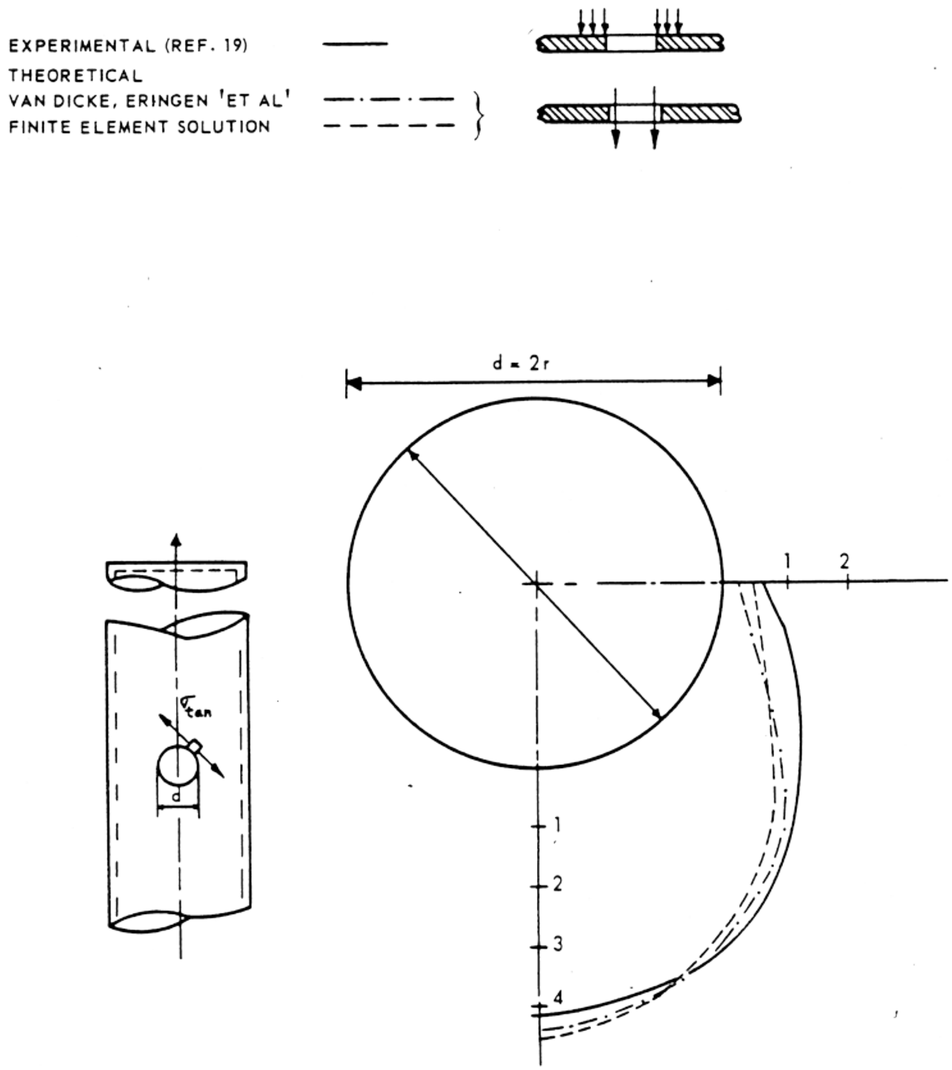

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FIGURE 5

Figure 6

SURFACE STRESSES AROUND-OPENING EDGE
(VESSEL SUBJECT TO INTERNAL PRESSURE)

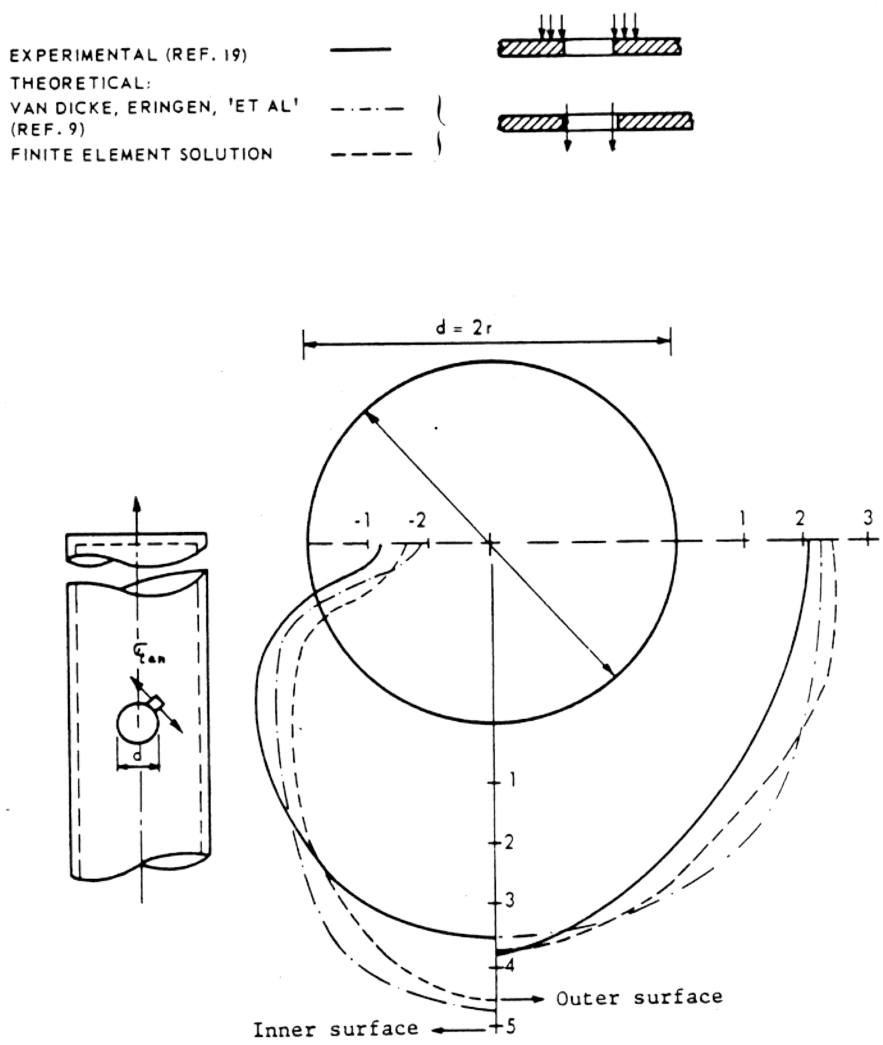

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FIGURE 6

Figure 7

HOOP STRESSES ALONG LONGITUDINAL AXIS
(VESSEL SUBJECT TO INTERNAL PRESSURE)

EXPERIMENTAL (REF. 19) PHOTOELASTICITY ————————
HUGGENBERGER TENSOMETER o
FINITE ELEMENT SOLUTION — — — — — — — —

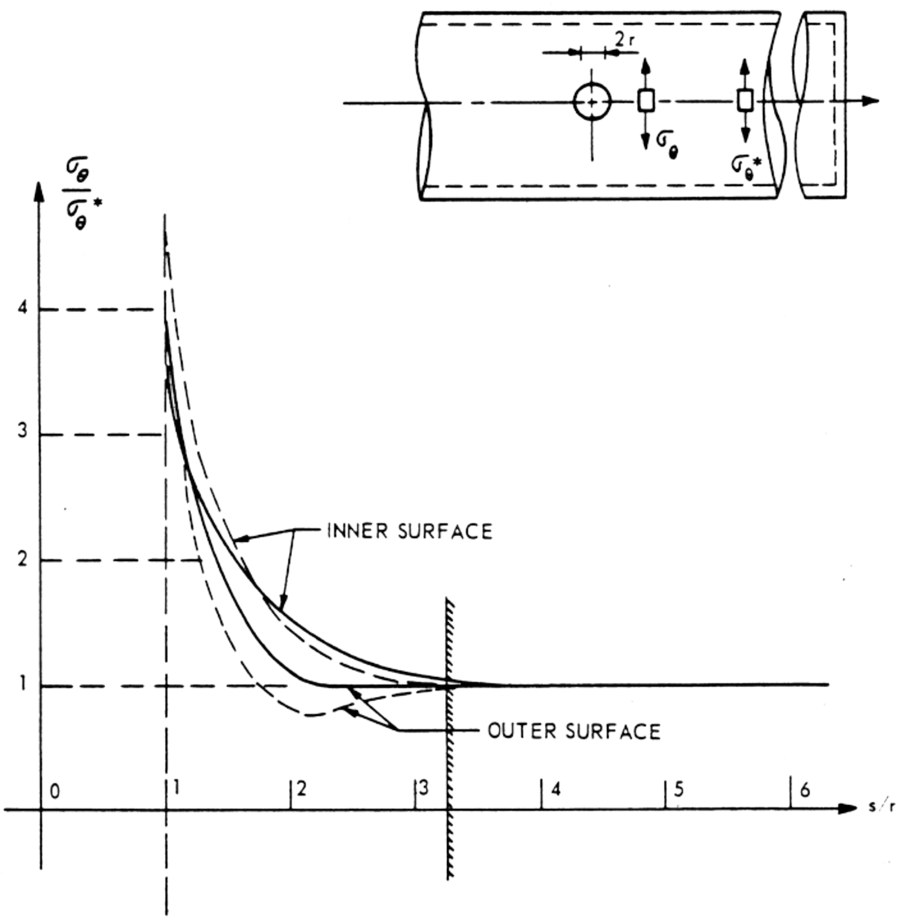

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FIGURE 7

Figure 8

AXIAL STRESSES ALONG TRANSVERSE AXIS
(VESSEL SUBJECT TO INTERNAL PRESSURE)

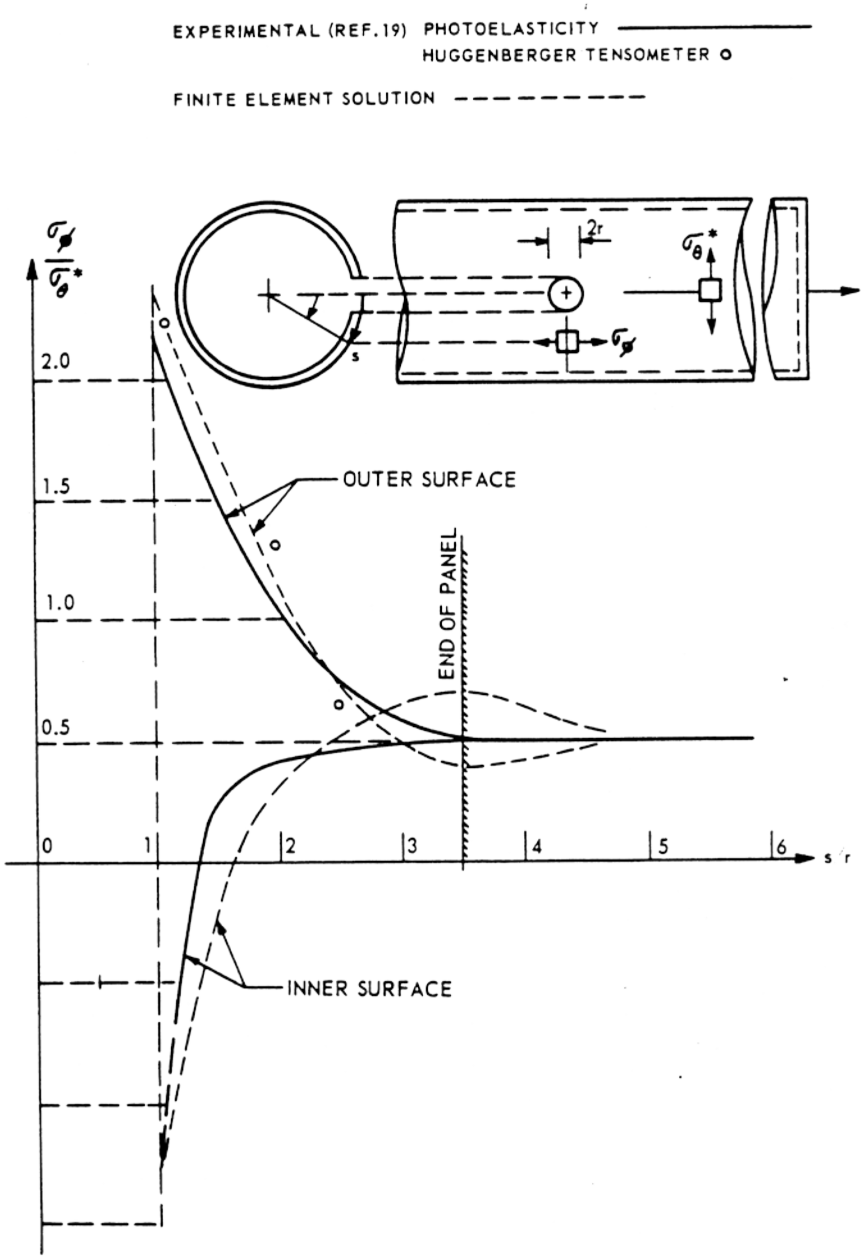

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FIGURE 8

Figure 9

HOOP STRESS-RESULTANT N_θ ALONG SYMMETRY AXES
(TEST PROBLEM)

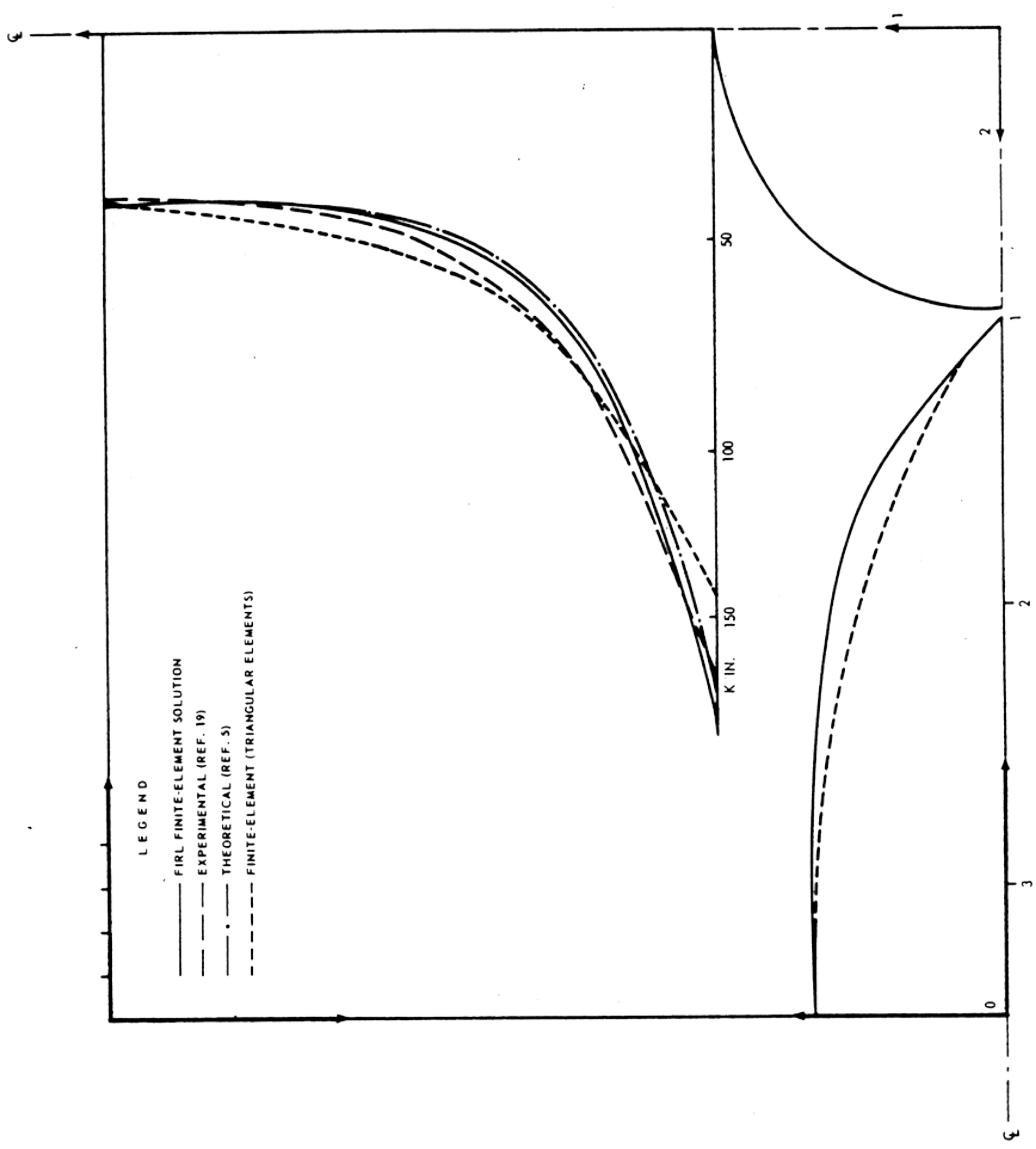

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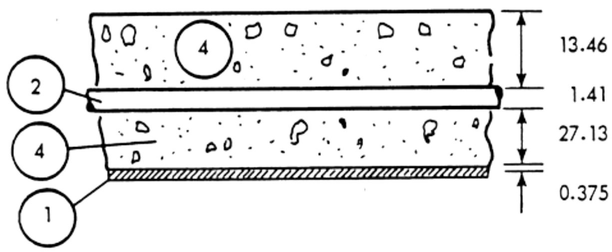
FIGURE 9

Figure 10

LAYER THICKNESSES AND DESIGNATION

TYPICAL SHELL



REINFORCED SHELL

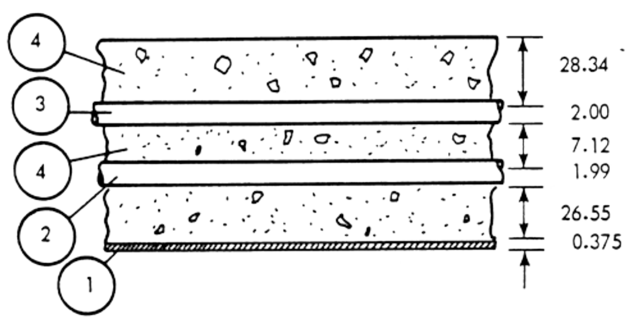


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FIGURE 10

Figure 11

NODAL FORCES DUE TO CURVATURE OF TENDONS IN NEIGHBORHOOD OF OPENING

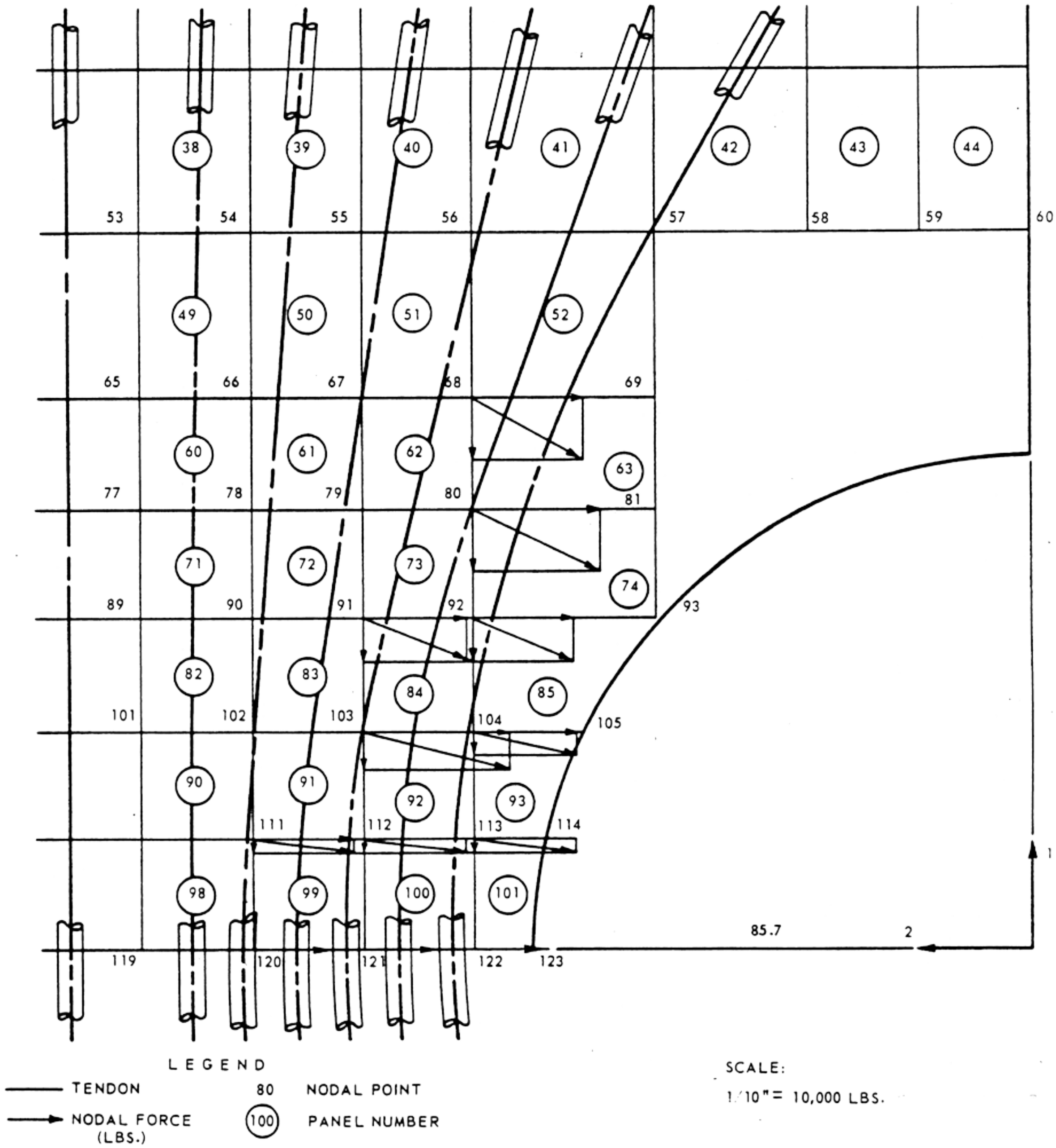

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FIGURE 11

Figure 12

STRESS DISTRIBUTION AROUND OPENINGS
(THERMAL GRADIENT NEAR EQUIPMENT OPENING)

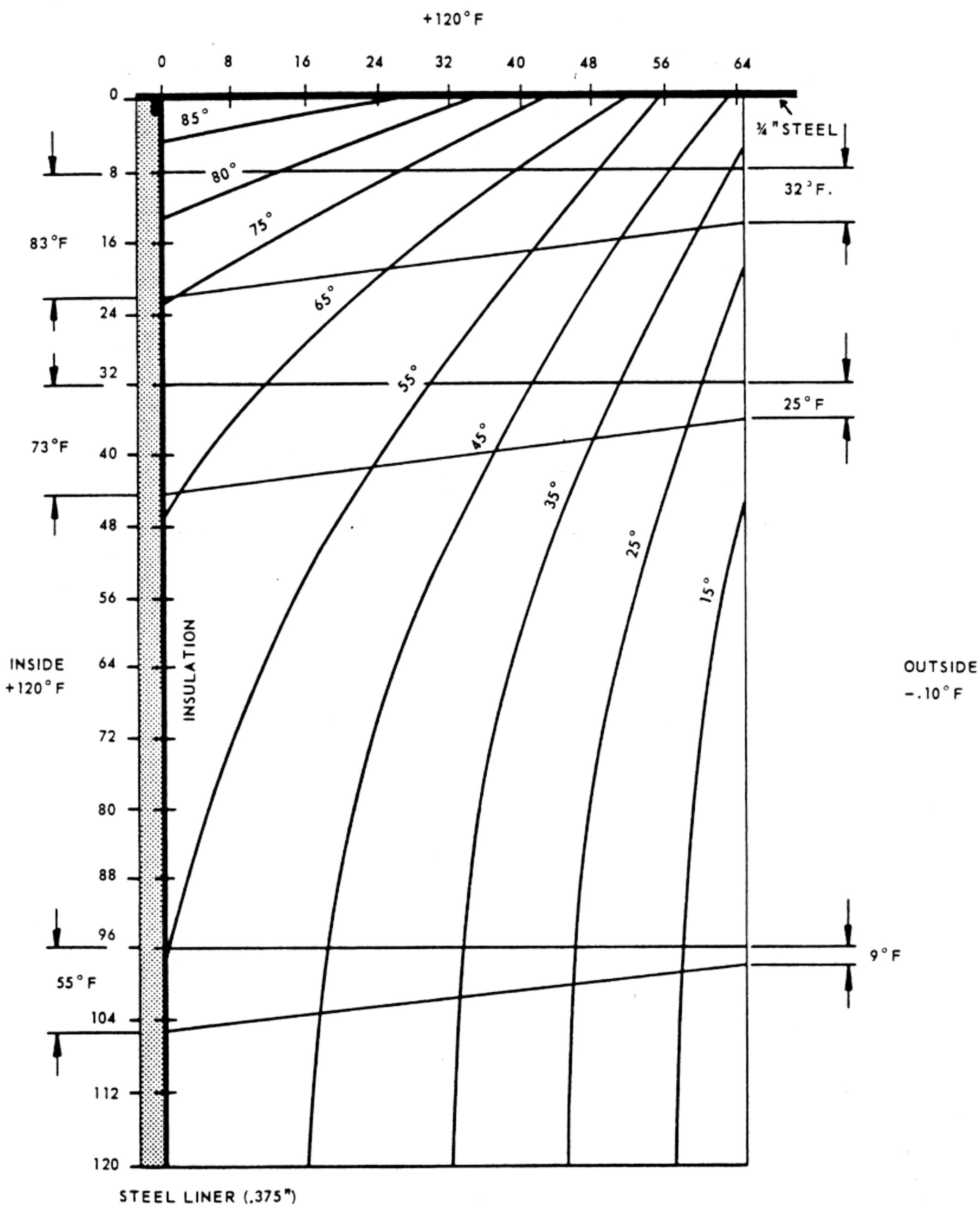

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FIGURE 12

Figure 13 Steady State Temperature Distributions - Winter Gradient

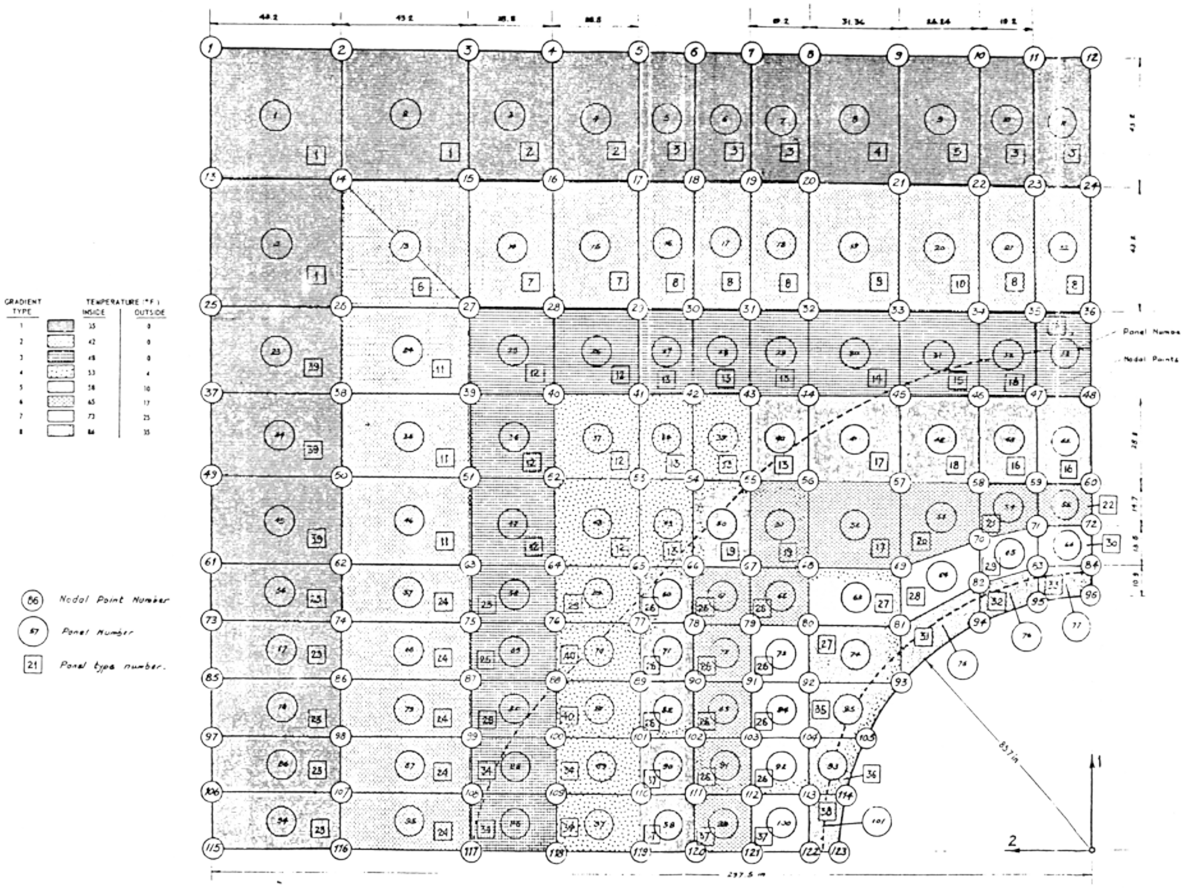
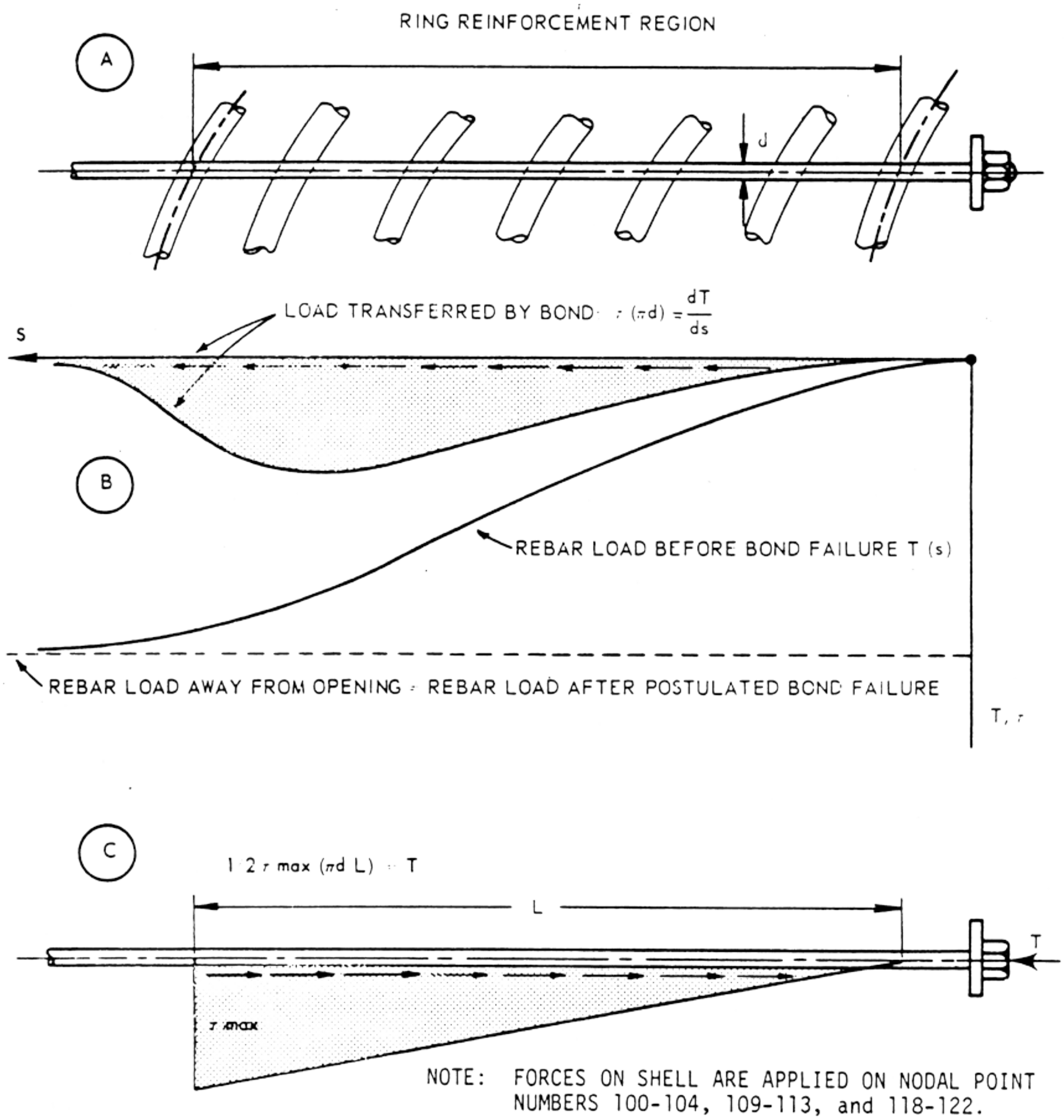


Figure 14

STRESS DISTRIBUTION AROUND OPENINGS
(EFFECT OF BOND FAILURE ALONG TERMINATED REBARS)



LOADS APPLIED IN FINITE ELEMENT ANALYSIS

GILBERT ASSOCIATES, INC.

FIGURE 14

Figure 15

HOOP STRESS RESULTANTS ALONG HORIZONTAL AND VERTICAL SYMMETRY AXES
(INTERNAL PRESSURE = 69 PSI

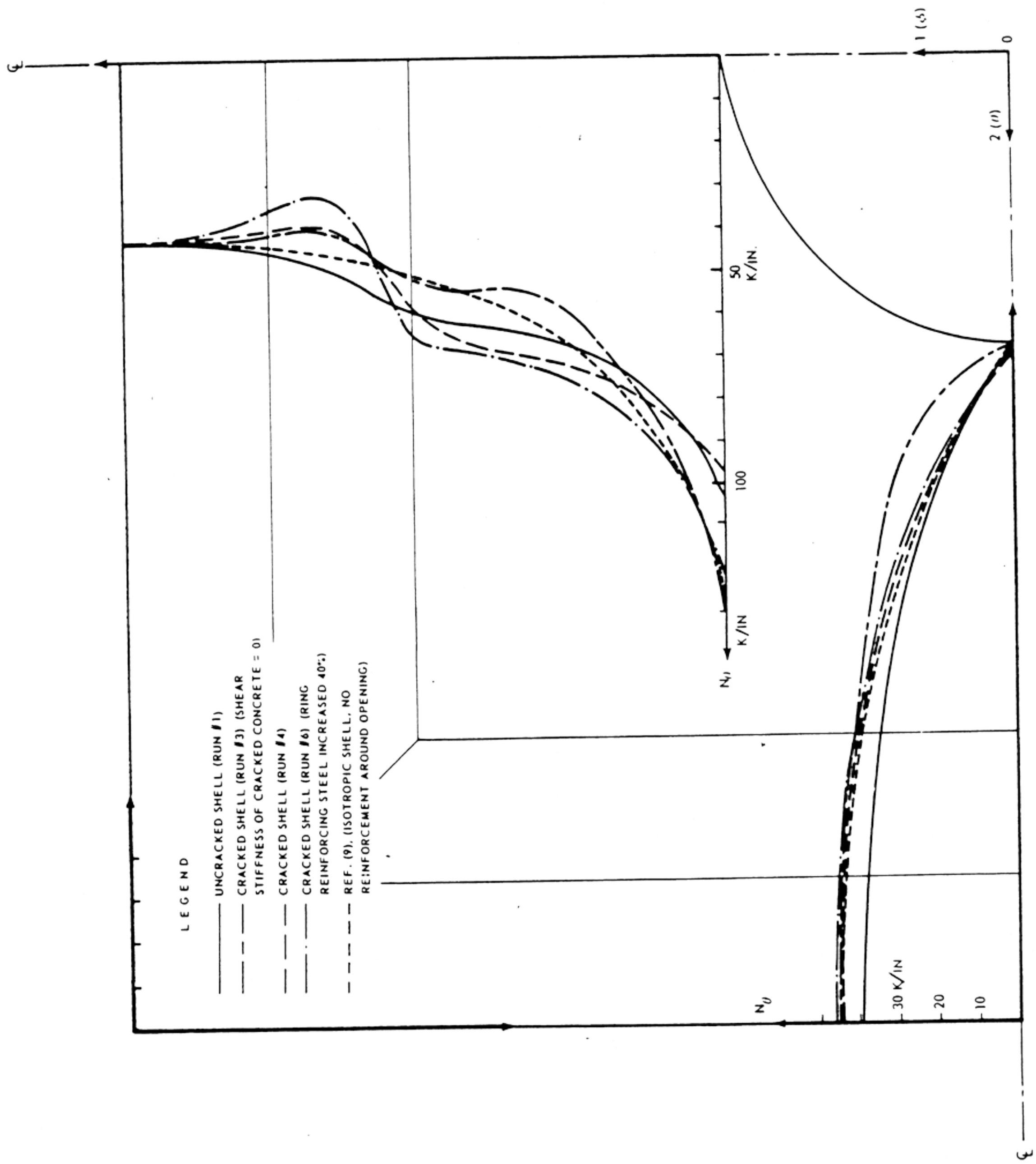


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FIGURE 15

Figure 16

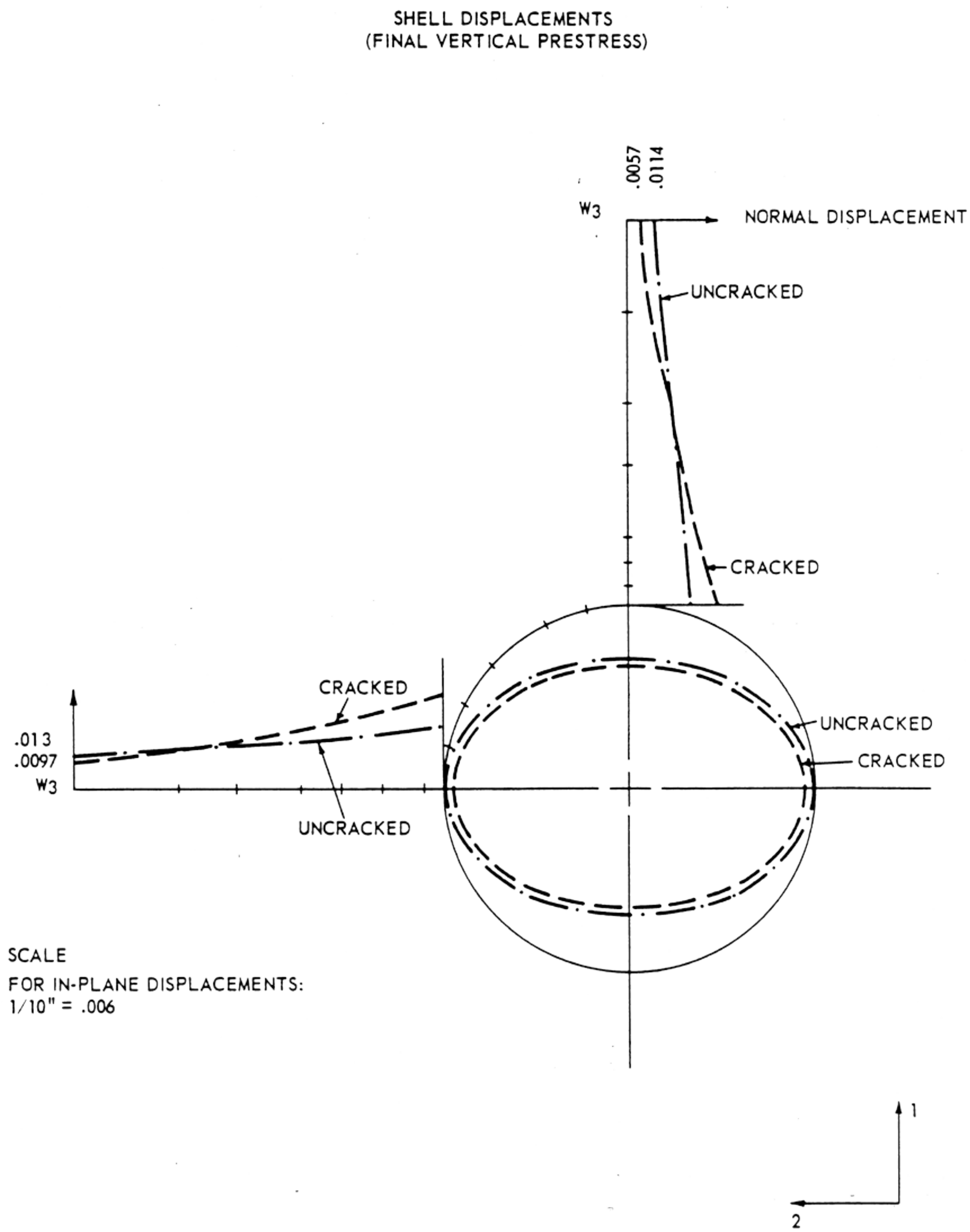

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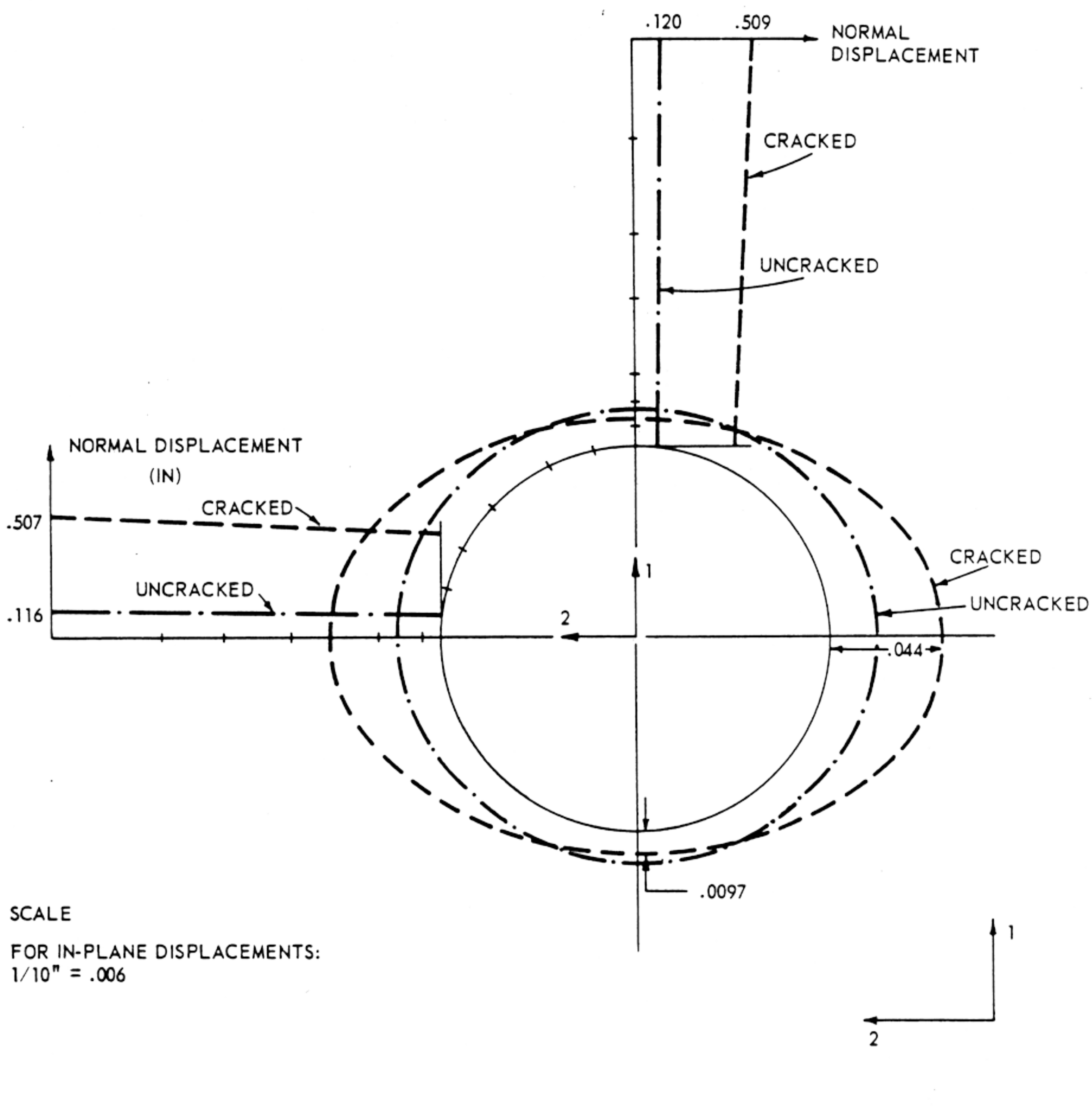
FIGURE 16

Figure 17

SHELL DISPLACEMENTS (69 PSI INTERNAL PRESSURE)



SCALE

FOR IN-PLANE DISPLACEMENTS:
1/10" = .006

GILBERT ASSOCIATES, INC. FIGURE 17

Figure 18

INTERACTION DIAGRAM FOR AXIAL COMPRESSION/TENSION AND BENDING

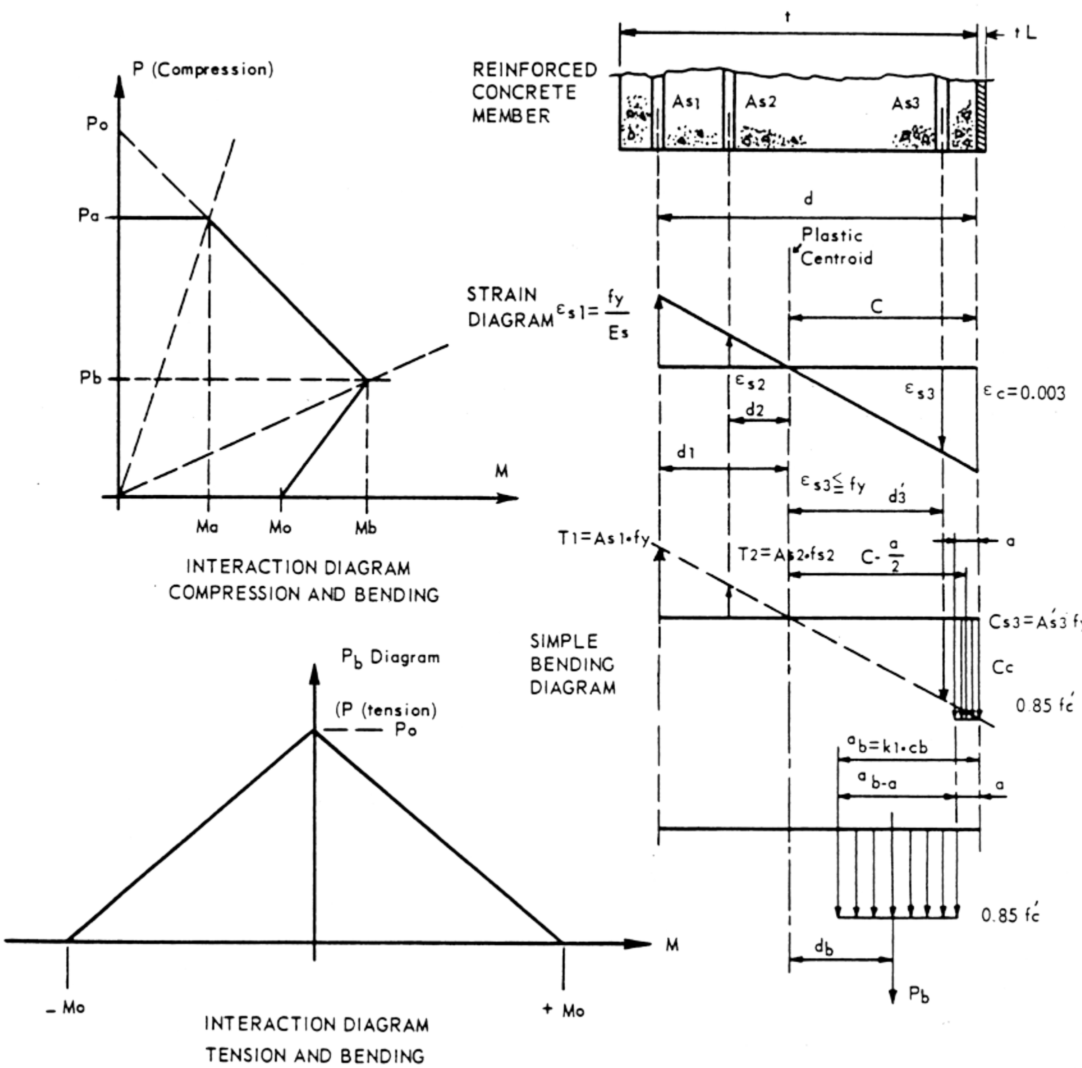


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FIGURE 18

Figure 19

INTERACTION DIAGRAM
RING STEEL DIRECTION
ELEMENTS NO. 73 & 74

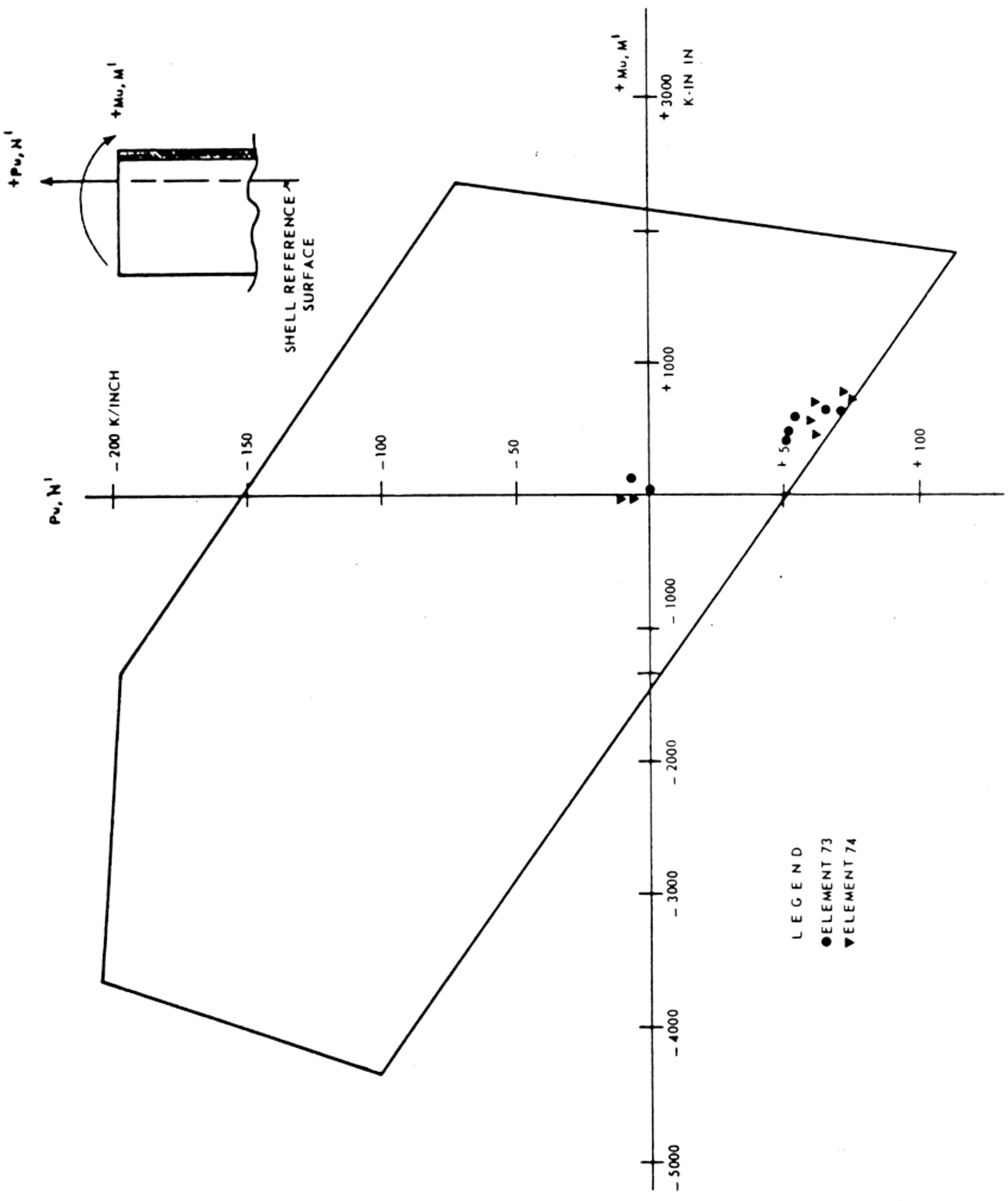

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FIGURE 19

Figure 20

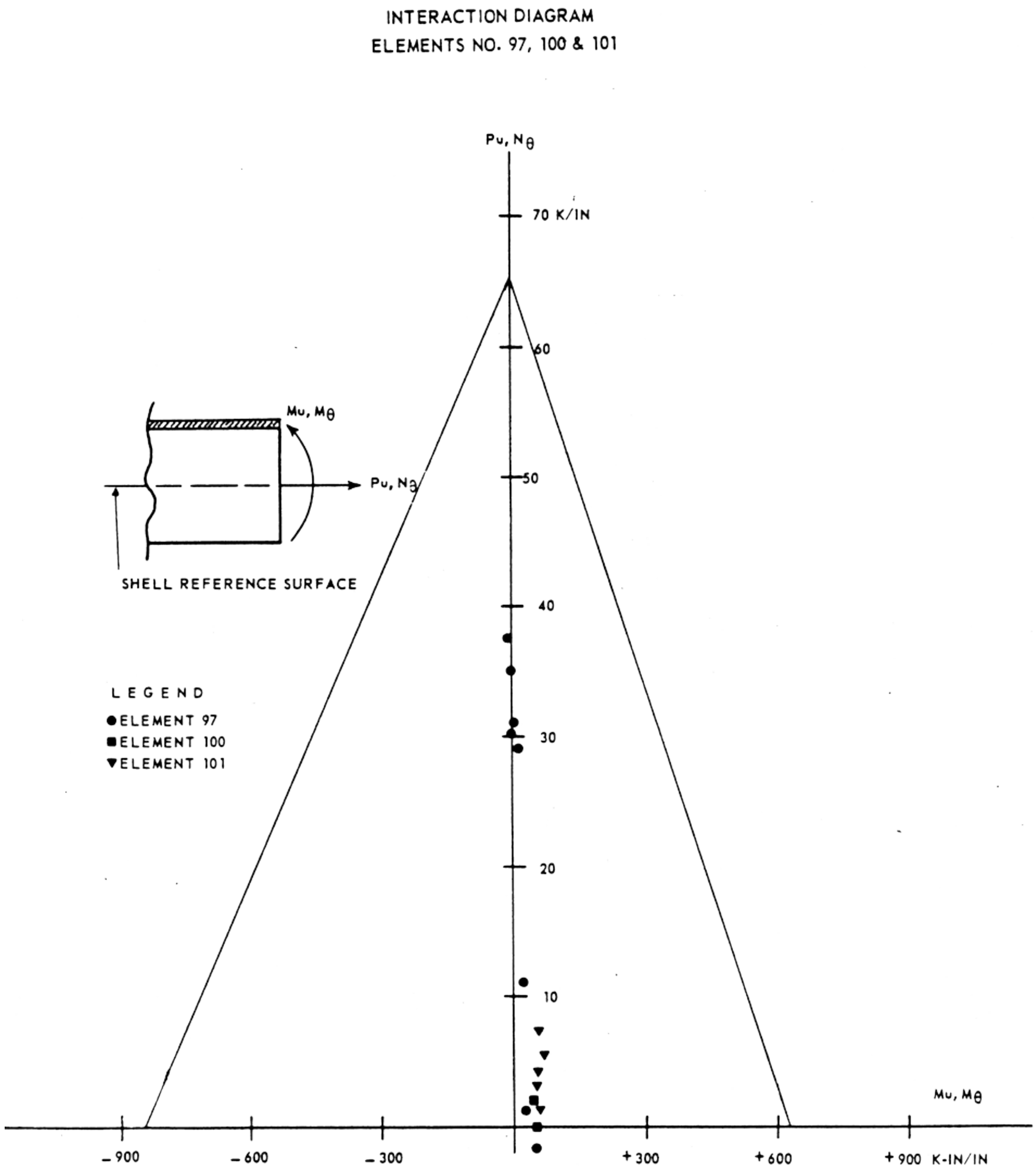

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FIGURE 20

Figure 21

INTERACTION DIAGRAM
ELEMENTS NO. 97, 100 & 101

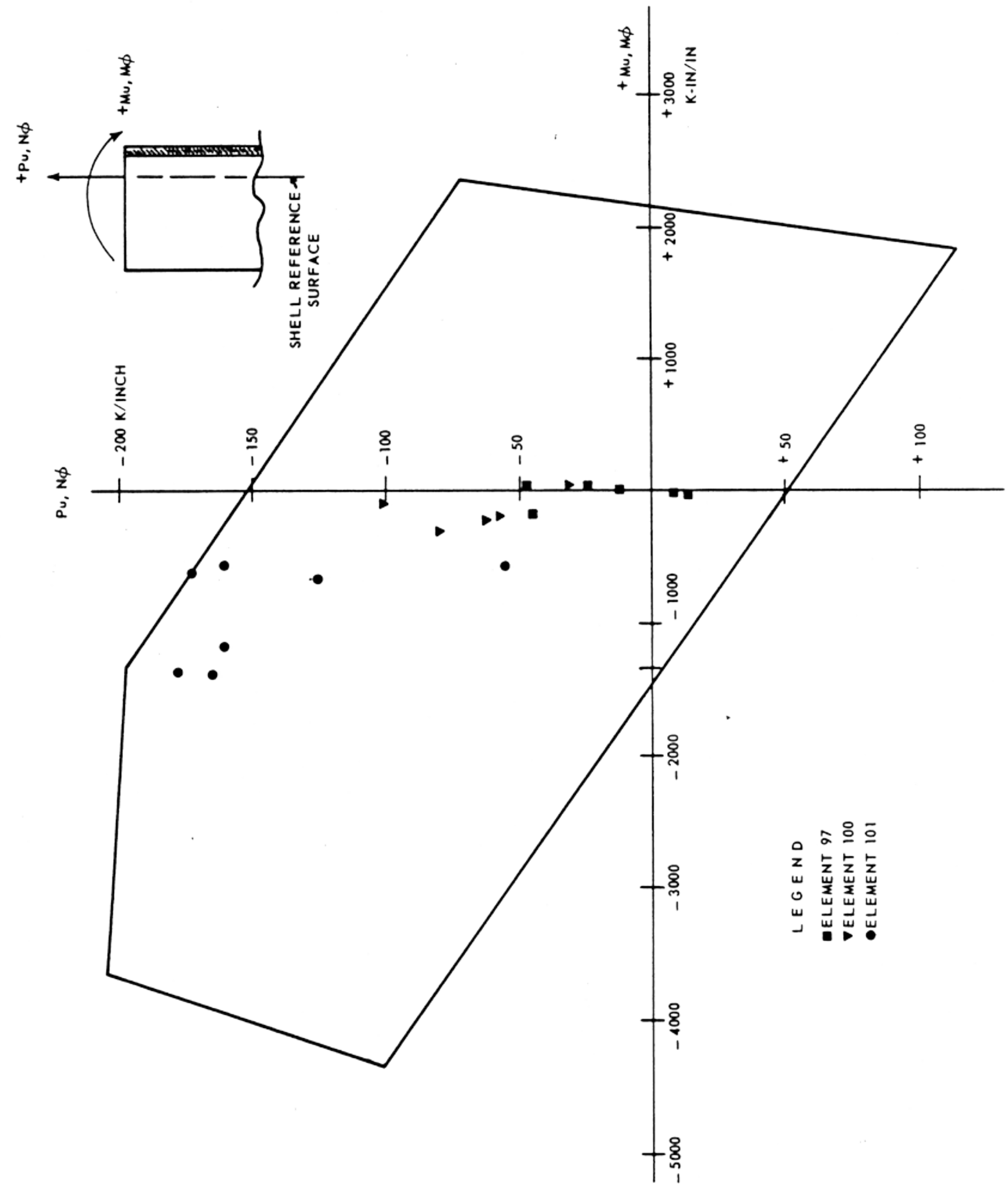

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FIGURE 21

Figure 22

INTERACTION DIAGRAM
ELEMENTS NO. 33, 55, 66 & 77

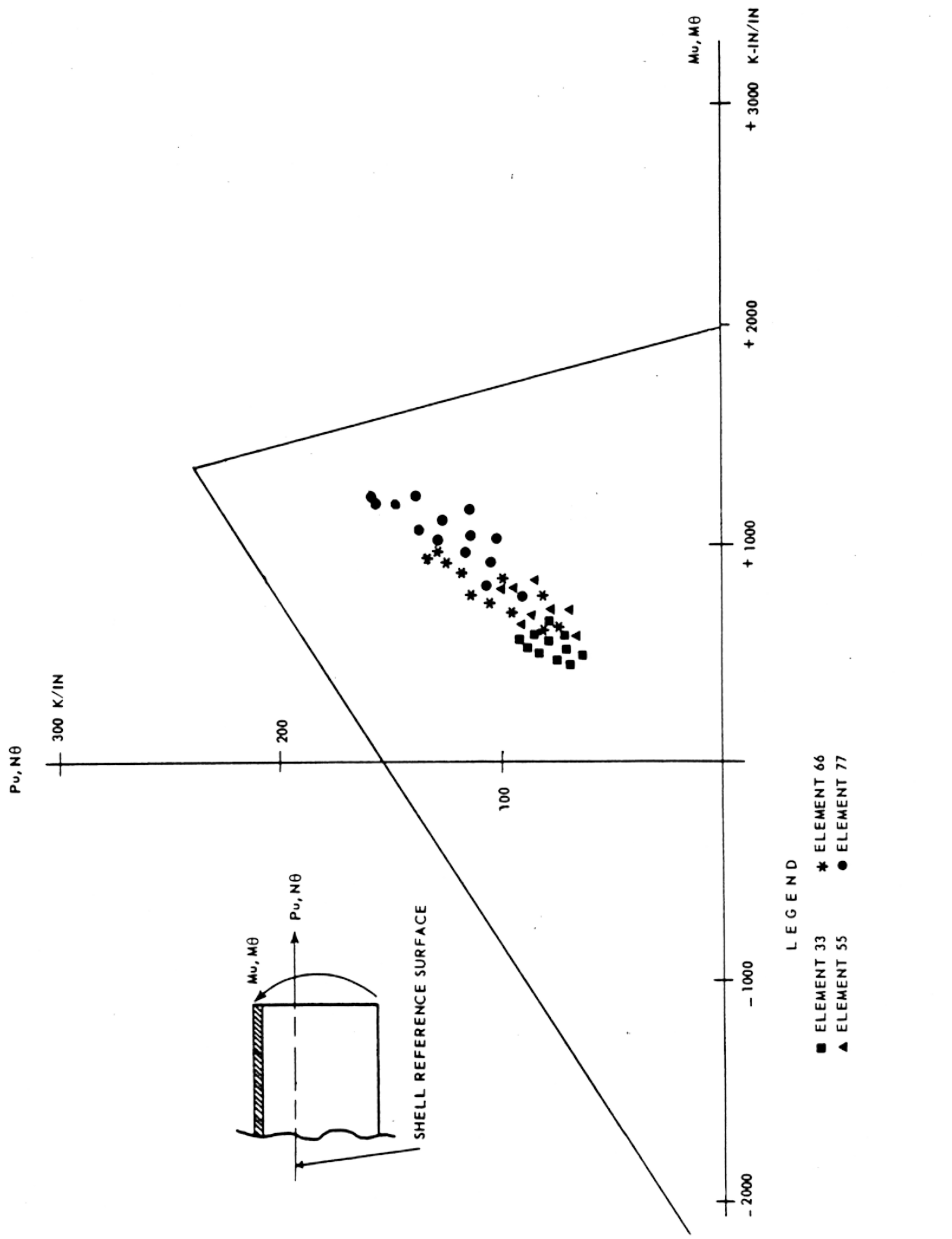

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FIGURE 22

Figure 23

INTERACTION DIAGRAM
ELEMENT NO. 77

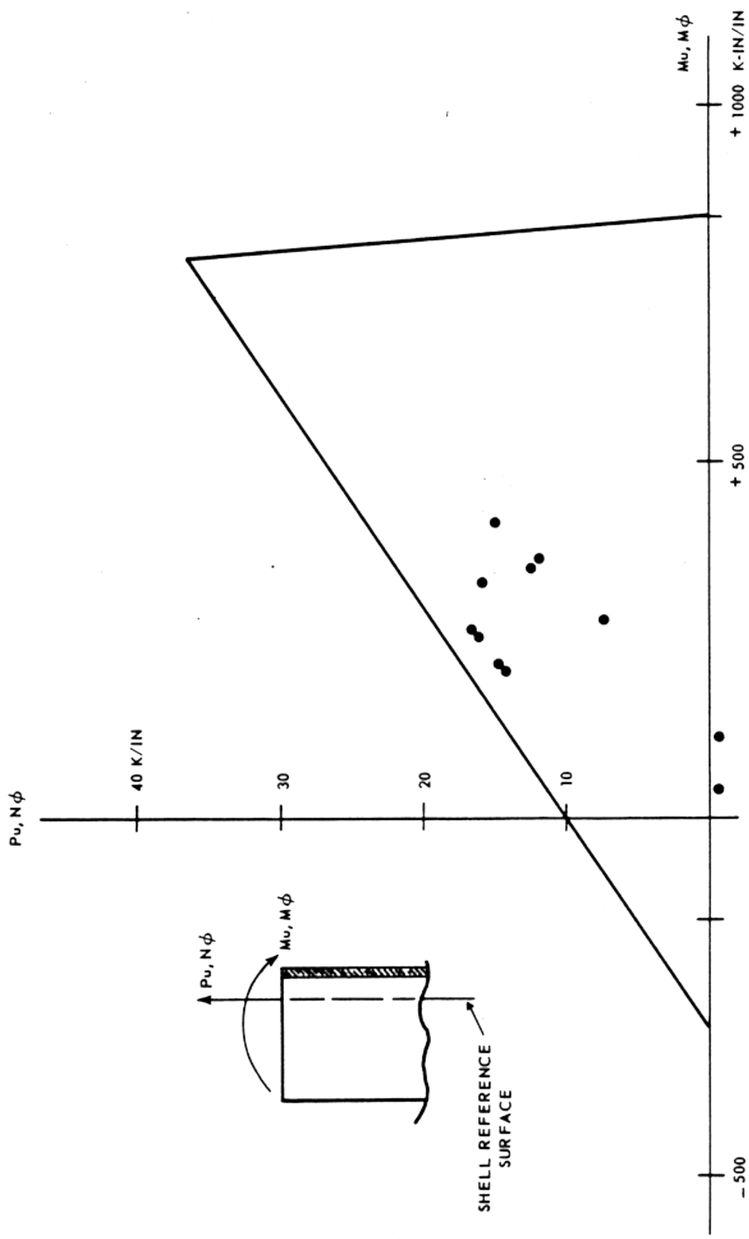

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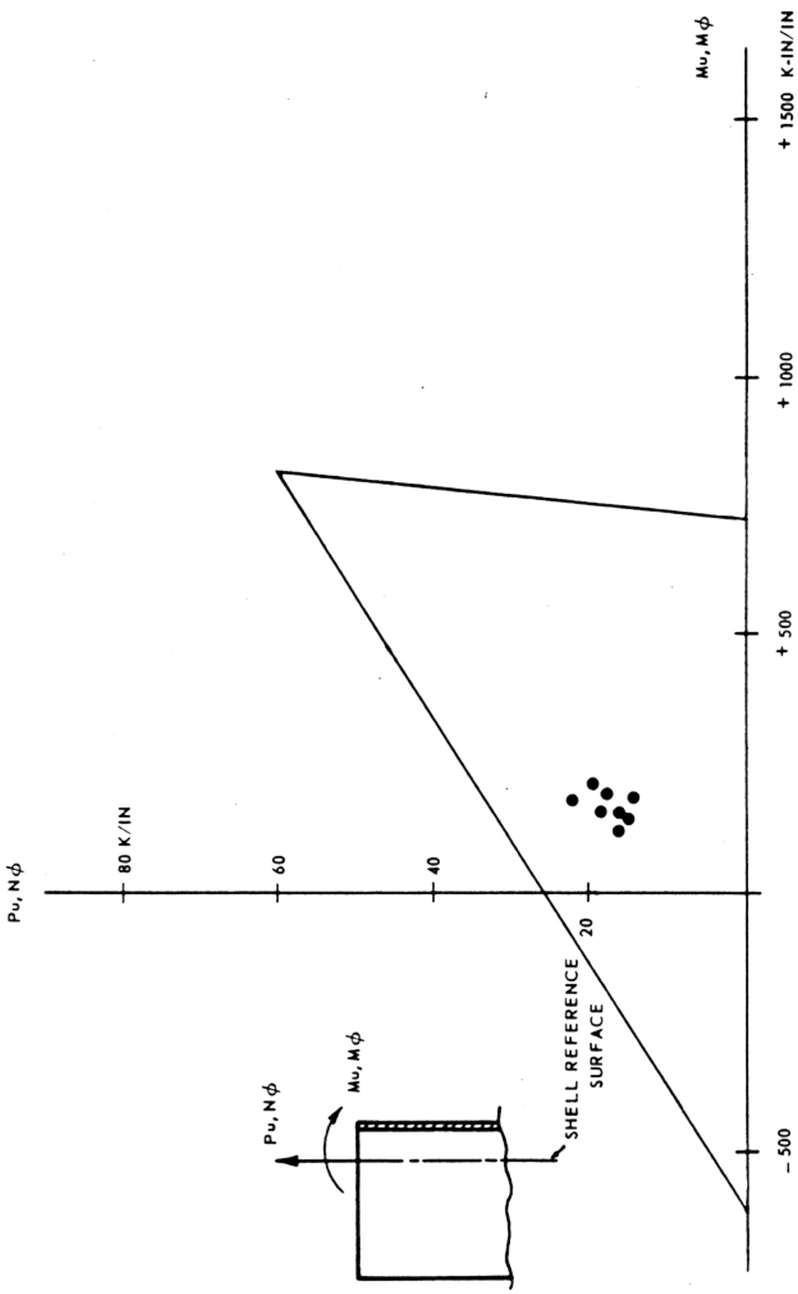
FIGURE 23

Figure 24

INTERACTION DIAGRAM

ELEMENT NO. 55



GILBERT ASSOCIATES, INC.

FIGURE 24

Drawings

APPENDIX 3B

D R A W I N G S

GILBERT ASSOCIATES, INC.

Figure Drawing 1Reactor Containment Vessel - Equipment/Personnel Access Reinforcement - Enlarged Sections

Figure Deleted

Refer to Drawing
D421-0024 Rev. 006

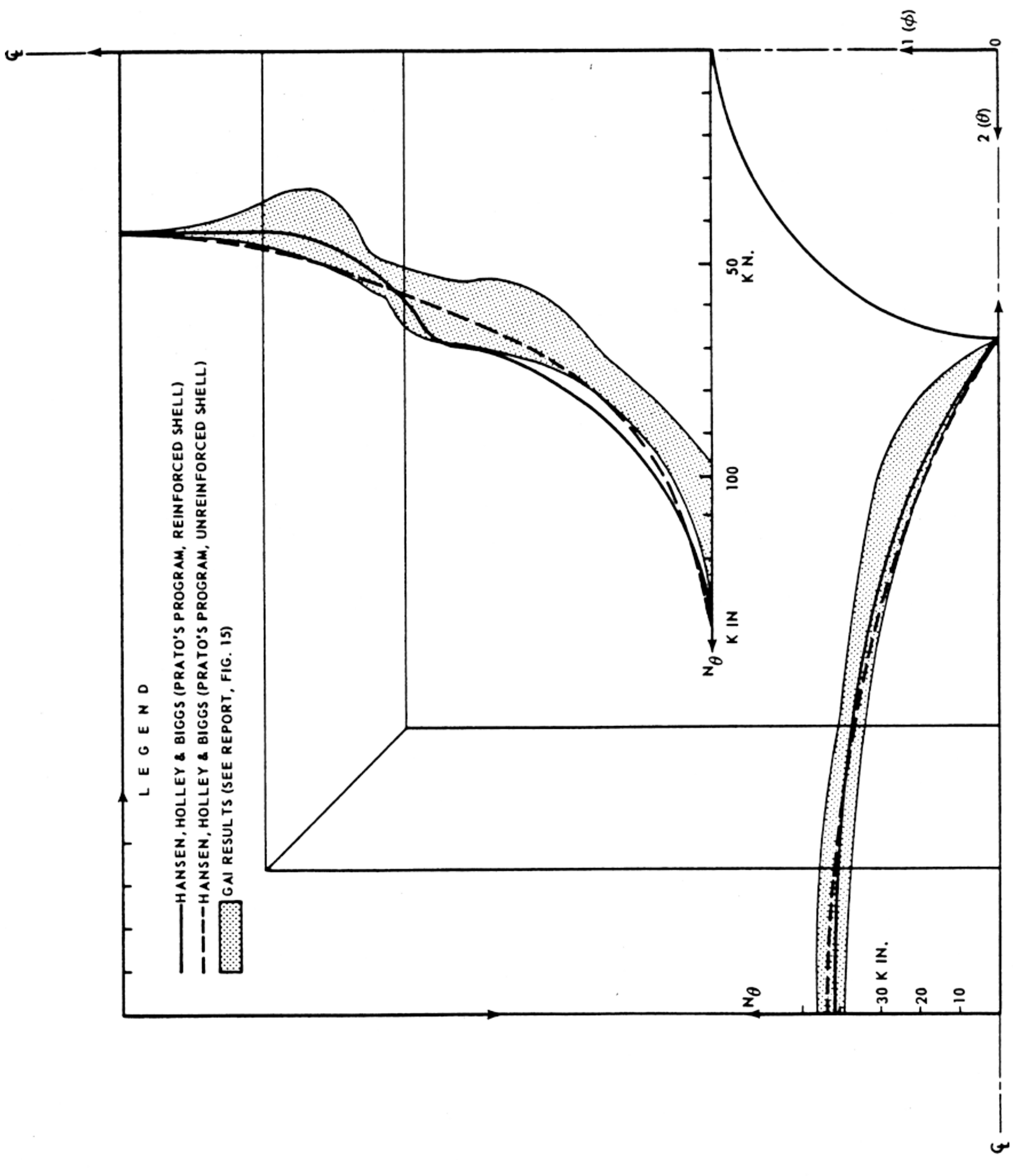
Figure Drawing 2Reactor Containment Vessel - Equipment Access Opening Reinforcement - Stretch-out & Sections

Figure Deleted

Refer to Drawing
D421-0022 Rev. 007

Figure I *Comparison of H.H. & GAI Results Hoop Stress Resultants Along Horizontal and Vertical Symmetry Axes (Internal Pressure = 69 PSI)*

COMPARISON OF HH & B AND GAI RESULTS
HOOP STRESS RESULTANTS ALONG HORIZONTAL AND VERTICAL SYMMETRY AXES
(INTERNAL PRESSURE = 69 PSI)



GILBERT ASSOCIATES, INC. FIGURE I

Figure Drawing 1Reactor Containment Vessel - Equipment/Personnel Access Reinforcement - Enlarged Sections

Figure Deleted

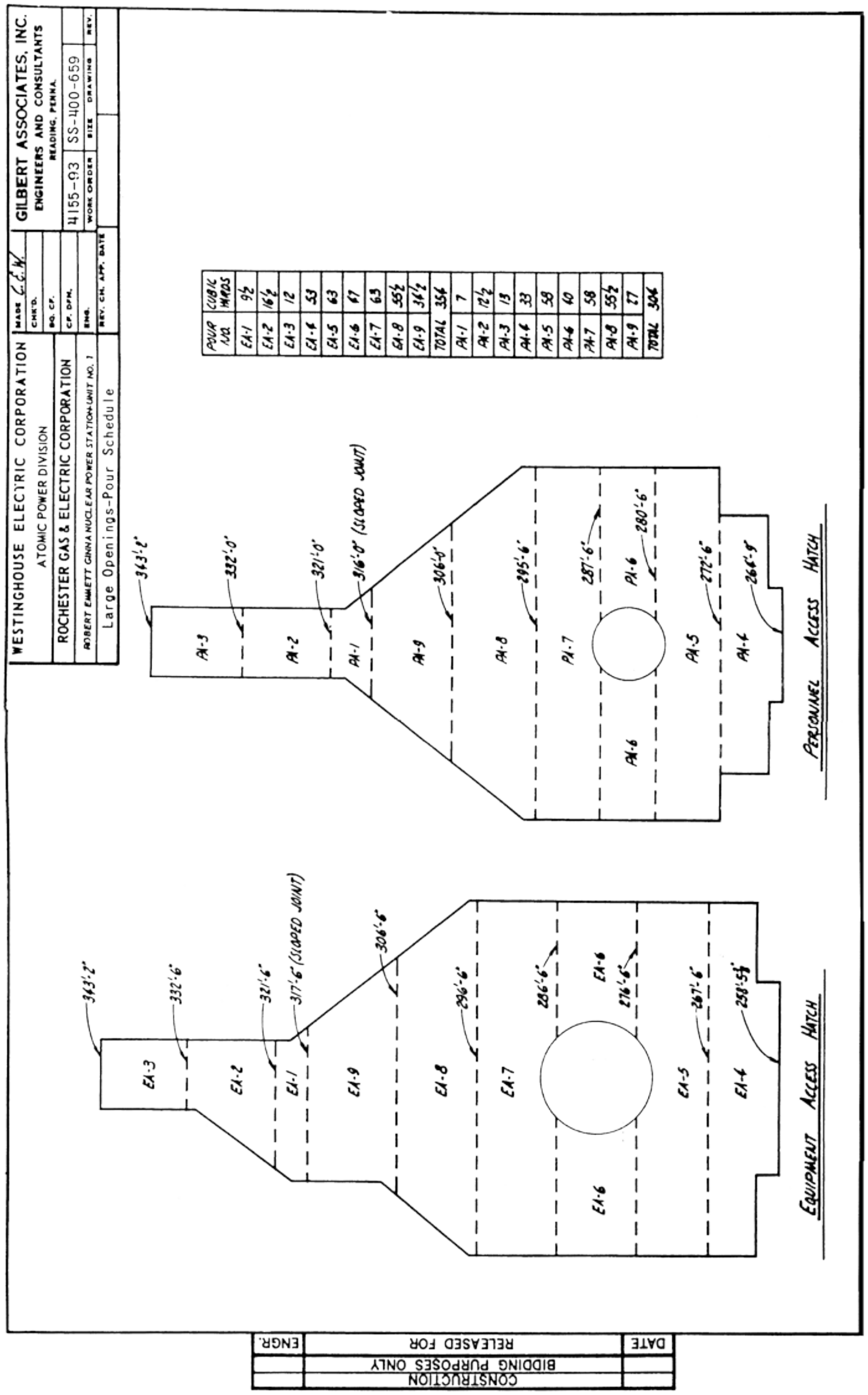
Refer to Drawing
D421-0024 Rev. 006

Figure Drawing 2Reactor Containment Vessel - Equipment Access Opening Reinforcement - Stretch-out & Sections

Figure Deleted

Refer to Drawing
D421-0022 Rev. 007

Figure Drawing 3 Large Openings - Pour Schedule



APPENDIX 3C

CONTAINMENT SHELL STRESS CALCULATION RESULTS

Table 3C-1 CONTAINMENT SHELL STRESS CALCULATION RESULTS

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-70.900	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
4	0.0	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
5	227.000	-30.000	79.600	0.0	0.0	55.300	0.0	0.099
6	8.000	0.0	-1.500	0.0	0.0	1.200	0.0	0.0
7	8.000	0.0	-1.500	0.0	0.0	1.200	0.0	0.0
8	70.300	0.0	0.0	0.0	0.0	0.0	0.0	0.0

COMBINED STRESSES - ELEMENT NO. 0

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-369.900	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
2	-420.729	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
3	-369.900	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
4	-420.729	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
5	-229.300	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
6	-280.129	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
7	-229.300	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
8	-280.129	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
9	-510.500	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
10	-561.329	0.0	130.200	99.500	0.0	-6.600	0.0	0.0
11	-510.500	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
12	-561.329	0.0	-130.200	0.0	0.0	6.600	0.0	0.0
13	-108.850	-34.500	221.740	99.500	0.0	56.995	0.0	0.114
14	-159.680	-34.500	221.740	99.500	0.0	56.995	0.0	0.114
15	-108.850	-34.500	-38.660	0.0	0.0	70.195	0.0	0.114
16	-159.680	-34.500	-38.660	0.0	0.0	70.195	0.0	0.114
17	-134.900	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
18	-185.729	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
19	-134.900	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
20	-185.729	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
21	-78.660	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
22	-129.490	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
23	-78.660	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
24	-129.490	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
25	-191.140	-30.000	708.300	99.500	0.0	49.900	0.0	0.099
26	-241.969	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
27	-191.140	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
28	-241.969	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
29	-21.400	-45.000	248.100	99.500	0.0	77.550	0.0	0.148
30	-72.229	-45.000	248.100	99.500	0.0	77.550	0.0	0.148
31	-21.400	-45.000	-12.300	0.0	0.0	90.750	0.0	0.148
32	-72.229	-45.000	-12.300	0.0	0.0	90.750	0.0	0.148
33	-7.850	-37.500	228.200	99.500	0.0	63.725	0.0	0.124
34	-58.680	-37.500	228.200	99.500	0.0	63.725	0.0	0.124
35	-7.850	-37.500	-32.200	0.0	0.0	76.925	0.0	0.124
36	-58.680	-37.500	-32.200	0.0	0.0	76.925	0.0	0.124
37	-148.450	-37.500	228.200	99.500	0.0	63.725	0.0	0.124
38	-199.279	-37.500	228.200	99.500	0.0	63.725	0.0	0.124
39	-148.450	-37.500	-32.200	0.0	0.0	76.925	0.0	0.124
40	-199.279	-37.500	-32.200	0.0	0.0	76.925	0.0	0.124
41	5.700	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
42	-45.130	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
43	5.700	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
44	-45.130	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
45	-275.500	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
46	-326.329	-30.000	208.300	99.500	0.0	49.900	0.0	0.099
47	-275.500	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099
48	-326.329	-30.000	-52.100	0.0	0.0	63.100	0.0	0.099

Sheet 2 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 3

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-69.400	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.038
4	0.0	16.100	-38.300	0.0	0.0	4.200	0.0	0.101
5	227.000	106.000	127.400	0.0	0.0	36.200	0.0	0.149
6	8.000	2.500	-0.600	0.0	0.0	0.800	0.0	0.001
7	8.000	2.500	-0.600	0.0	0.0	0.800	0.0	0.001
8	68.300	0.0	0.0	0.0	0.0	0.0	0.0	0.002

COMBINED STRESSES - ELEMENT NO. 3

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-368.400	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.038
2	-419.229	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.038
3	-368.400	16.100	-38.300	0.0	0.0	4.200	0.0	0.101
4	-419.229	16.100	-38.300	0.0	0.0	4.200	0.0	0.101
5	-231.800	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.034
6	-282.629	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.034
7	-231.800	16.100	-38.300	0.0	0.0	4.200	0.0	0.105
8	-282.629	16.100	-38.300	0.0	0.0	4.200	0.0	0.105
9	-505.000	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.042
10	-555.829	-9.700	95.600	99.500	0.0	-0.300	0.0	-0.042
11	-505.000	16.100	-38.300	0.0	0.0	4.200	0.0	0.097
12	-555.829	16.100	-38.300	0.0	0.0	4.200	0.0	0.097
13	-107.350	112.200	242.110	99.500	0.0	41.330	0.0	0.133
14	-158.180	112.200	242.110	99.500	0.0	41.330	0.0	0.133
15	-107.350	138.000	108.210	0.0	0.0	45.830	0.0	0.272
16	-158.180	138.000	108.210	0.0	0.0	45.830	0.0	0.272
17	-133.400	98.800	222.400	99.500	0.0	36.700	0.0	0.112
18	-184.229	98.800	222.400	99.500	0.0	36.700	0.0	0.112
19	-133.400	124.600	88.500	0.0	0.0	41.200	0.0	0.251
20	-184.229	124.600	88.500	0.0	0.0	41.200	0.0	0.251
21	-78.760	98.800	222.400	99.500	0.0	36.700	0.0	0.114
22	-129.590	98.800	222.400	99.500	0.0	36.700	0.0	0.114
23	-78.760	124.600	88.500	0.0	0.0	41.200	0.0	0.253
24	-129.590	124.600	88.500	0.0	0.0	41.200	0.0	0.253
25	-188.040	98.800	222.400	99.500	0.0	36.700	0.0	0.110
26	-238.869	98.800	222.400	99.500	0.0	36.700	0.0	0.110
27	-188.040	124.600	88.500	0.0	0.0	41.200	0.0	0.249
28	-238.869	124.600	88.500	0.0	0.0	41.200	0.0	0.249
29	-19.900	151.800	286.100	99.500	0.0	54.800	0.0	0.186
30	-70.729	151.800	286.100	99.500	0.0	54.800	0.0	0.186
31	-19.900	177.600	152.200	0.0	0.0	59.300	0.0	0.325
32	-70.729	177.600	152.200	0.0	0.0	59.300	0.0	0.325
33	-8.350	125.300	254.250	99.500	0.0	45.750	0.0	0.151
34	-59.180	125.300	254.250	99.500	0.0	45.750	0.0	0.151
35	-8.350	151.100	120.350	0.0	0.0	50.250	0.0	0.290
36	-59.180	151.100	120.350	0.0	0.0	50.250	0.0	0.290
37	-144.950	125.300	254.250	99.500	0.0	45.750	0.0	0.147
38	-195.779	125.300	254.250	99.500	0.0	45.750	0.0	0.147
39	-144.950	151.100	120.350	0.0	0.0	50.250	0.0	0.286
40	-195.779	151.100	120.350	0.0	0.0	50.250	0.0	0.286
41	3.200	98.800	222.400	99.500	0.0	36.700	0.0	0.116
42	-47.630	98.800	222.400	99.500	0.0	36.700	0.0	0.116
43	3.200	124.600	88.500	0.0	0.0	41.200	0.0	0.255
44	-47.630	124.600	88.500	0.0	0.0	41.200	0.0	0.255
45	-270.000	98.800	222.400	99.500	0.0	36.700	0.0	0.108
46	-320.829	98.800	222.400	99.500	0.0	36.700	0.0	0.108
47	-270.000	124.600	88.500	0.0	0.0	41.200	0.0	0.247
48	-320.829	124.600	88.500	0.0	0.0	41.200	0.0	0.247

Sheet 3 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 6

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-67.800	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	-3.600	62.000	99.500	0.0	4.000	0.0	-0.075
4	0.0	25.900	-30.100	0.0	0.0	2.400	0.0	0.110
5	227.000	190.600	199.400	0.0	0.0	20.900	0.0	0.226
6	8.000	4.300	1.200	0.0	0.0	0.500	0.0	0.003
7	8.000	4.300	1.200	0.0	0.0	0.500	0.0	0.003
8	66.300	0.0	0.0	0.0	0.0	0.0	0.0	0.003

COMBINED STRESSES - ELEMENT NO. 6

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-366.800	-3.600	62.000	99.500	0.0	4.000	0.0	-0.075
2	-417.629	-3.600	62.000	99.500	0.0	4.000	0.0	-0.075
3	-366.800	25.900	-30.100	0.0	0.0	2.400	0.0	0.110
4	-417.629	25.900	-30.100	0.0	0.0	2.400	0.0	0.110
5	-234.200	-3.600	62.000	99.500	0.0	4.000	0.0	-0.069
6	-285.029	-3.600	62.000	99.500	0.0	4.000	0.0	-0.069
7	-234.200	25.900	-30.100	0.0	0.0	2.400	0.0	0.116
8	-285.029	25.900	-30.100	0.0	0.0	2.400	0.0	0.116
9	-499.400	-3.600	62.000	99.500	0.0	4.000	0.0	-0.081
10	-550.229	-3.600	62.000	99.500	0.0	4.000	0.0	-0.081
11	-499.400	25.900	-30.100	0.0	0.0	2.400	0.0	0.104
12	-550.229	25.900	-30.100	0.0	0.0	2.400	0.0	0.104
13	-105.750	215.590	291.310	99.500	0.0	28.035	0.0	0.185
14	-156.580	215.590	291.310	99.500	0.0	28.035	0.0	0.185
15	-105.750	245.090	199.210	0.0	0.0	26.435	0.0	0.370
16	-156.580	245.090	199.210	0.0	0.0	26.435	0.0	0.370
17	-131.800	191.300	262.600	99.500	0.0	25.400	0.0	0.154
18	-182.629	191.300	267.600	99.500	0.0	25.400	0.0	0.154
19	-131.800	220.800	170.500	0.0	0.0	23.800	0.0	0.339
20	-182.629	220.800	170.500	0.0	0.0	23.800	0.0	0.339
21	-78.760	191.300	262.600	99.500	0.0	25.400	0.0	0.156
22	-129.589	191.300	262.600	99.500	0.0	25.400	0.0	0.156
23	-78.760	220.800	170.500	0.0	0.0	23.800	0.0	0.341
24	-129.589	220.800	170.500	0.0	0.0	23.800	0.0	0.341
25	-184.840	191.300	262.600	99.500	0.0	25.400	0.0	0.152
26	-235.669	191.300	262.600	99.500	0.0	25.400	0.0	0.152
27	-184.840	220.800	170.500	0.0	0.0	23.800	0.0	0.337
28	-235.669	220.800	170.500	0.0	0.0	23.800	0.0	0.337
29	-18.300	286.600	362.300	99.500	0.0	35.850	0.0	0.267
30	-69.129	286.600	362.300	99.500	0.0	35.850	0.0	0.267
31	-18.300	316.100	270.200	0.0	0.0	34.250	0.0	0.452
32	-69.129	316.100	270.200	0.0	0.0	34.250	0.0	0.452
33	-8.750	238.950	312.450	99.500	0.0	30.625	0.0	0.213
34	-59.579	238.950	312.450	99.500	0.0	30.625	0.0	0.213
35	-8.750	268.450	220.350	0.0	0.0	29.025	0.0	0.398
36	-59.579	268.450	220.350	0.0	0.0	29.025	0.0	0.398
37	-141.350	238.950	312.450	99.500	0.0	30.625	0.0	0.207
38	-192.179	238.950	312.450	99.500	0.0	30.625	0.0	0.207
39	-141.350	268.450	220.350	0.0	0.0	29.025	0.0	0.392
40	-192.179	268.450	220.350	0.0	0.0	29.025	0.0	0.392
41	0.800	191.300	262.600	99.500	0.0	25.400	0.0	0.160
42	-50.029	191.300	262.600	99.500	0.0	25.400	0.0	0.160
43	0.800	220.800	170.500	0.0	0.0	23.800	0.0	0.345
44	-50.029	220.800	170.500	0.0	0.0	23.800	0.0	0.345
45	-264.400	191.300	262.600	99.500	0.0	25.400	0.0	0.148
46	-315.229	191.300	262.600	99.500	0.0	25.400	0.0	0.148
47	-264.400	220.800	170.500	0.0	0.0	23.800	0.0	0.333
48	-315.229	220.800	170.500	0.0	0.0	23.800	0.0	0.333

Sheet 4 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 10

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-65.800	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	19.800	27.300	99.500	0.0	7.200	0.0	-0.113
4	0.0	31.600	-19.100	0.0	0.0	0.600	0.0	0.122
5	227.000	243.000	282.200	0.0	0.0	6.200	0.0	0.314
6	8.000	5.500	3.000	0.0	0.0	0.100	0.0	0.005
7	8.000	5.500	3.000	0.0	0.0	0.100	0.0	0.005
8	63.600	0.0	0.0	0.0	0.0	0.0	0.0	0.005

COMBINED STRESSES - ELEMENT NO. 10

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-364.800	19.800	27.300	99.500	0.0	7.200	0.0	-0.113
2	-415.629	19.800	27.300	99.500	0.0	7.200	0.0	-0.113
3	-364.800	31.600	-19.100	0.0	0.0	0.600	0.0	0.122
4	-415.629	31.600	-19.100	0.0	0.0	0.600	0.0	0.122
5	-237.600	19.800	27.300	99.500	0.0	7.200	0.0	-0.103
6	-288.429	19.800	27.300	99.500	0.0	7.200	0.0	-0.103
7	-237.600	31.600	-19.100	0.0	0.0	0.600	0.0	0.132
8	-288.429	31.600	-19.100	0.0	0.0	0.600	0.0	0.132
9	-492.000	19.800	27.300	99.500	0.0	7.200	0.0	-0.123
10	-542.829	19.800	27.300	99.500	0.0	7.200	0.0	-0.123
11	-492.000	31.600	-19.100	0.0	0.0	0.600	0.0	0.112
12	-542.829	31.600	-19.100	0.0	0.0	0.600	0.0	0.112
13	-103.750	299.250	351.830	99.500	0.0	14.330	0.0	0.248
14	-154.580	299.250	351.830	99.500	0.0	14.330	0.0	0.248
15	-103.750	311.050	305.430	0.0	0.0	7.730	0.0	0.483
16	-154.580	311.050	305.430	0.0	0.0	7.730	0.0	0.483
17	-129.800	268.300	312.500	99.500	0.0	13.500	0.0	0.206
18	-180.629	268.300	312.500	99.500	0.0	13.500	0.0	0.206
19	-129.800	280.100	266.100	0.0	0.0	6.900	0.0	0.441
20	-180.629	280.100	266.100	0.0	0.0	6.900	0.0	0.441
21	-78.920	268.300	312.500	99.500	0.0	13.500	0.0	0.210
22	-129.749	268.300	312.500	99.500	0.0	13.500	0.0	0.210
23	-78.920	280.100	266.100	0.0	0.0	6.900	0.0	0.445
24	-129.749	280.100	266.100	0.0	0.0	6.900	0.0	0.445
25	-180.680	268.300	312.500	99.500	0.0	13.500	0.0	0.202
26	-231.509	268.300	312.500	99.500	0.0	13.500	0.0	0.202
27	-180.680	280.100	266.100	0.0	0.0	6.900	0.0	0.437
28	-231.509	280.100	266.100	0.0	0.0	6.900	0.0	0.437
29	-16.300	389.800	453.600	99.500	0.0	16.600	0.0	0.363
30	-67.129	389.800	453.600	99.500	0.0	16.600	0.0	0.363
31	-16.300	401.600	407.200	0.0	0.0	10.000	0.0	0.598
32	-67.129	401.600	407.200	0.0	0.0	10.000	0.0	0.598
33	-9.450	329.050	383.050	99.500	0.0	15.050	0.0	0.289
34	-60.279	329.050	383.050	99.500	0.0	15.050	0.0	0.289
35	-9.450	340.850	336.650	0.0	0.0	8.450	0.0	0.524
36	-60.279	340.850	336.650	0.0	0.0	8.450	0.0	0.524
37	-136.650	329.050	383.050	99.500	0.0	15.050	0.0	0.279
38	-187.479	329.050	383.050	99.500	0.0	15.050	0.0	0.279
39	-136.650	340.850	336.650	0.0	0.0	8.450	0.0	0.514
40	-187.479	340.850	336.650	0.0	0.0	8.450	0.0	0.514
41	-2.600	268.300	312.500	99.500	0.0	13.500	0.0	0.216
42	-53.429	268.300	312.500	99.500	0.0	13.500	0.0	0.216
43	-2.600	280.100	266.100	0.0	0.0	6.900	0.0	0.451
44	-53.429	280.100	266.100	0.0	0.0	6.900	0.0	0.451
45	-257.000	268.300	312.500	99.500	0.0	13.500	0.0	0.196
46	-307.829	268.300	312.500	99.500	0.0	13.500	0.0	0.196
47	-257.000	280.100	266.100	0.0	0.0	6.900	0.0	0.431
48	-307.829	280.100	266.100	0.0	0.0	6.900	0.0	0.431

Sheet 5 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 15

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)		MERIDIONAL SHEAR (K/FT)		RADIAL DISPLACEMENT (IN)
1	-63.300	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	59.700	0.0	99.500	0.0	8.400	0.0	-0.143
4	0.0	30.900	-15.500	0.0	0.0	-0.700	0.0	0.126
5	227.000	243.600	363.100	0.0	0.0	-4.800	0.0	0.401
6	8.000	5.500	5.000	0.0	0.0	-0.100	0.0	0.007
7	8.000	5.500	5.000	0.0	0.0	-0.100	0.0	0.007
8	60.200	0.0	0.0	0.0	0.0	0.0	0.0	0.007

COMBINED STRESSES - ELEMENT NO. 15

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)		MERIDIONAL SHEAR (K/FT)		RADIAL DISPLACEMENT (IN)
1	-362.300	59.700	0.0	99.500	0.0	8.400	0.0	-0.143
2	-413.129	59.700	0.0	99.500	0.0	8.400	0.0	-0.143
3	-362.300	30.900	-15.500	0.0	0.0	-0.700	0.0	0.126
4	-413.129	30.900	-15.500	0.0	0.0	-0.700	0.0	0.126
5	-241.900	59.700	0.0	99.500	0.0	8.400	0.0	-0.129
6	-292.729	59.700	0.0	99.500	0.0	8.400	0.0	-0.129
7	-241.900	30.900	-15.500	0.0	0.0	-0.700	0.0	0.140
8	-292.729	30.900	-15.500	0.0	0.0	-0.700	0.0	0.140
9	-482.700	59.700	0.0	99.500	0.0	8.400	0.0	-0.157
10	-533.529	59.700	0.0	99.500	0.0	8.400	0.0	-0.157
11	-482.700	30.900	-15.500	0.0	0.0	-0.700	0.0	0.112
12	-533.529	30.900	-15.500	0.0	0.0	-0.700	0.0	0.112
13	-101.250	339.840	417.564	99.500	0.0	2.880	0.0	0.318
14	-152.080	339.840	417.564	99.500	0.0	2.880	0.0	0.318
15	-101.250	311.040	402.064	0.0	0.0	-6.220	0.0	0.587
16	-152.080	311.040	402.064	0.0	0.0	-6.220	0.0	0.587
17	-127.300	308.800	368.100	99.500	0.0	3.500	0.0	0.265
18	-178.129	308.800	368.100	99.500	0.0	3.500	0.0	0.265
19	-127.300	280.000	352.600	0.0	0.0	-5.600	0.0	0.534
20	-178.129	280.000	352.600	0.0	0.0	-5.600	0.0	0.534
21	-79.140	308.800	368.100	99.500	0.0	3.500	0.0	0.271
22	-129.969	308.800	368.100	99.500	0.0	3.500	0.0	0.271
23	-79.140	280.000	352.600	0.0	0.0	-5.600	0.0	0.540
24	-129.969	280.000	352.600	0.0	0.0	-5.600	0.0	0.540
25	-175.460	308.800	368.100	99.500	0.0	3.500	0.0	0.259
26	-226.289	308.800	368.100	99.500	0.0	3.500	0.0	0.259
27	-175.460	280.000	352.600	0.0	0.0	-5.600	0.0	0.528
28	-226.289	280.000	352.600	0.0	0.0	-5.600	0.0	0.528
29	-13.800	430.600	549.650	99.500	0.0	1.100	0.0	0.465
30	-64.629	430.600	549.650	99.500	0.0	1.100	0.0	0.465
31	-13.800	401.800	534.150	0.0	0.0	-8.000	0.0	0.734
32	-64.629	401.800	534.150	0.0	0.0	-8.000	0.0	0.734
33	-10.350	369.700	458.875	99.500	0.0	2.300	0.0	0.372
34	-61.179	369.700	458.875	99.500	0.0	2.300	0.0	0.372
35	-10.350	340.900	443.375	0.0	0.0	-6.800	0.0	0.641
36	-61.179	340.900	443.375	0.0	0.0	-6.800	0.0	0.641
37	-130.750	369.700	458.875	99.500	0.0	2.300	0.0	0.358
38	-181.579	369.700	458.875	99.500	0.0	2.300	0.0	0.358
39	-130.750	340.900	443.375	0.0	0.0	-6.800	0.0	0.627
40	-181.579	340.900	443.375	0.0	0.0	-6.800	0.0	0.627
41	-6.900	308.800	368.100	99.500	0.0	3.500	0.0	0.279
42	-57.729	308.800	368.100	99.500	0.0	3.500	0.0	0.279
43	-6.900	280.000	352.600	0.0	0.0	-5.600	0.0	0.548
44	-57.729	280.000	352.600	0.0	0.0	-5.600	0.0	0.548
45	-247.700	308.800	368.100	99.500	0.0	3.500	0.0	0.251
46	-298.529	308.800	368.100	99.500	0.0	3.500	0.0	0.251
47	-247.700	280.000	352.600	0.0	0.0	-5.600	0.0	0.520
48	-298.529	280.000	352.600	0.0	0.0	-5.600	0.0	0.520

Sheet 6 of Table 3C-1 Elements 20, 30, 40, 60, 75

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 20

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-60.500	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	100.200	-14.600	99.500	0.0	7.600	0.0	-0.159
4	0.0	25.700	-2.700	0.0	0.0	-1.300	0.0	0.140
5	227.000	205.700	418.800	0.0	0.0	-9.700	0.0	0.460
6	8.000	4.600	6.000	0.0	0.0	-0.200	0.0	0.008
7	8.000	4.600	6.000	0.0	0.0	-0.200	0.0	0.008
8	56.900	0.0	0.0	0.0	0.0	0.0	0.0	0.010

COMBINED STRESSES - ELEMENT NO. 20

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-359.500	100.200	-14.600	99.500	0.0	7.600	0.0	-0.159
2	-410.330	100.200	-14.600	99.500	0.0	7.600	0.0	-0.159
3	-359.500	25.700	-2.700	0.0	0.0	-1.300	0.0	0.140
4	-410.330	25.700	-2.700	0.0	0.0	-1.300	0.0	0.140
5	-245.700	100.200	-14.600	99.500	0.0	7.600	0.0	-0.139
6	-296.530	100.200	-14.600	99.500	0.0	7.600	0.0	-0.139
7	-245.700	25.700	-2.700	0.0	0.0	-1.300	0.0	0.160
8	-296.530	25.700	-2.700	0.0	0.0	-1.300	0.0	0.160
9	-473.300	100.200	-14.600	99.500	0.0	7.600	0.0	-0.179
10	-524.129	100.200	-14.600	99.500	0.0	7.600	0.0	-0.179
11	-473.300	25.700	-2.700	0.0	0.0	-1.300	0.0	0.120
12	-524.129	25.700	-2.700	0.0	0.0	-1.300	0.0	0.120
13	-98.450	336.755	467.019	99.500	0.0	-3.555	0.0	0.370
14	-149.280	336.755	467.019	99.500	0.0	-3.555	0.0	0.370
15	-98.450	262.255	478.919	0.0	0.0	-12.455	0.0	0.669
16	-149.280	262.255	478.919	0.0	0.0	-12.455	0.0	0.669
17	-124.500	310.500	410.200	99.500	0.0	-2.300	0.0	0.309
18	-175.330	310.500	410.200	99.500	0.0	-2.300	0.0	0.309
19	-124.500	236.000	422.100	0.0	0.0	-11.200	0.0	0.608
20	-175.330	236.000	422.100	0.0	0.0	-11.200	0.0	0.608
21	-78.980	310.500	410.200	99.500	0.0	-2.300	0.0	0.317
22	-129.810	310.500	410.200	99.500	0.0	-2.300	0.0	0.317
23	-78.980	236.000	422.100	0.0	0.0	-11.200	0.0	0.616
24	-129.810	236.000	422.100	0.0	0.0	-11.200	0.0	0.616
25	-170.020	310.500	410.200	99.500	0.0	-2.300	0.0	0.301
26	-220.850	310.500	410.200	99.500	0.0	-2.300	0.0	0.301
27	-170.020	236.000	422.100	0.0	0.0	-11.200	0.0	0.600
28	-220.850	236.000	422.100	0.0	0.0	-11.200	0.0	0.600
29	-11.000	413.350	619.600	99.500	0.0	-7.150	0.0	0.539
30	-61.830	413.350	619.600	99.500	0.0	-7.150	0.0	0.539
31	-11.000	338.850	631.500	0.0	0.0	-16.050	0.0	0.838
32	-61.830	338.850	631.500	0.0	0.0	-16.050	0.0	0.838
33	-10.850	361.925	514.900	99.500	0.0	-4.725	0.0	0.434
34	-61.680	361.925	514.900	99.500	0.0	-4.725	0.0	0.434
35	-10.850	287.425	526.800	0.0	0.0	-13.625	0.0	0.733
36	-61.680	287.425	526.800	0.0	0.0	-13.625	0.0	0.733
37	-124.650	361.925	514.900	99.500	0.0	-4.725	0.0	0.414
38	-175.480	361.925	514.900	99.500	0.0	-4.725	0.0	0.414
39	-124.650	287.425	526.800	0.0	0.0	-13.625	0.0	0.713
40	-175.480	287.425	526.800	0.0	0.0	-13.625	0.0	0.713
41	-10.700	310.500	410.200	99.500	0.0	-2.300	0.0	0.329
42	-61.530	310.500	410.200	99.500	0.0	-2.300	0.0	0.329
43	-10.700	236.000	422.100	0.0	0.0	-11.200	0.0	0.628
44	-61.530	236.000	422.100	0.0	0.0	-11.200	0.0	0.628
45	-238.300	310.500	410.200	99.500	0.0	-2.300	0.0	0.289
46	-289.129	310.500	410.200	99.500	0.0	-2.300	0.0	0.289
47	-238.300	236.000	422.100	0.0	0.0	-11.200	0.0	0.588
48	-289.129	236.000	422.100	0.0	0.0	-11.200	0.0	0.588

Sheet 7 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 30

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-55.700	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	160.400	-19.200	99.500	0.0	4.300	0.0	-0.164
4	0.0	12.500	2.700	0.0	0.0	-1.200	0.0	0.146
5	227.000	102.800	469.000	0.0	0.0	-9.500	0.0	0.514
6	8.000	2.300	6.700	0.0	0.0	-0.200	0.0	0.009
7	8.000	2.300	6.700	0.0	0.0	-0.200	0.0	0.009
8	50.300	0.0	0.0	0.0	0.0	0.0	0.0	0.016

COMBINED STRESSES - ELEMENT NO. 30

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-354.700	160.400	-19.200	99.500	0.0	4.300	0.0	-0.164
2	-405.530	160.400	-19.200	99.500	0.0	4.300	0.0	-0.164
3	-354.700	12.500	2.700	0.0	0.0	-1.200	0.0	0.146
4	-405.530	12.500	2.700	0.0	0.0	-1.200	0.0	0.146
5	-254.100	160.400	-19.200	99.500	0.0	4.300	0.0	-0.132
6	-304.929	160.400	-19.200	99.500	0.0	4.300	0.0	-0.132
7	-254.100	12.500	2.700	0.0	0.0	-1.200	0.0	0.178
8	-304.929	12.500	2.700	0.0	0.0	-1.200	0.0	0.178
9	-455.300	160.400	-19.200	99.500	0.0	4.300	0.0	-0.196
10	-506.129	160.400	-19.200	99.500	0.0	4.400	0.0	-0.196
11	-455.300	12.500	2.700	0.0	0.0	-1.200	0.0	0.114
12	-506.129	12.500	2.700	0.0	0.0	-1.200	0.0	0.114
13	-93.650	278.620	520.149	99.500	0.0	-6.625	0.0	0.427
14	-144.480	278.620	520.149	99.500	0.0	-6.625	0.0	0.427
15	-93.650	130.720	542.050	0.0	0.0	-12.125	0.0	0.787
16	-144.480	130.720	542.050	0.0	0.0	-12.125	0.0	0.787
17	-119.700	265.500	456.500	99.500	0.0	-5.400	0.0	0.359
18	-170.530	265.500	456.500	99.500	0.0	-5.400	0.0	0.359
19	-119.700	117.600	478.400	0.0	0.0	-10.900	0.0	0.669
20	-170.530	117.600	478.400	0.0	0.0	-10.900	0.0	0.669
21	-79.460	265.500	456.500	99.500	0.0	-5.400	0.0	0.372
22	-130.290	265.500	456.500	99.500	0.0	-5.400	0.0	0.372
23	-79.460	117.600	478.400	0.0	0.0	-10.900	0.0	0.682
24	-130.290	117.600	478.400	0.0	0.0	-10.900	0.0	0.662
25	-159.940	265.500	456.500	99.500	0.0	-5.400	0.0	0.346
26	-210.770	265.500	456.500	99.500	0.0	-5.400	0.0	0.346
27	-159.940	117.600	478.400	0.0	0.0	-10.900	0.0	0.656
28	-210.770	117.600	478.400	0.0	0.0	-10.900	0.0	0.656
29	-6.200	316.900	691.000	99.500	0.0	-10.150	0.0	0.616
30	-57.030	316.900	691.000	99.500	0.0	-10.150	0.0	0.616
31	-6.200	169.000	712.900	0.0	0.0	-15.650	0.0	0.926
32	-57.030	169.000	712.900	0.0	0.0	-15.650	0.0	0.926
33	-12.650	291.200	573.750	99.500	0.0	-7.775	0.0	0.503
34	-63.480	291.200	573.750	99.500	0.0	-7.775	0.0	0.503
35	-12.650	143.300	595.650	0.0	0.0	-13.275	0.0	0.813
36	-63.480	143.300	595.650	0.0	0.0	-13.275	0.0	0.813
37	-113.250	291.200	573.750	99.500	0.0	-7.775	0.0	0.471
38	-164.080	291.200	573.750	99.500	0.0	-7.775	0.0	0.471
39	-113.250	143.300	595.650	0.0	0.0	-13.275	0.0	0.781
40	-164.080	143.300	595.650	0.0	0.0	-13.275	0.0	0.781
41	-19.100	265.500	456.500	99.500	0.0	-5.400	0.0	0.391
42	-69.930	265.500	456.500	99.500	0.0	-5.400	0.0	0.391
43	-19.100	117.600	478.400	0.0	0.0	-10.900	0.0	0.701
44	-69.930	117.600	478.400	0.0	0.0	-10.900	0.0	0.701
45	-220.300	265.500	456.500	99.500	0.0	-5.400	0.0	0.327
46	-271.129	265.500	456.500	99.500	0.0	-5.400	0.0	0.327
47	-220.300	117.600	478.400	0.0	0.0	-10.900	0.0	0.637
48	-271.129	117.600	478.400	0.0	0.0	-10.900	0.0	0.637

Sheet 8 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 40

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-50.500	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	188.000	-11.800	99.500	0.0	1.500	0.0	-0.156
4	0.0	3.300	2.700	0.0	0.0	-0.600	0.0	0.146
5	227.000	28.900	473.200	0.0	0.0	-5.200	0.0	0.518
6	8.000	0.600	6.700	0.0	0.0	-0.100	0.0	0.009
7	8.000	0.600	6.700	0.0	0.0	-0.100	0.0	0.009
8	46.700	0.0	0.0	0.0	0.0	0.0	0.0	0.021

COMBINED STRESSES - ELEMENT NO. 40

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-349.500	188.000	-11.800	99.500	0.0	1.500	0.0	-0.156
2	-400.330	188.000	-11.800	99.500	0.0	1.500	0.0	-0.156
3	-349.500	3.300	2.700	0.0	0.0	-0.600	0.0	0.146
4	-400.330	3.300	2.700	0.0	0.0	-0.600	0.0	0.146
5	-256.100	188.000	-11.800	99.500	0.0	1.500	0.0	-0.114
6	-306.929	188.000	-11.800	99.500	0.0	1.500	0.0	-0.114
7	-256.100	3.300	2.700	0.0	0.0	-0.600	0.0	0.188
8	-306.929	3.300	2.700	0.0	0.0	-0.600	0.0	0.188
9	-442.900	188.000	-11.800	99.500	0.0	1.500	0.0	-0.198
10	-493.729	188.000	-11.800	99.500	0.0	1.500	0.0	-0.198
11	-442.900	3.300	2.700	0.0	0.0	-0.600	0.0	0.104
12	-493.729	3.300	2.700	0.0	0.0	-0.600	0.0	0.104
13	-88.450	221.235	532.380	99.500	0.0	-4.480	0.0	0.440
14	-139.280	221.235	532.380	99.500	0.0	-4.480	0.0	0.440
15	-88.450	36.535	546.880	0.0	0.0	-6.560	0.0	0.742
16	-139.280	36.535	546.880	0.0	0.0	-6.580	0.0	0.742
17	-114.500	217.500	468.100	99.500	0.0	-3.800	0.0	0.371
18	-165.330	217.500	468.100	99.500	0.0	-3.800	0.0	0.371
19	-114.500	32.800	482.600	0.0	0.0	-5.900	0.0	0.673
20	-165.330	32.800	482.600	0.0	0.0	-5.900	0.0	0.673
21	-77.140	217.500	468.100	99.500	0.0	-3.800	0.0	0.388
22	-127.970	217.500	468.100	99.500	0.0	-3.800	0.0	0.388
23	-77.140	32.800	482.600	0.0	0.0	-5.900	0.0	0.690
24	-127.970	32.800	482.600	0.0	0.0	-5.900	0.0	0.690
25.	-151.860	217.500	468.100	99.500	0.0	-3.800	0.0	0.354
26	-202.690	217.500	468.100	99.500	0.0	-3.800	0.0	0.354
27	-151.860	32.800	482.600	0.0	0.0	-5.900	0.0	0.656
28	-202.690	32.800	482.600	0.0	0.0	-5.900	0.0	0.656
29	-1.000	231.950	704.700	99.500	0.0	-6.400	0.0	0.630
30	-51.830	231.950	704.700	99.500	0.0	-6.400	0.0	0.630
31	-1.000	47.250	719.200	0.0	0.0	-8.500	0.0	0.932
32	-51.830	47.250	719.200	0.0	0.0	-8.500	0.0	0.932
33	-11.050	224.725	586.400	99.500	0.0	-5.100	0.0	0.521
34	-61.880	224.725	586.400	99.500	0.0	-5.100	0.0	0.521
35	-11.050	40.025	600.900	0.0	0.0	-7.200	0.0	0.823
36	-61.880	40.025	600.900	0.0	0.0	-7.200	0.0	0.823
37	-104.450	224.725	586.400	99.500	0.0	-5.100	0.0	0.479
38	-155.280	224.725	586.400	99.500	0.0	-5.100	0.0	0.479
39	-104.450	40.025	600.900	0.0	0.0	-7.200	0.0	0.781
40	-155.280	40.025	600.900	0.0	0.0	-7.200	0.0	0.781
41	-21.100	217.500	468.100	99.500	0.0	-3.800	0.0	0.413
42	-71.930	217.500	468.100	99.500	0.0	-3.800	0.0	0.413
43	-21.100	32.800	482.600	0.0	0.0	-5.900	0.0	0.715
44	-71.930	32.800	482.600	0.0	0.0	-5.900	0.0	0.715
45	-207.900	217.500	468.100	99.500	0.0	-3.800	0.0	0.329
46	-258.729	217.500	468.100	99.500	0.0	-3.800	0.0	0.329
47	-207.900	32.800	482.600	0.0	0.0	-5.900	0.0	0.631
48	-258.729	32.800	482.600	0.0	0.0	-5.900	0.0	0.631

Sheet 9 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 60

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-40.300	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	192.300	0.0	99.500	0.0	-0.400	0.0	-0.144
4	0.0	-1.400	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	10.800	454.200	0.0	0.0	0.0	0.0	0.498
6	8.000	-0.200	6.700	0.0	0.0	0.0	0.0	0.009
7	8.000	-0.200	6.700	0.0	0.0	0.0	0.0	0.009
8	31.600	0.0	0.0	0.0	0.0	0.0	0.0	0.034

COMBINED STRESSES - ELEMENT NO. 60

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT
1	-339.300	192.300	0.0	99.500	0.0	-0.400	0.0	-0.144
2	-390.129	192.300	0.0	99.500	0.0	-0.400	0.0	-0.144
3	-339.300	-1.400	0.0	0.0	0.0	0.0	0.0	0.143
4	-390.129	-1.400	0.0	0.0	0.0	0.0	0.0	0.143
5	-276.100	192.300	0.0	99.500	0.0	-0.400	0.0	-0.076
6	-326.929	192.300	0.0	99.500	0.0	-0.400	0.0	-0.076
7	-276.100	-1.400	0.0	0.0	0.0	0.0	0.0	0.211
8	-326.929	-1.400	0.0	0.0	0.0	0.0	0.0	0.211
9	-402.500	192.300	0.0	99.500	0.0	-0.400	0.0	-0.212
10	-453.329	192.300	0.0	99.500	0.0	-0.400	0.0	-0.212
11	-402.500	-1.400	0.0	0.0	0.0	0.0	0.0	0.075
12	-453.329	-1.400	0.0	0.0	0.0	0.0	0.0	0.075
13	-78.250	204.720	522.330	99.500	0.0	-0.400	0.0	0.429
14	-129.080	204.720	522.330	99.500	0.0	-0.400	0.0	0.429
15	-78.250	11.020	522.330	0.0	0.0	0.0	0.0	0.716
16	-129.080	11.020	522.330	0.0	0.0	0.0	0.0	0.716
17	-104.300	202.900	460.900	99.500	0.0	-0.400	0.0	0.363
18	-155.129	202.900	460.900	99.500	0.0	-0.400	0.0	0.363
19	-104.300	9.200	460.900	0.0	0.0	0.0	0.0	0.650
20	-155.129	9.200	460.900	0.0	0.0	0.0	0.0	0.650
21	-79.020	202.900	460.900	99.500	0.0	-0.400	0.0	0.390
22	-129.849	202.900	460.900	99.500	0.0	-0.400	0.0	0.390
23	-79.020	9.200	460.900	0.0	0.0	0.0	0.0	0.677
24	-129.849	9.200	460.900	0.0	0.0	0.0	0.0	0.677
25	-129.580	202.900	460.900	99.500	0.0	-0.400	0.0	0.336
26	-180.409	202.900	460.900	99.500	0.0	-0.400	0.0	0.336
27	-129.580	9.200	460.900	0.0	0.0	0.0	0.0	0.623
28	-180.409	9.200	460.900	0.0	0.0	0.0	0.0	0.623
29	9.200	208.300	688.000	99.500	0.0	-0.400	0.0	0.612
30	-41.629	208.300	688.000	99.500	0.0	-0.400	0.0	0.612
31	9.200	14.600	688.000	0.0	0.0	0.0	0.0	0.899
32	-41.629	14.600	688.000	0.0	0.0	0.0	0.0	0.899
33	-15.950	205.600	574.450	99.500	0.0	-0.400	0.0	0.521
34	-66.779	205.600	574.450	99.500	0.0	-0.400	0.0	0.521
35	-15.950	11.900	574.450	0.0	0.0	0.0	0.0	0.808
36	-66.779	11.900	574.450	0.0	0.0	0.0	0.0	0.808
37	-79.150	205.600	574.450	99.500	0.0	-0.400	0.0	0.453
38	-129.979	205.600	574.450	99.500	0.0	-0.400	0.0	0.453
39	-79.150	11.900	574.450	0.0	0.0	0.0	0.0	0.740
40	-129.979	11.900	574.450	0.0	0.0	0.0	0.0	0.740
41	-41.100	202.900	460.900	99.500	0.0	-0.400	0.0	0.431
42	-91.929	202.900	460.900	99.500	0.0	-0.400	0.0	0.431
43	-41.100	9.200	460.900	0.0	0.0	0.0	0.0	0.718
44	-91.929	9.200	460.900	0.0	0.0	0.0	0.0	0.718
45	-167.500	202.900	460.900	99.500	0.0	-0.400	0.0	0.295
46	-218.329	202.900	460.900	99.500	0.0	-0.400	0.0	0.295
47	-167.500	9.200	460.900	0.0	0.0	0.0	0.0	0.582
48	-218.329	9.200	460.900	0.0	0.0	0.0	0.0	0.582

Sheet 10 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 75

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-32.700	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	185.600	0.0	99.500	0.0	0.0	0.0	-0.142
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	-7.100	454.000	0.0	0.0	0.0	0.0	0.492
6	8.000	0.0	6.700	0.0	0.0	0.0	0.0	0.009
7	8.000	0.0	6.700	0.0	0.0	0.0	0.0	0.009
8	23.300	0.0	0.0	0.0	0.0	0.0	0.0	0.044

COMBINED STRESSES - ELEMENT NO. 75

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-331.700	185.600	0.0	99.500	0.0	0.0	0.0	-0.142
2	-382.530	185.600	0.0	99.500	0.0	0.0	0.0	-0.142
3	-331.700	0.0	0.0	0.0	0.0	0.0	0.0	0.143
4	-382.530	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	-285.100	185.600	0.0	99.500	0.0	0.0	0.0	-0.054
6	-335.929	185.600	0.0	99.500	0.0	0.0	0.0	-0.054
7	-285.100	0.0	0.0	0.0	0.0	0.0	0.0	0.231
8	-335.929	0.0	0.0	0.0	0.0	0.0	0.0	0.231
9	-378.300	185.600	0.0	99.500	0.0	0.0	0.0	-0.230
10	-429.129	185.600	0.0	99.500	0.0	0.0	0.0	-0.230
11	-378.300	0.0	0.0	0.0	0.0	0.0	0.0	0.055
12	-429.129	0.0	0.0	0.0	0.0	0.0	0.0	0.055
13	-70.650	177.435	522.100	99.500	0.0	0.0	0.0	0.424
14	-121.480	177.435	522.100	99.500	0.0	0.0	0.0	0.424
15	-70.650	-8.165	522.100	0.0	0.0	0.0	0.0	0.709
16	-121.480	-8.165	522.100	0.0	0.0	0.0	0.0	0.709
17	-96.700	178.500	460.700	99.500	0.0	0.0	0.0	0.359
18	-147.530	178.500	460.700	99.500	0.0	0.0	0.0	0.359
19	-96.700	-7.100	460.700	0.0	0.0	0.0	0.0	0.644
20	-147.530	-7.100	460.700	0.0	0.0	0.0	0.0	0.644
21	-78.060	178.500	460.700	99.500	0.0	0.0	0.0	0.394
22	-128.890	178.500	460.700	99.500	0.0	0.0	0.0	0.394
23	-78.060	-7.100	460.700	0.0	0.0	0.0	0.0	0.679
24	-128.890	-7.100	460.700	0.0	0.0	0.0	0.0	0.679
25	-115.340	178.500	460.700	99.500	0.0	0.0	0.0	0.324
26	-166.170	178.500	460.700	99.500	0.0	0.0	0.0	0.324
27	-115.340	-7.100	460.700	0.0	0.0	0.0	0.0	0.609
28	-166.170	-7.100	460.700	0.0	0.0	0.0	0.0	0.609
29	16.800	174.950	687.700	99.500	0.0	0.0	0.0	0.605
30	-34.030	174.950	687.700	99.500	0.0	0.0	0.0	0.605
31	16.800	-10.650	687.700	0.0	0.0	0.0	0.0	0.890
32	-34.030	-10.650	687.700	0.0	0.0	0.0	0.0	0.890
33	-16.650	176.725	574.200	99.500	0.0	0.0	0.0	0.526
34	-67.480	176.725	574.200	99.500	0.0	0.0	0.0	0.526
35	-16.650	-8.875	574.200	0.0	0.0	0.0	0.0	0.811
36	-67.480	-8.875	574.200	0.0	0.0	0.0	0.0	0.811
37	-63.250	176.725	574.200	99.500	0.0	0.0	0.0	0.438
38	-114.080	176.725	574.200	99.500	0.0	0.0	0.0	0.438
39	-63.250	-8.875	574.200	0.0	0.0	0.0	0.0	0.723
40	-114.080	-8.875	574.200	0.0	0.0	0.0	0.0	0.723
41	-50.100	178.500	460.700	99.500	0.0	0.0	0.0	0.447
42	-100.930	178.500	460.700	99.500	0.0	0.0	0.0	0.447
43	-50.100	-7.100	460.700	0.0	0.0	0.0	0.0	0.732
44	-100.930	-7.100	460.700	0.0	0.0	0.0	0.0	0.732
45	-143.300	178.500	460.700	99.500	0.0	0.0	0.0	0.271
46	-194.130	178.500	460.700	99.500	0.0	0.0	0.0	0.271
47	-143.300	-7.100	460.700	0.0	0.0	0.0	0.0	0.556
48	-194.130	-7.100	460.700	0.0	0.0	0.0	0.0	0.556

Sheet 11 of Table 3C-1 Elements 85, 90, 95, 99, 102

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 85

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-27.600	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	186.000	0.0	99.500	0.0	0.0	0.0	-0.143
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	-3.900	438.000	0.0	0.0	0.0	0.0	0.480
6	8.000	-80.000	25.800	0.0	0.0	0.0	0.0	0.030
7	8.000	-90.000	35.000	0.0	0.0	0.0	0.0	0.040
8	18.400	0.0	0.0	0.0	0.0	0.0	0.0	0.050

COMBINED STRESSES - ELEMENT NO. 85

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-326.600	186.000	0.0	99.500	0.0	0.0	0.0	-0.143
2	-377.429	186.000	0.0	99.500	0.0	0.0	0.0	-0.143
3	-326.600	0.0	0.0	0.0	0.0	0.0	0.0	0.143
4	-377.429	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	-289.800	186.000	0.0	99.500	0.0	0.0	0.0	-0.043
6	-340.629	186.000	0.0	99.500	0.0	0.0	0.0	-0.043
7	-289.800	0.0	0.0	0.0	0.0	0.0	0.0	0.243
8	-340.629	0.0	0.0	0.0	0.0	0.0	0.0	0.243
9	-363.400	186.000	0.0	99.500	0.0	0.0	0.0	-0.243
10	-414.229	186.000	0.0	99.500	0.0	0.0	0.0	-0.243
11	-363.400	0.0	0.0	0.0	0.0	0.0	0.0	0.043
12	-414.229	0.0	0.0	0.0	0.0	0.0	0.0	0.043
13	-65.550	181.515	503.700	99.500	0.0	0.0	0.0	0.409
14	-116.380	181.515	503.700	99.500	0.0	0.0	0.0	0.409
15	-65.550	-4.485	503.700	0.0	0.0	0.0	0.0	0.695
16	-116.380	-4.485	503.700	0.0	0.0	0.0	0.0	0.695
17	-91.600	102.100	463.800	99.500	0.0	0.0	0.0	0.367
18	-142.429	102.100	463.800	99.500	0.0	0.0	0.0	0.367
19	-91.600	-83.900	463.800	0.0	0.0	0.0	0.0	0.653
20	-142.429	-83.900	463.800	0.0	0.0	0.0	0.0	0.653
21	-76.880	102.100	463.800	99.500	0.0	0.0	0.0	0.407
22	-127.709	102.100	463.800	99.500	0.0	0.0	0.0	0.407
23	-76.880	-83.900	463.800	0.0	0.0	0.0	0.0	0.693
24	-127.709	-83.900	463.800	0.0	0.0	0.0	0.0	0.693
25	-106.320	102.100	463.800	99.500	0.0	0.0	0.0	0.327
26	-157.149	102.100	463.800	99.500	0.0	0.0	0.0	0.327
27	-106.320	-83.900	463.800	0.0	0.0	0.0	0.0	0.613
28	-157.149	-83.900	463.800	0.0	0.0	0.0	0.0	0.613
29	21.900	90.150	692.000	99.500	0.0	0.0	0.0	0.617
30	-28.929	90.150	692.000	99.500	0.0	0.0	0.0	0.617
31	21.900	-95.850	692.000	0.0	0.0	0.0	0.0	0.903
32	-28.929	-95.850	692.000	0.0	0.0	0.0	0.0	0.903
33	-16.450	91.125	582.500	99.500	0.0	0.0	0.0	0.547
34	-67.279	91.125	582.500	99.500	0.0	0.0	0.0	0.547
35	-16.450	-94.875	582.500	0.0	0.0	0.0	0.0	0.833
36	-67.279	-94.875	582.500	0.0	0.0	0.0	0.0	0.833
37	-53.250	91.125	582.500	99.500	0.0	0.0	0.0	0.447
38	-104.079	91.125	582.500	99.500	0.0	0.0	0.0	0.447
39	-53.250	-94.875	582.500	0.0	0.0	0.0	0.0	0.733
40	-104.079	-94.875	582.500	0.0	0.0	0.0	0.0	0.733
41	-54.800	102.100	463.800	99.500	0.0	0.0	0.0	0.467
42	-105.629	102.100	463.800	99.500	0.0	0.0	0.0	0.467
43	-54.800	-83.900	463.800	0.0	0.0	0.0	0.0	0.753
44	-105.629	-83.900	463.800	0.0	0.0	0.0	0.0	0.753
45	-128.400	102.100	463.800	99.500	0.0	0.0	0.0	0.267
46	-179.229	102.100	463.800	99.500	0.0	0.0	0.0	0.267
47	-128.400	-83.900	463.800	0.0	0.0	0.0	0.0	0.553
48	-179.229	-83.900	463.800	0.0	0.0	0.0	0.0	0.553

Sheet 12 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 90

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-25.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	149.100	20.000	99.500	0.0	0.800	0.0	-0.121
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	34.700	428.000	0.0	0.0	-0.400	0.0	0.470
6	8.000	-85.700	54.100	0.0	0.0	0.900	0.0	0.061
7	8.000	-97.600	61.400	0.0	0.0	1.000	0.0	0.069
8	16.100	0.0	0.0	0.0	0.0	0.0	0.0	0.053

COMBINED STRESSES - ELEMENT NO. 90

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-324.000	149.100	20.000	99.500	0.0	0.800	0.0	-0.121
2	-374.830	149.100	20.000	99.500	0.0	0.800	0.0	-0.121
3	-324.000	0.0	0.0	0.0	0.0	0.0	0.0	0.143
4	-374.830	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	-291.800	149.100	20.000	99.500	0.0	0.800	0.0	-0.015
6	-342.629	149.100	20.000	99.500	0.0	0.800	0.0	-0.015
7	-291.800	0.0	0.0	0.0	0.0	0.0	0.0	0.249
8	-342.629	0.0	0.0	0.0	0.0	0.0	0.0	0.249
9	-356.200	149.100	20.000	99.500	0.0	0.800	0.0	-0.227
10	-407.030	149.100	20.000	99.500	0.0	0.800	0.0	-0.227
11	-356.200	0.0	0.0	0.0	0.0	0.0	0.0	0.037
12	-407.030	0.0	0.0	0.0	0.0	0.0	0.0	0.037
13	-62.950	189.005	512.200	99.500	0.0	0.340	0.0	0.419
14	-113.780	189.005	512.200	99.500	0.0	0.340	0.0	0.419
15	-62.950	39.905	492.200	0.0	0.0	-0.460	0.0	0.683
16	-113.780	39.905	492.200	0.0	0.0	-0.460	0.0	0.683
17	-89.000	98.100	502.100	99.500	0.0	1.300	0.0	0.410
18	-139.830	98.100	502.100	99.500	0.0	1.300	0.0	0.410
19	-89.000	-51.000	482.100	0.0	0.0	0.500	0.0	0.674
20	-139.830	-51.000	482.100	0.0	0.0	0.500	0.0	0.674
21	-76.120	98.100	502.100	99.500	0.0	1.300	0.0	0.452
22	-126.950	98.100	502.100	99.500	0.0	1.300	0.0	0.452
23	-76.120	-51.000	482.100	0.0	0.0	0.500	0.0	0.716
24	-126.950	-51.000	482.100	0.0	0.0	0.500	0.0	0.716
25	-101.880	98.100	502.100	99.500	0.0	1.300	0.0	0.368
26	-152.710	98.100	502.100	99.500	0.0	1.300	0.0	0.368
27	-101.880	-51.000	482.100	0.0	0.0	0.500	0.0	0.632
28	-152.710	-51.000	482.100	0.0	0.0	0.500	0.0	0.632
29	24.500	103.550	723.400	99.500	0.0	1.200	0.0	0.653
30	-26.330	103.550	723.400	99.500	0.0	1.200	0.0	0.653
31	24.500	-45.550	703.400	0.0	0.0	0.400	0.0	0.917
32	-26.330	-45.550	703.400	0.0	0.0	0.400	0.0	0.917
33	-16.150	94.875	616.400	99.500	0.0	1.300	0.0	0.588
34	-66.980	94.875	616.400	99.500	0.0	1.300	0.0	0.588
35	-16.150	-54.225	596.400	0.0	0.0	0.500	0.0	0.852
36	-66.980	-54.225	596.400	0.0	0.0	0.500	0.0	0.852
37	-48.350	94.875	616.400	99.500	0.0	1.300	0.0	0.482
38	-99.180	94.875	616.400	99.500	0.0	1.300	0.0	0.482
39	-48.350	-54.225	596.400	0.0	0.0	0.500	0.0	0.746
40	-99.180	-54.225	596.400	0.0	0.0	0.500	0.0	0.746
41	-56.800	98.100	502.100	99.500	0.0	1.300	0.0	0.516
42	-107.630	98.100	502.100	99.500	0.0	1.300	0.0	0.516
43	-56.800	-51.000	482.100	0.0	0.0	0.500	0.0	0.780
44	-107.630	-51.000	482.100	0.0	0.0	0.500	0.0	0.780
45	-121.200	98.100	502.100	99.500	0.0	1.300	0.0	0.304
46	-172.030	98.100	502.100	99.500	0.0	1.300	0.0	0.304
47	-121.200	-51.000	482.100	0.0	0.0	0.500	0.0	0.568
48	-172.030	-51.000	482.100	0.0	0.0	0.500	0.0	0.568

Sheet 13 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 95

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-22.500	20.000	0.0	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	157.300	34.600	99.500	0.0	3.100	0.0	-0.105
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	7.700	354.000	0.0	0.0	-12.800	0.0	0.388
6	8.000	-66.800	102.400	0.0	0.0	6.800	0.0	0.114
7	8.000	-76.100	97.900	0.0	0.0	7.700	0.0	0.109
8	14.000	0.0	0.0	0.0	0.0	0.0	0.0	0.055

COMBINED STRESSES - ELEMENT NO. 95

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-321.500	177.300	34.600	99.500	0.0	3.100	0.0	-0.105
2	-372.330	177.300	34.600	99.500	0.0	3.100	0.0	-0.105
3	-321.500	20.000	0.0	0.0	0.0	0.0	0.0	0.143
4	-372.330	20.000	0.0	0.0	0.0	0.0	0.0	0.143
5	-293.500	177.300	34.600	99.500	0.0	3.100	0.0	0.005
6	-344.330	177.300	34.600	99.500	0.0	3.100	0.0	0.005
7	-293.500	20.000	0.0	0.0	0.0	0.0	0.0	0.253
8	-344.330	20.000	0.0	0.0	0.0	0.0	0.0	0.253
9	-349.500	177.300	34.600	99.500	0.0	3.100	0.0	-0.215
10	-400.330	177.300	34.600	99.500	0.0	3.100	0.0	-0.215
11	-349.500	20.000	0.0	0.0	0.0	0.0	0.0	0.033
12	-400.330	20.000	0.0	0.0	0.0	0.0	0.0	0.033
13	-60.450	186.155	441.700	99.500	0.0	-11.620	0.0	0.341
14	-111.280	186.155	441.700	99.500	0.0	-11.620	0.0	0.341
15	-60.450	28.855	407.100	0.0	0.0	-14.720	0.0	0.589
16	-111.280	28.855	407.100	0.0	0.0	-14.720	0.0	0.589
17	-86.500	118.200	491.000	99.500	0.0	-2.900	0.0	0.397
18	-137.330	118.200	491.000	99.500	0.0	-2.900	0.0	0.397
19	-86.500	-39.100	456.400	0.0	0.0	-6.000	0.0	0.645
20	-137.330	-39.100	456.400	0.0	0.0	-6.000	0.0	0.645
21	-75.300	118.200	491.000	99.500	0.0	-2.900	0.0	0.441
22	-126.130	118.200	491.000	99.500	0.0	-2.900	0.0	0.441
23	-75.300	-39.100	456.400	0.0	0.0	-6.000	0.0	0.689
24	-126.130	-39.100	456.400	0.0	0.0	-6.000	0.0	0.689
25	-97.700	118.200	491.000	99.500	0.0	-2.900	0.0	0.353
26	-148.530	118.200	491.000	99.500	0.0	-2.900	0.0	0.353
27	-97.700	-39.100	456.400	0.0	0.0	-6.000	0.0	0.601
28	-148.530	-39.100	456.400	0.0	0.0	-6.000	0.0	0.601
29	27.000	112.750	663.500	99.500	0.0	-8.400	0.0	0.586
30	-23.830	112.750	663.500	99.500	0.0	-8.400	0.0	0.586
31	27.000	-44.550	628.900	0.0	0.0	-11.500	0.0	0.834
32	-23.830	-44.550	628.900	0.0	0.0	-11.500	0.0	0.834
33	-15.750	110.825	575.000	99.500	0.0	-5.200	0.0	0.544
34	-66.580	110.825	575.000	99.500	0.0	-5.200	0.0	0.544
35	-15.750	-46.475	540.400	0.0	0.0	-8.300	0.0	0.792
36	-66.580	-46.475	540.400	0.0	0.0	-8.300	0.0	0.792
37	-43.750	110.825	575.000	99.500	0.0	-5.200	0.0	0.434
38	-94.580	110.825	575.000	99.500	0.0	-5.200	0.0	0.434
39	-43.750	-46.475	540.400	0.0	0.0	-8.300	0.0	0.682
40	-94.580	-46.475	540.400	0.0	0.0	-8.300	0.0	0.682
41	-58.500	118.200	491.000	99.500	0.0	-2.900	0.0	0.507
42	-109.330	118.200	491.000	99.500	0.0	-2.900	0.0	0.507
43	-58.500	-39.100	456.400	0.0	0.0	-6.000	0.0	0.755
44	-109.330	-39.100	456.400	0.0	0.0	-6.000	0.0	0.755
45	-114.500	118.200	491.000	99.500	0.0	-2.900	0.0	0.287
46	-165.330	118.200	491.000	99.500	0.0	-2.900	0.0	0.287
47	-114.500	-39.100	456.400	0.0	0.0	-6.000	0.0	0.535
48	-165.330	-39.100	456.400	0.0	0.0	-6.000	0.0	0.535

Sheet 14 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 99

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-20.400	27.800	20.400	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	173.800	48.300	99.500	0.0	5.900	0.0	-0.090
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	-60.500	322.000	0.0	0.0	-21.600	0.0	0.353
6	8.000	-28.400	120.700	0.0	0.0	13.700	0.0	0.134
7	8.000	-32.300	134.500	0.0	0.0	15.600	0.0	0.149
8	12.300	0.0	0.0	0.0	0.0	0.0	0.0	0.058

COMBINED STRESSES - ELEMENT NO. 99

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-319.400	201.600	68.700	99.500	0.0	5.900	0.0	-0.090
2	-370.229	201.600	68.700	99.500	0.0	5.900	0.0	-0.090
3	-319.400	27.800	20.400	0.0	0.0	0.0	0.0	0.143
4	-370.229	27.800	20.400	0.0	0.0	0.0	0.0	0.143
5	-294.800	201.600	68.700	99.500	0.0	5.900	0.0	0.026
6	-345.629	201.600	68.700	99.500	0.0	5.900	0.0	0.026
7	-294.800	27.800	20.400	0.0	0.0	0.0	0.0	0.259
8	-345.629	27.800	20.400	0.0	0.0	0.0	0.0	0.259
9	-344.000	201.600	68.700	99.500	0.0	5.900	0.0	-0.206
10	-394.829	201.600	68.700	99.500	0.0	5.900	0.0	-0.206
11	-344.000	27.800	20.400	0.0	0.0	0.0	0.0	0.027
12	-394.829	27.800	20.400	0.0	0.0	0.0	0.0	0.027
13	-58.350	132.025	439.000	99.500	0.0	-18.940	0.0	0.316
14	-109.180	132.025	439.000	99.500	0.0	-18.940	0.0	0.316
15	-58.350	-41.775	390.700	0.0	0.0	-24.840	0.0	0.549
16	-109.180	-41.775	390.700	0.0	0.0	-24.840	0.0	0.549
17	-84.400	112.700	511.400	99.500	0.0	-2.000	0.0	0.397
18	-135.229	112.700	511.400	99.500	0.0	-2.000	0.0	0.397
19	-84.400	-61.100	463.100	0.0	0.0	-7.900	0.0	0.630
20	-135.229	-61.100	463.100	0.0	0.0	-7.900	0.0	0.630
21	-74.560	112.700	511.400	99.500	0.0	-2.000	0.0	0.443
22	-125.389	112.700	511.400	99.500	0.0	-2.000	0.0	0.443
23	-74.560	-61.100	463.100	0.0	0.0	-7.900	0.0	0.676
24	-125.389	-61.100	463.100	0.0	0.0	-7.900	0.0	0.676
25	-94.240	112.700	511.400	99.500	0.0	-2.000	0.0	0.351
26	-145.069	112.700	511.400	99.500	0.0	-2.000	0.0	0.351
27	-94.240	-61.100	463.100	0.0	0.0	-7.900	0.0	0.584
28	-145.069	-61.100	463.100	0.0	0.0	-7.900	0.0	0.584
29	29.100	78.550	686.200	99.500	0.0	-10.900	0.0	0.588
30	-21.729	78.550	686.200	99.500	0.0	-10.900	0.0	0.588
31	29.100	-95.250	637.900	0.0	0.0	-16.800	0.0	0.821
32	-21.729	-95.250	637.900	0.0	0.0	-16.800	0.0	0.821
33	-15.350	93.675	605.700	99.500	0.0	-5.500	0.0	0.558
34	-66.179	93.675	605.700	99.500	0.0	-5.500	0.0	0.558
35	-15.350	-80.125	557.400	0.0	0.0	-11.400	0.0	0.791
36	-66.179	-80.125	557.400	0.0	0.0	-11.400	0.0	0.791
37	-39.950	93.675	605.700	99.500	0.0	-5.500	0.0	0.442
38	-90.779	93.675	605.700	99.500	0.0	-5.500	0.0	0.442
39	-39.950	-80.125	557.400	0.0	0.0	-11.400	0.0	0.675
40	-90.779	-80.125	557.400	0.0	0.0	-11.400	0.0	0.675
41	-59.800	112.700	511.400	99.500	0.0	-2.000	0.0	0.513
42	-110.630	112.700	511.400	99.500	0.0	-2.000	0.0	0.513
43	-59.800	-61.100	463.100	0.0	0.0	-7.900	0.0	0.746
44	-110.630	-61.100	463.100	0.0	0.0	-7.900	0.0	0.746
45	-109.000	112.700	511.400	99.500	0.0	-2.000	0.0	0.281
46	-159.829	112.700	511.400	99.500	0.0	-2.000	0.0	0.281
47	-109.000	-61.100	463.100	0.0	0.0	-7.900	0.0	0.514
48	-159.829	-61.100	463.100	0.0	0.0	-7.900	0.0	0.514

Sheet 15 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 102

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-19.400	31.000	18.300	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.100	-24.800	28.200	0.0	-1.000	0.0	-0.093
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	-126.700	210.000	0.0	0.0	-18.200	0.0	0.346
6	8.000	-0.300	54.000	0.0	0.0	-5.600	0.0	0.179
7	8.000	-0.300	59.800	0.0	0.0	-6.400	0.0	0.200
8	11.200	0.0	0.0	0.0	0.0	0.0	0.0	0.059

COMBINED STRESSES - ELEMENT NO. 102

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-318.400	59.100	-6.500	28.200	0.0	-1.000	0.0	-0.093
2	-369.229	59.100	-6.500	28.200	0.0	-1.000	0.0	-0.093
3	-318.400	31.000	18.300	0.0	0.0	0.0	0.0	0.143
4	-369.229	31.000	18.300	0.0	0.0	0.0	0.0	0.143
5	-296.000	59.100	-6.500	28.200	0.0	-1.000	0.0	0.025
6	-346.829	59.100	-6.500	28.200	0.0	-1.000	0.0	0.025
7	-296.000	31.000	18.300	0.0	0.0	0.0	0.0	0.261
8	-346.829	31.000	18.300	0.0	0.0	0.0	0.0	0.261
9	-340.800	59.100	-6.500	28.200	0.0	-1.000	0.0	-0.211
10	-391.629	59.100	-6.500	28.200	0.0	-1.000	0.0	-0.211
11	-340.800	31.000	18.300	0.0	0.0	0.0	0.0	0.025
12	-391.629	31.000	18.300	0.0	0.0	0.0	0.0	0.025
13	-57.350	-86.605	235.000	28.200	0.0	-21.930	0.0	0.305
14	-108.180	-86.605	235.000	28.200	0.0	-21.930	0.0	0.305
15	-57.350	-114.705	259.800	0.0	0.0	-20.930	0.0	0.541
16	-108.180	-114.705	259.800	0.0	0.0	-20.930	0.0	0.541
17	-83.400	-67.900	257.500	28.200	0.0	-24.800	0.0	0.432
18	-134.229	-67.900	257.500	28.200	0.0	-24.800	0.0	0.432
19	-83.400	-96.000	282.300	0.0	0.0	-23.800	0.0	0.668
20	-134.229	-96.000	282.300	0.0	0.0	-23.800	0.0	0.668
21	-74.440	-67.900	257.500	28.200	0.0	-24.800	0.0	0.479
22	-125.269	-67.900	257.500	28.200	0.0	-24.800	0.0	0.479
23	-74.440	-96.000	282.300	0.0	0.0	-23.800	0.0	0.715
24	-125.269	-96.000	282.300	0.0	0.0	-23.800	0.0	0.715
25	-92.360	-67.900	257.500	28.200	0.0	-24.800	0.0	0.385
26	-143.189	-67.900	257.500	28.200	0.0	-24.800	0.0	0.385
27	-92.360	-96.000	282.300	0.0	0.0	-23.800	0.0	0.621
28	-143.189	-96.000	282.300	0.0	0.0	-23.800	0.0	0.621
29	30.100	-131.250	368.300	28.200	0.0	-34.700	0.0	0.626
30	-20.729	-131.250	368.300	28.200	0.0	-34.700	0.0	0.626
31	30.100	-159.350	393.100	0.0	0.0	-33.700	0.0	0.862
32	-20.729	-159.350	393.100	0.0	0.0	-33.700	0.0	0.862
33	-15.450	-99.575	315.800	28.200	0.0.	-30.150	0.0	0.598
34	-66.279	-99.575	315.800	28.200	0.0	-30.150	0.0	0.598
35	-15.450	-127.675	340.600	0.0	0.0	-29.150	0.0	0.834
36	-66.279	-127.675	340.600	0.0	0.0	-29.150	0.0	0.834
37	-37.850	-99.575	315.800	28.200	0.0	-30.150	0.0	0.480
38	-88.679	-99.575	315.800	28.200	0.0	-30.150	0.0	0.480
39	-37.850	-127.675	340.600	0.0	0.0	-29.150	0.0	0.716
40	-88.679	-127.675	340.600	0.0	0.0	-29.150	0.0	0.716
41	-61.000	-67.900	257.500	28.200	0.0	-24.800	0.0	0.550
42	-111.829	-67.900	257.500	28.200	0.0	-24.800	0.0	0.550
43	-61.000	-96.000	282.300	0.0	0.0	-23.800	0.0	0.786
44	-111.829	-96.000	282.300	0.0	0.0	-23.800	0.0	0.786
45	-105.800	-67.900	257.500	28.200	0.0	-24.800	0.0	0.314
46	-156.629	-67.900	257.500	28.200	0.0	-24.800	0.0	0.314
47	-105.800	-96.000	282.300	0.0	0.0	-23.800	0.0	0.550
48	-156.629	-96.000	282.300	0.0	0.0	-23.800	0.0	0.550

Sheet 16 of Table 3C-1 Elements 105, 108, 111 (3 pages)

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 105

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-18.500	32.300	16.200	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	31.900	-12.100	28.200	0.0	1.000	0.0	-0.072
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	0.0	-199.100	182.000	0.0	0.0	-25.000	0.0	0.301
6	8.000	8.700	84.400	0.0	0.0	-1.000	0.0	0.229
7	8.000	10.000	95.600	0.0	0.0	-1.100	0.0	0.259
8	10.000	0.0	0.0	0.0	0.0	0.0	0.0	0.062

COMBINED STRESSES - ELEMENT NO. 105

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-317.500	64.200	4.100	28.200	0.0	1.000	0.0	-0.072
2	-368.330	64.200	4.100	28.200	0.0	1.000	0.0	-0.072
3	-317.500	32.300	16.200	0.0	0.0	0.0	0.0	0.143
4	-368.330	32.300	16.200	0.0	0.0	0.0	0.0	0.143
5	-297.500	64.200	4.100	28.200	0.0	1.000	0.0	0.052
6	-348.330	64.200	4.100	28.200	0.0	1.000	0.0	0.052
7	-297.500	32.300	16.200	0.0	0.0	0.0	0.0	0.267
8	-348.330	32.300	16.200	0.0	0.0	0.0	0.0	0.267
9	-337.500	64.200	4.100	28.200	0.0	1.000	0.0	-0.196
10	-388.330	64.200	4.100	28.200	0.0	1.000	0.0	-0.196
11	-337.500	32.300	16.200	0.0	0.0	0.0	0.0	0.019
12	-388.330	32.300	16.200	0.0	0.0	0.0	0.0	0.019
13	-317.500	-164.765	213.400	28.200	0.0	-27.750	0.0	0.274
14	-368.330	-164.765	213.400	28.200	0.0	-27.750	0.0	0.274
15	-317.500	-196.665	225.500	0.0	0.0	-28.750	0.0	0.489
16	-368.330	-196.665	225.500	0.0	0.0	-28.750	0.0	0.489
17	-309.500	-126.200	270.500	28.200	0.0	-25.000	0.0	0.458
18	-360.330	-126.200	270.500	28.200	0.0	-25.000	0.0	0.458
19	-309.500	-158.100	282.600	0.0	0.0	-26.000	0.0	0.673
20	-360.330	-158.100	282.600	0.0	0.0	-26.000	0.0	0.673
21	-301.500	-126.200	270.500	28.200	0.0	-25.000	0.0	0.508
22	-352.330	-126.200	270.500	28.200	0.0	-25.000	0.0	0.508
23	-301.500	-158.100	282.600	0.0	0.0	-26.000	0.0	0.723
24	-352.330	-158.100	282.600	0.0	0.0	-26.000	0.0	0.723
25	-317.500	-126.200	270.500	28.200	0.0	-25.000	0.0	0.408
26	-368.329	-126.200	270.500	28.200	0.0	-25.000	0.0	0.408
27	-317.500	-158.100	282.600	0.0	0.0	-26.000	0.0	0.623
28	-368.329	-158.100	282.600	0.0	0.0	-26.000	0.0	0.623
29	-309.500	-224.450	372.700	28.200	0.0	-37.600	0.0	0.638
30	-360.330	-224.450	372.700	28.200	0.0	-37.600	0.0	0.638
31	-309.500	-256.350	384.800	0.0	0.0	-38.600	0.0	0.853
32	-360.330	-256.350	384.800	0.0	0.0	-38.600	0.0	0.853
33	-299.500	-174.675	327.200	28.200	0.0	-31.350	0.0	0.625
34	-350.330	-174.675	327.200	28.200	0.0	-31.350	0.0	0.625
35	-299.500	-206.575	339.300	0.0	0.0	-32.350	0.0	0.840
36	-350.330	-206.575	339.300	0.0	0.0	-32.350	0.0	0.840
37	-319.500	-174.675	327.200	28.200	0.0	-31.350	0.0	0.501
38	-370.330	-174.675	327.200	28.200	0.0	-31.350	0.0	0.501
39	-319.500	-206.575	339.300	0.0	0.0	-32.350	0.0	0.716
40	-370.330	-206.575	339.300	0.0	0.0	-32.350	0.0	0.716
41	-289.500	-126.200	270.500	28.200	0.0	-25.000	0.0	0.582
42	-340.330	-126.200	270.500	28.200	0.0	-25.000	0.0	0.582
43	-289.500	-158.100	282.600	0.0	0.0	-26.000	0.0	0.797
44	-340.330	-158.100	282.600	0.0	0.0	-26.000	0.0	0.797
45	-329.500	-126.200	270.500	28.200	0.0	-25.000	0.0	0.334
46	-380.330	-126.200	270.500	28.200	0.0	-25.000	0.0	0.334
47	-329.500	-158.100	282.600	0.0	0.0	-26.000	0.0	0.549
48	-380.330	-158.100	282.600	0.0	0.0	-26.000	0.0	0.549

Sheet 17 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 108

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-17.500	31.000	14.100	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	31.800	-3.700	28.200	0.0	1.200	0.0	-0.058
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	0.0	19.800	229.000	0.0	0.0	3.100	0.0	0.368
6	8.000	8.200	103.700	0.0	0.0	0.900	0.0	0.261
7	8.000	9.800	119.900	0.0	0.0	1.000	0.0	0.299
8	9.100	0.0	0.0	0.0	0.0	0.0	0.0	0.062

COMBINED STRESSES - ELEMENT NO. 108

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-316.500	62.800	10.400	28.200	0.0	1.200	0.0	-0.058
2	-367.330	62.800	10.400	28.200	0.0	1.200	0.0	-0.058
3	-316.500	31.000	14.100	0.0	0.0	0.0	0.0	0.143
4	-367.330	31.000	14.100	0.0	0.0	0.0	0.0	0.143
5	-298.300	62.800	10.400	28.200	0.0	1.200	0.0	0.066
6	-349.129	62.800	10.400	28.200	0.0	1.200	0.0	0.066
7	-298.300	31.000	14.100	0.0	0.0	0.0	0.0	0.267
8	-349.129	31.000	14.100	0.0	0.0	0.0	0.0	0.267
9	-334.700	62.800	10.400	28.200	0.0	1.200	0.0	-0.182
10	-385.530	62.800	10.400	28.200	0.0	1.200	0.0	-0.182
11	-334.700	31.000	14.100	0.0	0.0	0.0	0.0	0.019
12	-385.530	31.000	14.100	0.0	0.0	0.0	0.0	0.019
13	-316.500	85.570	273.750	28.200	0.0	4.765	0.0	0.365
14	-367.330	85.570	273.750	28.200	0.0	4.765	0.0	0.365
15	-316.500	53.770	277.450	0.0	0.0	3.565	0.0	0.566
16	-367.330	53.770	277.450	0.0	0.0	3.565	0.0	0.566
17	-308.500	90.800	343.100	28.200	0.0	5.200	0.0	0.571
18	-359.330	90.800	343.100	28.200	0.0	5.200	0.0	0.571
19	-308.500	59.000	346.800	0.0	0.0	4.000	0.0	0.772
20	-359.330	59.000	346.800	0.0	0.0	4.000	0.0	0.772
21	-301.220	90.800	343.100	28.200	0.0	5.200	0.0	0.621
22	-352.050	90.800	343.100	28.200	0.0	5.200	0.0	0.621
23	-301.220	59.000	346.800	0.0	0.0	4.000	0.0	0.822
24	-352.050	59.000	346.800	0.0	0.0	4.000	0.0	0.822
25	-315.780	90.800	343.100	28.200	0.0	5.200	0.0	0.521
26	-366.609	90.800	343.100	28.200	0.0	5.200	0.0	0.521
27	-315.780	59.000	346.800	0.0	0.0	4.000	0.0	0.722
28	-366.609	59.000	346.800	0.0	0.0	4.000	0.0	0.722
29	-308.500	102.300	473.800	28.200	0.0	6.850	0.0	0.793
30	-359.330	102.300	473.800	28.200	0.0	6.850	0.0	0.793
31	-308.500	70.500	477.500	0.0	0.0	5.650	0.0	0.994
32	-359.330	70.500	477.500	0.0	0.0	5.650	0.0	0.994
33	-299.400	97.350	416.550	28.200	0.0	6.075	0.0	0.763
34	-350.229	97.350	416.550	28.200	0.0	6.075	0.0	0.763
35	-299.400	65.550	420.250	0.0	0.0	4.875	0.0	0.964
36	-350.229	65.550	420.250	0.0	0.0	4.875	0.0	0.964
37	-317.600	97.350	416.550	28.200	0.0	6.075	0.0	0.639
38	-368.429	97.350	416.550	28.200	0.0	6.075	0.0	0.639
39	-317.600	65.550	420.250	0.0	0.0	4.875	0.0	0.840
40	-368.429	65.550	420.250	0.0	0.0	4.875	0.0	0.840
41	-290.300	90.800	343.100	28.200	0.0	5.200	0.0	0.695
42	-341.129	90.800	343.100	28.200	0.0	5.200	0.0	0.695
43	-290.300	59.000	346.800	0.0	0.0	4.000	0.0	0.896
44	-341.129	59.000	346.800	0.0	0.0	4.000	0.0	0.896
45	-326.700	90.800	343.100	28.200	0.0	5.200	0.0	0.447
46	-377.530	90.800	343.100	28.200	0.0	5.200	0.0	0.447
47	-326.700	59.000	346.800	0.0	0.0	4.000	0.0	0.648
48	-377.530	59.000	346.800	0.0	0.0	4.000	0.0	0.648

Sheet 18 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. -111

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.800	26.500	12.200	0.0	0.0	0.0	0.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	0.0	10.300	243.000	0.0	0.0	3.300	0.0	0.402
6	111.000	5.000	111.000	0.0	0.0	1.100	0.0	0.273
7	126.000	5.700	126.000	0.0	0.0	1.300	0.0	0.309
8	8.200	0.0	0.0	0.0	0.0	0.0	0.0	0.063

COMBINED STRESSES - ELEMENT NO. -111

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-315.800	54.700	12.200	28.200	0.0	0.0	0.0	-0.052
2	-366.629	54.700	12.200	28.200	0.0	0.0	0.0	-0.052
3	-315.800	26.500	12.200	0.0	0.0	0.0	0.0	0.143
4	-366.629	26.500	12.200	0.0	0.0	0.0	0.0	0.143
5	-299.400	54.700	12.200	28.200	0.0	0.0	0.0	0.074
6	-350.229	54.700	12.200	28.200	0.0	0.0	0.0	0.074
7	-299.400	26.500	12.200	0.0	0.0	0.0	0.0	0.269
8	-350.229	26.500	12.200	0.0	0.0	0.0	0.0	0.269
9	-332.200	54.700	12.200	28.200	0.0	0.0	0.0	-0.178
10	-383.029	54.700	12.200	28.200	0.0	0.0	0.0	-0.178
11	-332.200	26.500	12.200	0.0	0.0	0.0	0.0	0.017
12	-383.029	26.500	12.200	0.0	0.0	0.0	0.0	0.017
13	-315.800	66.545	291.650	28.200	0.0	3.795	0.0	0.410
14	-366.629	66.545	291.650	28.200	0.0	3.795	0.0	0.410
15	-315.800	38.345	291.650	0.0	0.0	3.795	0.0	0.605
16	-366.629	38.345	291.650	0.0	0.0	3.795	0.0	0.605
17	-204.800	70.000	366.200	28.200	0.0	4.400	0.0	0.623
18	-255.629	70.000	366.200	28.200	0.0	4.400	0.0	0.623
19	-204.800	41.800	366.200	0.0	0.0	4.400	0.0	0.818
20	-255.629	41.800	366.200	0.0	0.0	4.400	0.0	0.818
21	-198.240	70.000	366.200	28.200	0.0	4.400	0.0	0.673
22	-249.069	70.000	366.200	28.200	0.0	4.400	0.0	0.673
23	-198.240	41.800	366.200	0.0	0.0	4.400	0.0	0.868
24	-249.069	41.800	366.200	0.0	0.0	4.400	0.0	0.868
25	-211.360	70.000	366.200	28.200	0.0	4.400	0.0	0.573
26	-262.189	70.000	366.200	28.200	0.0	4.400	0.0	0.573
27	-211.360	41.800	366.200	0.0	0.0	4.400	0.0	0.768
28	-262.189	41.800	366.200	0.0	0.0	4.400	0.0	0.768
29	-189.800	75.850	502.700	28.200	0.0	6.250	0.0	0.860
30	-240.629	75.850	502.700	28.200	0.0	6.250	0.0	0.860
31	-189.800	47.650	502.700	0.0	0.0	6.250	0.0	1.055
32	-240.629	47.650	502.700	0.0	0.0	6.250	0.0	1.055
33	-181.600	73.275	441.950	28.200	0.0	5.425	0.0	0.822
34	-232.429	73.275	441.950	28.200	0.0	5.425	0.0	0.822
35	-181.600	45.075	441.950	0.0	0.0	5.425	0.0	1.017
36	-232.429	45.075	441.950	0.0	0.0	5.425	0.0	1.017
37	-198.000	73.275	441.950	28.200	0.0	5.425	0.0	0.696
38	-248.829	73.275	441.950	28.200	0.0	5.425	0.0	0.696
39	-198.000	45.075	441.950	0.0	0.0	5.425	0.0	0.891
40	-248.829	45.075	441.950	0.0	0.0	5.425	0.0	0.891
41	-188.400	70.000	366.200	28.200	0.0	4.400	0.0	0.749
42	-239.229	70.000	366.200	28.200	0.0	4.400	0.0	0.749
43	-188.400	41.800	366.200	0.0	0.0	4.400	0.0	0.944
44	-239.229	41.800	366.200	0.0	0.0	4.400	0.0	0.944
45	-221.200	70.000	366.200	28.200	0.0	4.400	0.0	0.497
46	-272.029	70.000	366.200	28.200	0.0	4.400	0.0	0.497
47	-221.200	41.800	366.200	0.0	0.0	4.400	0.0	0.692
48	-272.029	41.800	366.200	0.0	0.0	4.400	0.0	0.692

Sheet 19 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 111

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.800	28.000	12.200	0.0	0.0	0.0	.0	0.0
2	-299.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	10.300	243.000	0.0	0.0	3.300	0.0	0.402
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	8.200	0.0	0.0	0.0	0.0	0.0	0.0	0.063

COMBINED STRESSES - ELEMENT NO. 111

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-315.800	56.200	12.200	28.200	0.0	0.0	0.0	-0.052
2	-366.629	56.200	12.200	28.200	0.0	0.0	0.0	-0.052
3	-315.800	28.000	12.200	0.0	0.0	0.0	0.0	0.143
4	-366.629	28.000	12.200	0.0	0.0	0.0	0.0	0.143
5	-299.400	56.200	12.200	28.200	0.0	0.0	0.0	0.074
6	-350.229	56.200	12.200	28.200	0.0	0.0	0.0	0.074
7	-299.400	28.000	12.200	0.0	0.0	0.0	0.0	0.269
8	-350.229	28.000	12.200	0.0	0.0	0.0	0.0	0.269
9	-332.200	56.200	12.200	28.200	0.0	0.0	0.0	-0.178
10	-383.029	56.200	12.200	28.200	0.0	0.0	0.0	-0.178
11	-332.200	28.000	12.200	0.0	0.0	0.0	0.0	0.017
12	-383.029	28.000	12.200	0.0	0.0	0.0	0.0	0.017
13	-54.750	68.045	291.650	28.200	0.0	3.795	0.0	0.410
14	-105.580	68.045	291.650	28.200	0.0	3.795	0.0	0.410
15	-54.750	39.845	291.650	0.0	0.0	3.795	0.0	0.605
16	-105.580	39.845	291.650	0.0	0.0	3.795	0.0	0.605
17	22.200	66.500	366.200	28.200	0.0	3.300	0.0	0.623
18	-28.629	66.500	366.200	28.200	0.0	3.300	0.0	0.623
19	22.200	38.300	366.200	0.0	0.0	3.300	0.0	0.818
20	-28.629	38.300	366.200	0.0	0.0	3.300	[illegible]	0.818
21	28.760	66.500	366.200	28.200	0.0	3.300	0.0	0.673
22	-22.069	66.500	366.200	28.200	0.0	3.300	0.0	0.673
23	28.760	38.300	366.200	0.0	0.0	3.300	0.0	0.868
24	-22.069	38.300	366.200	0.0	0.0	3.300	0.0	0.868
25	15.640	66.500	366.200	28.200	0.0	3.300	0.0	0.573
26	-35.189	66.500	366.200	28.200	0.0	3.300	0.0	0.573
27	15.640	38.300	366.200	0.0	0.0	3.300	0.0	0.768
28	-35.189	38.300	366.200	0.0	0.0	3.300	0.0	0.768
29	150.700	71.650	502.700	28.200	0.0	4.950	0.0	0.860
30	99.871	71.650	502.700	28.200	0.0	4.950	0.0	0.860
31	150.700	43.450	502.700	0.0	0.0	4.950	0.0	1.055
32	99.871	43.450	502.700	0.0	0.0	4.950	0.0	1.055
33	102.150	69.075	441.950	28.200	0.0	4.125	0.0	0.822
34	51.321	69.075	441.950	28.200	0.0	4.125	0.0	0.822
35	102.150	40.875	441.950	0.0	0.0	4.125	0.0	1.017
36	51.321	40.875	441.950	0.0	0.0	4.125	0.0	1.017
37	85.750	69.075	441.950	28.200	0.0	4.125	0.0	0.696
38	34.921	69.075	441.950	28.200	0.0	4.125	0.0	0.696
39	85.750	40.875	441.950	0.0	0.0	4.125	0.0	0.891
40	34.921	40.875	441.950	0.0	0.0	4.125	0.0	0.891
41	38.600	66.500	366.200	28.200	0.0	3.300	0.0	0.749
42	-12.229	66.500	366.200	28.200	0.0	3.300	0.0	0.749
43	38.600	38.300	366.200	0.0	0.0	3.300	0.0	0.944
44	-12.229	38.300	366.200	0.0	0.0	3.300	0.0	0.944
45	5.800	66.500	366.200	28.200	0.0	3.300	0.0	0.497
46	-45.029	66.500	366.200	28.200	0.0	3.300	0.0	0.497
47	5.800	38.300	366.200	0.0	0.0	3.300	0.0	0.692
48	-45.029	38.300	366.200	0.0	0.0	3.300	0.0	0.692

Sheet 20 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 111

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.800	-1.500	12.200	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	10.300	243.000	0.0	0.0	3.300	0.0	0.402
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	8.200	0.0	0.0	0.0	0.0	0.0	0.0	0.063

COMBINED STRESSES - ELEMENT NO. 111

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.800	26.700	12.200	28.200	0.0	0.0	0.0	-0.052
2	-16.800	26.700	12.200	28.200	0.0	0.0	0.0	-0.052
3	-16.800	-1.500	12.200	0.0	0.0	0.0	0.0	0.143
4	-16.800	-1.500	12.200	0.0	0.0	0.0	0.0	0.143
5	-0.400	26.700	12.200	28.200	0.0	0.0	0.0	0.074
6	-0.400	26.700	12.200	28.200	0.0	0.0	0.0	0.074
7	-0.400	-1.500	12.200	0.0	0.0	0.0	0.0	0.269
8	-0.400	-1.500	12.200	0.0	0.0	0.0	0.0	0.269
9	-33.200	26.700	12.200	28.200	0.0	0.0	0.0	-0.178
10	-33.200	26.700	12.200	28.200	0.0	0.0	0.0	-0.178
11	-33.200	-1.500	12.200	0.0	0.0	0.0	0.0	0.017
12	-33.200	-1.500	12.200	0.0	0.0	0.0	0.0	0.017
13	244.250	38.545	291.650	28.200	0.0	3.795	0.0	0.410
14	244.250	38.545	291.650	28.200	0.0	3.795	0.0	0.410
15	244.250	10.345	291.650	0.0	0.0	3.795	0.0	0.605
16	244.250	10.345	291.650	0.0	0.0	3.795	0.0	0.605
17	321.200	37.000	366.200	28.200	0.0	3.300	0.0	0.623
18	321.200	37.000	366.200	28.200	0.0	3.300	0.0	0.623
19	321.200	8.800	366.200	0.0	0.0	3.300	0.0	0.818
20	321.200	8.800	366.200	0.0	0.0	3.300	0.0	0.818
21	327.760	37.000	366.200	28.200	0.0	3.300	0.0	0.673
22	327.760	37.000	366.200	28.200	0.0	3.300	0.0	0.673
23	327.760	8.800	366.200	0.0	0.0	3.300	0.0	0.868
24	327.760	8.800	366.200	0.0	0.0	3.300	0.0	0.868
25	314.640	37.000	366.200	28.200	0.0	3.300	0.0	0.573
26	314.640	37.000	366.200	28.200	0.0	3.300	0.0	0.573
27	314.640	8.800	366.200	0.0	0.0	3.300	0.0	0.768
28	314.640	8.800	366.200	0.0	0.0	3.300	0.0	0.768
29	449.700	42.150	502.700	28.200	0.0	4.950	0.0	0.860
30	449.700	42.150	502.700	28.200	0.0	4.950	0.0	0.860
31	449.700	13.950	502.700	0.0	0.0	4.950	0.0	1.055
32	449.700	13.950	502.700	0.0	0.0	4.950	0.0	1.055
33	401.150	39.575	441.950	28.200	0.0	4.125	0.0	0.822
34	401.150	39.575	441.950	28.200	0.0	4.125	0.0	0.822
35	401.150	11.375	441.950	0.0	0.0	4.125	0.0	1.017
36	401.150	11.375	441.950	0.0	0.0	4.125	0.0	1.017
37	384.750	39.575	441.950	28.200	0.0	4.125	0.0	0.696
38	384.750	39.575	441.950	28.200	0.0	4.125	0.0	0.696
39	384.750	11.375	441.950	0.0	0.0	4.125	0.0	0.891
40	384.750	11.375	441.950	0.0	0.0	4.125	0.0	0.891
41	337.600	37.000	366.200	28.200	0.0	3.300	0.0	0.749
42	337.600	37.000	366.200	28.200	0.0	3.300	0.0	0.749
43	337.600	8.800	366.200	0.0	0.0	3.300	0.0	0.944
44	337.600	8.800	366.200	0.0	0.0	3.300	0.0	0.944
45	304.800	37.000	366.200	28.200	0.0	3.300	0.0	0.497
46	304.800	37.000	366.200	28.200	0.0	3.300	0.0	0.497
47	304.800	8.800	366.200	0.0	0.0	3.300	0.0	0.692
48	304.800	8.800	366.200	0.0	0.0	3.300	0.0	0.692

Sheet 21 of Table 3C-1 Elements 114, 117, 123, 130, 131

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 114

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.100	0.0	10.500	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	4.300	243.000	0.0	0.0	2.000	0.0	0.402
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	7.400	0.0	0.0	0.0	0.0	0.0	0.0	0.064

COMBINED STRESSES - ELEMENT NO. 114

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-16.100	28.200	10.500	28.200	0.0	0.0	0.0	-0.052
2	-16.100	28.200	10.500	28.200	0.0	0.0	0.0	-0.052
3	-16.100	0.0	10.500	0.0	0.0	0.0	0.0	0.143
4	-16.100	0.0	10.500	0.0	0.0	0.0	0.0	0.143
5	-1.300	28.200	10.500	28.200	0.0	0.0	0.0	0.076
6	-1.300	28.200	10.500	28.200	0.0	0.0	0.0	0.076
7	-1.300	0.0	10.500	0.0	0.0	0.0	0.0	0.271
8	-1.300	0.0	10.500	0.0	0.0	0.0	0.0	0.271
9	-30.900	28.200	10.500	28.200	0.0	0.0	0.0	-0.180
10	-30.900	28.200	10.500	28.200	0.0	0.0	0.0	-0.180
11	-30.900	0.0	10.500	0.0	0.0	0.0	0.0	0.015
12	-30.900	0.0	10.500	0.0	0.0	0.0	0.0	0.015
13	244.950	33.145	289.950	28.200	0.0	2.300	0.0	0.410
14	244.950	33.145	289.950	28.200	0.0	2.300	0.0	0.410
15	244.950	4.945	289.950	0.0	0.0	2.300	0.0	0.605
16	244.950	4.945	289.950	0.0	0.0	2.300	0.0	0.605
17	321.900	32.500	364.500	28.200	0.0	2.000	0.0	0.623
18	321.900	32.500	364.500	28.200	0.0	2.000	0.0	0.623
19	321.900	4.300	364.500	0.0	0.0	2.000	0.0	0.818
20	321.900	4.300	364.500	0.0	0.0	2.000	0.0	0.818
21	327.820	32.500	364.500	28.200	0.0	2.000	0.0	0.674
22	327.820	32.500	364.500	28.200	0.0	2.000	0.0	0.674
23	327.820	4.300	364.500	0.0	0.0	2.000	0.0	0.869
24	327.820	4.300	364.500	0.0	0.0	2.000	0.0	0.869
25	315.980	32.500	364.500	28.200	0.0	2.000	0.0	0.572
26	315.980	32.500	364.500	28.200	0.0	2.000	0.0	0.572
27	315.980	4.300	364.500	0.0	0.0	2.000	0.0	0.767
28	315.980	4.300	364.500	0.0	0.0	2.000	0.0	0.767
29	450.400	34.650	501.000	28.200	0.0	3.000	0.0	0.860
30	450.400	34.650	501.000	28.200	0.0	3.000	0.0	0.860
31	450.400	6.450	501.000	0.0	0.0	3.000	0.0	1.055
32	450.400	6.450	501.000	0.0	0.0	3.000	0.0	1.055
33	401.050	33.575	440.250	28.200	0.0	2.500	0.0	0.823
34	401.050	33.575	440.250	28.200	0.0	2.500	0.0	0.823
35	401.050	5.375	440.250	0.0	0.0	2.500	0.0	1.018
36	401.050	5.375	440.250	0.0	0.0	2.500	0.0	1.018
37	386.250	33.575	440.250	28.200	0.0	2.500	0.0	0.695
38	386.250	33.575	440.250	28.200	0.0	2.500	0.0	0.695
39	386.250	5.375	440.250	0.0	0.0	2.500	0.0	0.890
40	386.250	5.375	440.250	0.0	0.0	2.500	0.0	0.890
41	336.700	32.500	364.500	28.200	0.0	2.000	0.0	0.751
42	336.700	32.500	364.500	28.200	0.0	2.000	0.0	0.751
43	336.700	4.300	364.500	0.0	0.0	2.000	0.0	0.946
44	336.700	4.300	364.500	0.0	0.0	2.000	0.0	0.946
45	307.100	32.500	364.500	28.200	0.0	2.000	0.0	0.495
46	307.100	32.500	364.500	28.200	0.0	2.000	0.0	0.495
47	307.100	4.300	364.500	0.0	0.0	2.000	0.0	0.690
48	307.100	4.300	364.500	0.0	0.0	2.000	0.0	0.690

Sheet 22 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 117

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-15.400	0.0	8.700	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	0.200	238.000	0.0	0.0	0.800	0.0	0.393
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	6.500	0.0	0.0	0.0	0.0	0.0	0.0	0.064

COMBINED STRESSES - ELEMENT NO. 117

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-15.400	28.200	8.700	28.200	0.0	0.0	0.0	-0.052
2	-15.400	28.200	8.700	28.200	0.0	0.0	0.0	-0.052
3	-15.400	0.0	8.700	0.0	0.0	0.0	0.0	0.143
4	-15.400	0.0	8.700	0.0	0.0	0.0	0.0	0.143
5	-2.400	28.200	8.700	28.200	0.0	0.0	0.0	0.076
6	-2.400	28.200	8.700	28.200	0.0	0.0	0.0	0.076
7	-2.400	0.0	8.700	0.0	0.0	0.0	0.0	0.271
8	-2.400	0.0	8.700	0.0	0.0	0.0	0.0	0.271
9	-28.400	28.200	8.700	28.200	0.0	0.0	0.0	-0.180
10	-28.400	28.200	8.700	28.200	0.0	0.0	0.0	-0.180
11	-28.400	0.0	8.700	0.0	0.0	0.0	0.0	0.015
12	-28.400	0.0	8.700	0.0	0.0	0.0	0.0	0.015
13	245.650	28.430	282.400	28.200	0.0	0.920	0.0	0.400
14	245.650	28.430	282.400	28.200	0.0	0.920	0.0	0.400
15	245.650	0.230	282.400	0.0	0.0	0.920	0.0	0.595
16	245.650	0.230	282.400	0.0	0.0	0.920	0.0	0.595
17	322.600	28.400	357.700	28.200	0.0	0.800	0.0	0.614
18	322.600	28.400	357.700	28.200	0.0	0.800	0.0	0.614
19	322.600	0.200	357.700	0.0	0.0	0.800	0.0	0.809
20	322.600	0.200	357.700	0.0	0.0	0.800	0.0	0.809
21	327.800	28.400	357.700	28.200	0.0	0.800	0.0	0.665
22	327.800	28.400	357.700	28.200	0.0	0.800	0.0	0.665
23	327.800	0.200	357.700	0.0	0.0	0.800	0.0	0.860
24	327.800	0.200	357.700	0.0	0.0	0.800	0.0	0.860
25	317.400	28.400	357.700	28.200	0.0	0.800	0.0	0.563
26	317.400	28.400	357.700	28.200	0.0	0.800	0.0	0.563
27	317.400	0.200	357.700	0.0	0.0	0.800	0.0	0.758
28	317.400	0.200	357.700	0.0	0.0	0.800	0.0	0.758
29	451.100	28.500	491.700	28.200	0.0	1.200	0.0	0.846
30	451.100	28.500	491.700	28.200	0.0	1.200	0.0	0.846
31	451.100	0.300	491.700	0.0	0.0	1.200	0.0	1.041
32	451.100	0.300	491.700	0.0	0.0	1.200	0.0	1.041
33	400.850	28.450	432.200	28.200	0.0	1.000	0.0	0.812
34	400.850	28.450	432.200	28.200	0.0	1.000	0.0	0.812
35	400.850	0.250	432.200	0.0	0.0	1.000	0.0	1.007
36	400.850	0.250	432.200	0.0	0.0	1.000	0.0	1.007
37	387.850	28.450	432.200	28.200	0.0	1.000	0.0	0.684
38	387.850	28.450	432.200	28.200	0.0	1.000	0.0	0.684
39	387.850	0.250	432.200	0.0	0.0	1.000	0.0	0.879
40	387.850	0.250	432.200	0.0	0.0	1.000	0.0	0.879
41	335.600	28.400	357.700	28.200	0.0	0.800	0.0	0.742
42	335.600	28.400	357.700	28.200	0.0	0.800	0.0	0.742
43	335.600	0.200	357.700	0.0	0.0	0.800	0.0	0.937
44	335.600	0.200	357.700	0.0	0.0	0.800	0.0	0.937
45	309.600	28.400	357.700	28.200	0.0	0.800	0.0	0.486
46	309.600	28.400	357.700	28.200	0.0	0.800	0.0	0.486
47	309.600	0.200	357.700	0.0	0.0	0.800	0.0	0.681
48	309.600	0.200	357.700	0.0	0.0	0.800	0.0	0.681

Sheet 23 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 123

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-14.300	0.0	5.500	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	0.0	230.000	0.0	0.0	0.0	0.0	0.388
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	4.900	0.0	0.0	0.0	0.0	0.0	0.0	0.063

COMBINED STRESSES - ELEMENT NO. 123

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-14.300	28.200	5.500	28.200	0.0	0.0	0.0	-0.052
2	-14.300	28.200	5.500	28.200	0.0	0.0	0.0	-0.052
3	-14.300	0.0	5.500	0.0	0.0	0.0	0.0	0.143
4	-14.300	0.0	5.500	0.0	0.0	0.0	0.0	0.143
5	-4.500	28.200	5.500	28.200	0.0	0.0	0.0	0.074
6	-4.500	28.200	5.500	28.200	0.0	0.0	0.0	0.074
7	-4.500	0.0	5.500	0.0	0.0	0.0	0.0	0.269
8	-4.500	0.0	5.500	0.0	0.0	0.0	0.0	0.269
9	-24.100	28.200	5.500	28.200	0.0	0.0	0.0	-0.178
10	-24.100	28.200	5.500	28.200	0.0	0.0	0.0	-0.178
11	-24.100	0.0	5.500	0.0	0.0	0.0	0.0	0.017
12	-24.100	0.0	5.500	0.0	0.0	0.0	0.0	0.017
13	246.750	28.200	270.000	28.200	0.0	0.0	0.0	0.394
14	246.750	28.200	270.000	28.200	0.0	0.0	0.0	0.394
15	246.750	0.0	270.000	0.0	0.0	0.0	0.0	0.589
16	246.750	0.0	270.000	0.0	0.0	0.0	0.0	0.589
17	323.700	28.200	346.500	28.200	0.0	0.0	0.0	0.609
18	323.700	28.200	346.500	28.200	0.0	0.0	0.0	0.609
19	323.700	0.0	346.500	0.0	0.0	0.0	0.0	0.804
20	323.700	0.0	346.500	0.0	0.0	0.0	0.0	0.804
21	327.620	28.200	346.500	28.200	0.0	0.0	0.0	0.659
22	327.620	28.200	346.500	28.200	0.0	0.0	0.0	0.659
23	327.620	0.0	346.500	0.0	0.0	0.0	0.0	0.854
24	327.620	0.0	346.500	0.0	0.0	0.0	0.0	0.854
25	319.780	28.200	346.500	28.200	0.0	0.0	0.0	0.559
26	319.780	28.200	346.500	28.200	0.0	0.0	0.0	0.559
27	319.780	0.0	346.500	0.0	0.0	0.0	0.0	0.754
28	319.780	0.0	346.500	0.0	0.0	0.0	0.0	0.754
29	452.200	28.200	476.500	28.200	0.0	0.0	0.0	0.839
30	452.200	28.200	476.500	28.200	0.0	0.0	0.0	0.839
31	452.200	0.0	476.500	0.0	0.0	0.0	0.0	1.034
32	452.200	0.0	476.500	0.0	0.0	0.0	0.0	1.034
33	400.350	28.200	419.000	28.200	0.0	0.0	0.0	0.805
34	400.350	28.200	419.000	28.200	0.0	0.0	0.0	0.805
35	400.350	0.0	419.000	0.0	0.0	0.0	0.0	1.000
36	400.350	0.0	419.000	0.0	0.0	0.0	0.0	1.000
37	390.550	28.200	419.000	28.200	0.0	0.0	0.0	0.679
38	390.550	28.200	419.000	28.200	0.0	0.0	0.0	0.679
39	390.550	0.0	419.000	0.0	0.0	0.0	0.0	0.874
40	390.550	0.0	419.000	0.0	0.0	0.0	0.0	0.874
41	333.500	28.200	346.500	28.200	0.0	0.0	0.0	0.735
42	333.500	28.200	346.500	28.200	0.0	0.0	0.0	0.735
43	333.500	0.0	346.500	0.0	0.0	0.0	0.0	0.930
44	333.500	0.0	346.500	0.0	0.0	0.0	0.0	0.930
45	313.900	28.200	346.500	28.200	0.0	0.0	0.0	0.483
46	313.900	28.200	346.500	28.200	0.0	0.0	0.0	0.483
47	313.900	0.0	346.500	0.0	0.0	0.0	0.0	0.678
48	313.900	0.0	346.500	0.0	0.0	0.0	0.0	0.678

Sheet 24 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 130

FUND. LOAD NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-13.200	0.0	2.200	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	0.0	227.000	0.0	0.0	0.0	0.0	0.383
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	3.500	0.0	0.0	0.0	0.0	0.0	0.0	0.059

COMBINED STRESSES - ELEMENT NO. 130

LOAD. COMB. NO.	MERIDIONAL STRESS RES (K/FT)	MERIDIONAL STRESS CPL (K-FT/FT)	HOOP STRESS RES (K/FT)	HOOP STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-13.200	28.200	2.200	28.200	0.0	0.0	0.0	-0.052
2	-13.200	28.200	2.200	28.200	0.0	0.0	0.0	-0.052
3	-13.200	0.0	2.200	0.0	0.0	0.0	0.0	0.143
4	-13.200	0.0	2.200	0.0	0.0	0.0	0.0	0.143
5	-6.200	28.200	2.200	28.200	0.0	0.0	0.0	0.066
6	-6.200	28.200	2.200	28.200	0.0	0.0	0.0	0.066
7	-6.200	0.0	2.200	0.0	0.0	0.0	0.0	0.261
8	-6.200	0.0	2.200	0.0	0.0	0.0	0.0	0.261
9	-20.200	28.200	2.200	28.200	0.0	0.0	0.0	-0.170
10	-20.200	28.200	2.200	28.200	0.0	0.0	0.0	-0.170
11	-20.200	0.0	2.200	0.0	0.0	0.0	0.0	0.025
12	-20.200	0.0	2.200	0.0	0.0	0.0	0.0	0.025
13	247.850	28.200	263.250	28.200	0.0	0.0	0.0	0.388
14	247.850	28.200	263.250	28.200	0.0	0.0	0.0	0.388
15	247.850	0.0	263.250	0.0	0.0	0.0	0.0	0.583
16	247.850	0.0	263.250	0.0	0.0	0.0	0.0	0.583
17	324.800	28.200	340.200	28.200	0.0	0.0	0.0	0.604
18	324.800	28.200	340.200	28.200	0.0	0.0	0.0	0.604
19	324.800	0.0	340.200	0.0	0.0	0.0	0.0	0.799
20	324.800	0.0	340.200	0.0	0.0	0.0	0.0	0.799
21	327.600	28.200	340.200	28.200	0.0	0.0	0.0	0.651
22	327.600	28.200	340.200	28.200	0.0	0.0	0.0	0.651
23	327.600	0.0	340.200	0.0	0.0	0.0	0.0	0.846
24	327.600	0.0	340.200	0.0	0.0	0.0	0.0	0.846
25	322.000	28.200	340.200	28.200	0.0	0.0	0.0	0.557
26	322.000	28.200	340.200	28.200	0.0	0.0	0.0	0.557
27	322.000	0.0	340.200	0.0	0.0	0.0	0.0	0.752
28	322.000	0.0	340.200	0.0	0.0	0.0	0.0	0.752
29	453.300	28.200	468.700	28.200	0.0	0.0	0.0	0.831
30	453.300	28.200	468.700	28.200	0.0	0.0	0.0	0.831
31	453.300	0.0	468.700	0.0	0.0	0.0	0.0	1.026
32	453.300	0.0	468.700	0.0	0.0	0.0	0.0	1.026
33	400.050	28.200	411.950	28.200	0.0	0.0	0.0	0.795
34	400.050	28.200	411.950	28.200	0.0	0.0	0.0	0.795
35	400.050	0.0	411.950	0.0	0.0	0.0	0.0	0.990
36	400.050	0.0	411.950	0.0	0.0	0.0	0.0	0.990
37	393.050	28.200	411.950	28.200	0.0	0.0	0.0	0.677
38	393.050	28.200	411.950	28.200	0.0	0.0	0.0	0.677
39	393.050	0.0	411.950	0.0	0.0	0.0	0.0	0.872
40	393.050	0.0	411.950	0.0	0.0	0.0	0.0	0.872
41	331.800	28.200	340.200	28.200	0.0	0.0	0.0	0.722
42	331.800	28.200	340.200	28.200	0.0	0.0	0.0	0.722
43	331.800	0.0	340.200	0.0	0.0	0.0	0.0	0.917
44	331.800	0.0	340.200	0.0	0.0	0.0	0.0	0.917
45	317.800	28.200	340.200	28.200	0.0	0.0	0.0	0.486
46	317.800	28.200	340.200	28.200	0.0	0.0	0.0	0.486
47	317.800	0.0	340.200	0.0	0.0	0.0	0.0	0.681
48	317.800	0.0	340.200	0.0	0.0	0.0	0.0	0.681

Sheet 25 of Table 3C-1

Table 3C-1

CONTAINMENT SHELL STRESS CALCULATION RESULTS (Continued)

ELEMENT NO. 131

FUND. LOAD NO.	------ MERIDIONAL ----- STRESS RES (K/FT)	STRESS CPL (K-FT/FT)	------- HOOP ------- STRESS RES (K/FT)	STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	28.200	0.0	28.200	0.0	0.0	0.0	-0.052
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.143
5	227.000	0.0	227.000	0.0	0.0	0.0	0.0	0.383
6	111.000	0.0	111.000	0.0	0.0	0.0	0.0	0.273
7	126.000	0.0	126.000	0.0	0.0	0.0	0.0	0.309
8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

COMBINED STRESSES - ELEMENT NO. 131

LOAD. COMB. NO.	------ MERIDIONAL ----- STRESS RES (K/FT)	STRESS CPL (K-FT/FT)	------- HOOP ------- STRESS RES (K/FT)	STRESS CPL (K-FT/FT)	MERIDIONAL SHEAR (K/FT)			RADIAL DISPLACEMENT (IN)
1	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
2	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
3	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
4	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
5	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
6	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
7	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
8	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
9	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
10	-10.200	28.200	-10.200	28.200	0.0	0.0	0.0	-0.052
11	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
12	-10.200	0.0	-10.200	0.0	0.0	0.0	0.0	0.143
13	250.850	28.200	250.850	28.200	0.0	0.0	0.0	0.388
14	250.850	28.200	250.850	28.200	0.0	0.0	0.0	0.388
15	250.850	0.0	250.850	0.0	0.0	0.0	0.0	0.583
16	250.850	0.0	250.850	0.0	0.0	0.0	0.0	0.583
17	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
18	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
19	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
20	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
21	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
22	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
23	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
24	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
25	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
26	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
27	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
28	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.794
29	456.300	28.200	456.300	28.200	0.0	0.0	0.0	0.831
30	456.300	28.200	456.300	28.200	0.0	0.0	0.0	0.831
31	456.300	0.0	456.300	0.0	0.0	0.0	0.0	1.026
32	456.300	0.0	456.300	0.0	0.0	0.0	0.0	1.026
33	399.550	28.200	399.550	28.200	0.0	0.0	0.0	0.736
34	399.550	28.200	399.550	28.200	0.0	0.0	0.0	0.736
35	399.550	0.0	399.550	0.0	0.0	0.0	0.0	0.931
36	399.550	0.0	399.550	0.0	0.0	0.0	0.0	0.931
37	399.550	28.200	399.550	28.200	0.0	0.0	0.0	0.736
38	399.550	28.200	399.550	28.200	0.0	0.0	0.0	0.736
39	399.550	0.0	399.550	0.0	0.0	0.0	0.0	0.931
40	399.550	0.0	399.950	0.0	0.0	0.0	0.0	0.931
41	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
42	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
43	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
44	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.749
45	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
46	327.800	28.200	327.800	28.200	0.0	0.0	0.0	0.604
47	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799
48	327.800	0.0	327.800	0.0	0.0	0.0	0.0	0.799

APPENDIX 3D

CONTAINMENT TENDON ANCHORAGE HARDWARE CAPACITY TESTS

by PITTSBURGH TESTING LABORATORY

Compressive Load Tests of 90 Wire Tendon Base Plate - Test on Concrete Stand

GTNNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 652408

March 29, 1967

ORDER No. PG-18619

CLIENT'S No. 217114-3

REPORT

Report of: Compressive Load Tests of 90 Wire Tendon Base Plate Test on Concrete Stand

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

We were requested to fabricate a concrete base plate in accordance with Ryerson Drawing SPI-1 dated 1/20/67. A concrete mix design, reinforcing bars, base plate and trumpet were submitted for fabrication of the concrete base plate.

The following concrete properties were recorded.

CONCRETE MIX DESIGN PER CU. YD.

Type III Portland Cement	**611 lbs.**
Dravo Corp. Siliceous Sand ASTM C-33	**1240 lbs. S.S.D.**
Dravo Corp. Siliceous Gravel 1" Size	**1850 lbs. S.S.D.**
Water	**300 lbs.**
Slump	**4 inches**

COMPRESSIVE STRENGTHS

Date of Testing	Sectional Area Sq. In.	Crushing Load Lbs.	Crushing Strength PSI	Age Days
March 8, 1967	28.27	92,000	3250	2
March 8, 1967	28.27	81,000	2870	2
			3060 Average	
March 9, 1967	28.27	115,000	4070	3
March 9, 1967	28.27	120,000	4240	3
			4150 Average	
March 10, 1967	28.27	124,000	4390	4
March 10, 1967	28.27	121,000	4280	4
			4340 Average	

Sheet 2 of Report

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 652408

March 29, 1967

ORDER No. PG-18619

CLIENT'S No. 21T114-3

REPORT

When the concrete in the stand had reached the requested strength, the stand was tested by the following method.

A compressive load of 742,000 lbs. was applied in increments of 106,000 lbs., and then released in increments of 106,000 lbs. The gage readings tabulated below were obtained using a deflectometer designed as shown on Page 5 of Ryerson instructions dated 2/2/67.

Cycle One was repeated, recording the same gage readings.

On the third cycle, dial gage readings were recorded only up to 742,000 lbs. The loading continued in 106,000 lbs. increments to 1,200,000 lbs. At 954,000 lbs. hairline cracks appeared on the sides of the stand. There were no other apparent defects at 1,200,000 lbs.

The dial gage instrument was designed so that measurements, either compressive or expansive, were recorded at a specified distance from the center line of the concrete stand of metal base plate.

Gage No.	Location
1	On the concrete 3 inches from edge of base plate.
2	On the base plate 7-1/2 inches from center line of stand.
3	On the base plate 4-3/4 inches from center line of stand.
4	On the base plate 6 inches from center line of stand.
5	On the concrete 1 inch from edge of base plate.

Sheet 3 of Report

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 652408

March 29, 1967

ORDER No. PG-18619

CLIENT'S No. 21T114-3

REPORT

LOAD DEFORMATION MEASUREMENTS

1st Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	.000
106,000	-.001	.000	.002	.001	.000
212,000	-.002	.001	.006	.004	-.001
318,000	-.002	.002	.009	.005	-.004
424,000	-.003	.002	.011	.007	-.007
530,000	-.004	.003	.013	.009	-.009
636,000	-.005	.004	.016	.011	-.010
742,000	-.006	.004	.018	.013	-.012
636,000	-.006	.004	.017	.012	-.013
530,000	-.005	.004	.016	.012	-.013
424,000	-.005	.004	.015	.011	-.012
318,000	-.004	.003	.014	.010	-.012
212,000	-.004	.003	.012	.008	-.012
106,000	-.003	.002	.009	.006	-.012
0	.000	.000	.003	.002	-.002

2nd Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	-.002
106,000	-.002	.001	.004	.003	-.007
212,000	-.003	.002	.004	.004	-.009
318,000	-.004	.003	.009	.006	-.011
424,000	-.005	.003	.010	.007	-.012
530,000	-.005	.003	.012	.008	-.013
636,000	-.006	.004	.013	.010	-.014
742,000	-.006	.004	.015	.011	-.015
636,000	-.006	.004	.014	.010	-.0165
530,000	-.006	.004	.013	.010	-.014
424,000	-.005	.0035	.012	.0085	-.013
318,000	-.005	.003	.011	.0075	-.0125
212,000	-.004	.003	.009	.006	-.0115
106,000	-.003	.002	.005	.004	-.010
0	.000	.000	.000	.000	-.002

Sheet 4 of Report

GINNA/UFSAR

IM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

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LABORATORY No. 652408

March 29, 1967

ORDER No. PG-18619

CLIENT'S No. 21T114-3

REPORT

LOAD DEFORMATION MEASUREMENTS

3rd Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	-.002
106,000	-.003	.002	.004	.003	-.009
212,000	-.004	.002	.007	.0045	-.011
318,000	-.004	.003	.009	.006	-.012
424,000	-.005	.003	.011	.007	-.013
530,000	-.006	.0035	.012	.0085	-.014
636,000	-.006	.004	.0135	.010	-.015
742,000	-.007	.004	.015	.011	-.0155

954,000 Hair line cracks visible.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Ryerson Steel
1-PTL Chicago

vs

3D-4

Compressive Load Tests of 90 Wire Tendon Base Plate - Test on Concrete Stand

GINNA/UFSAR

BASEPLATE
FOR 90 WIRE TENDON

A. LOADS

Loads developped by the 90 wire Tendon.

Ultimate Strength	1060 K	
Overstressing Force	848 K	
Initial Force	742 K	← x
Final Force	636 K	

x Design Force for Baseplate

B. SIZE.

φ od	18 1/2"	→	269 □"
φ id	6"	→	29 □"
Net Bearing Area			240 □"

Plate thickness 2 1/2"

CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS
RYERSON JOSEPH T. RYERSON & SON, INC.

MADE BY A.W.S. DATE 2/2/67
21 PT-34-114

Sheet 2 of Notes

C) BEARING STRESSES

1) Actual

Average 742'000/240 = 3090 psi

2) Allowable for Base-Slab

(use ACI Codes

$f_c' = f_{ci}' = 4000$ psi

$A_b' \rightarrow \phi\ 2\text{-}8" - 803\ \square"$ (Tendonspacing)

$f_{cp} = 0.6 \cdot 4000 \sqrt[3]{803/269} = 3450$ psi

> actual O.K.

3) Allowable for Wall and Dome

$f_c' = f_{ci}' = 5000$ psi

A_b' (use minimum 1" clearance around Plates

$\rightarrow \phi\ 20\tfrac{1}{2}" = 330\ \square"$

$f_{cp} = 0.6 \cdot 5000 \sqrt[3]{330/269} = 3210$ psi

> actual O.K

Conclusion: The Bearing plate size (see B.) is in accordance with the ACI-Code requirement as used on this Project.

CUSTOMER Bechtel Company

Palisades 5935-C-51

RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS

DATE 2/2/67

MADE BY AW

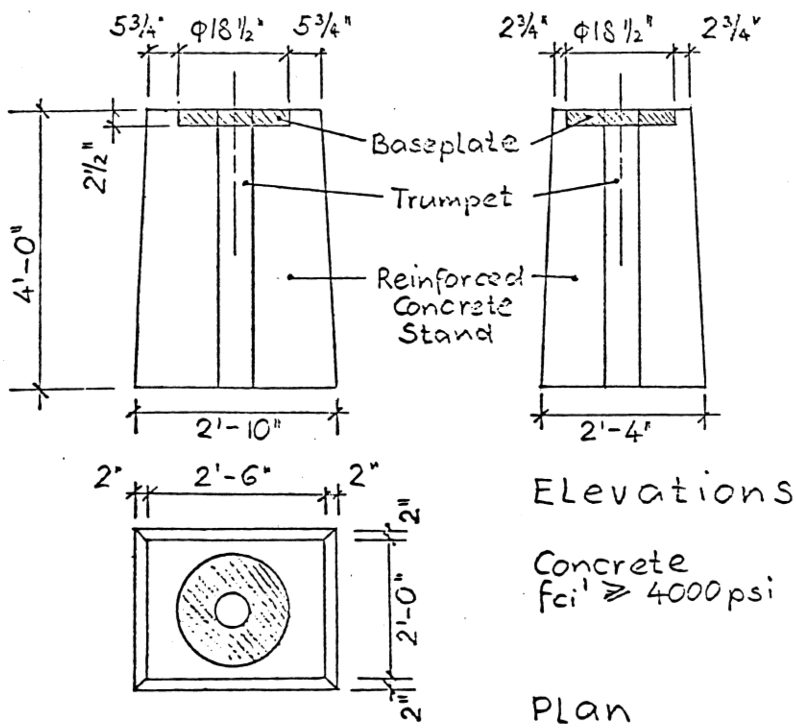
Sheet 3 of Notes

D) BASEPLATE TEST

To verify the Adequacy of Plate-thickness and Plate-Material-strength the following Test is proposed.

1) Test Set up

See Ryerson drawing SPT-1 dated 1-20-67.



CUSTOMER Bechtel Company

Palisades 5935-C-51

METALOGICS

RYERSON

JOSEPH T. RYERSON & SON, INC.

DATE 2/2/65

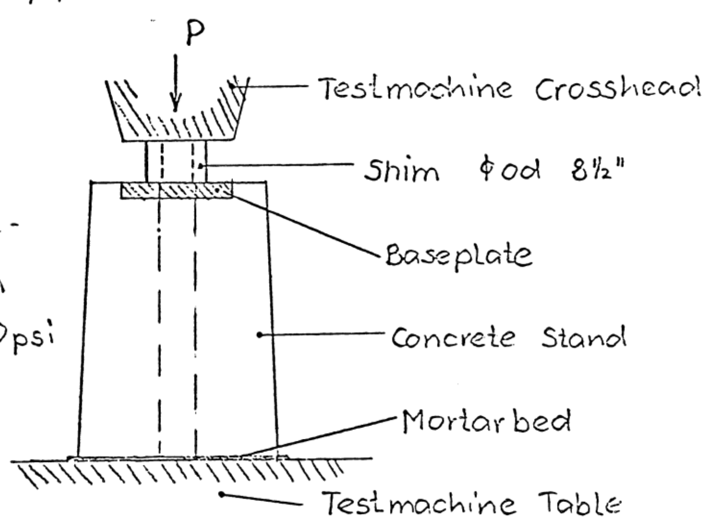
MADE BY A.V/S

Sheet 4 of Notes

GINNA/UFSAR

2) Application of Load

Concrete-Strength $f_{ci}' \geq 4000$ psi



a) Apply Load in increments of 106^k to 742^k max.

b) Release Load in increments of 106^k to zero.

c) Repeat a) and b)

d) Apply Load in increments of 106^k to Failure or Testmachine Capacity

e) Measure Deformations after each Load-increment of a), b) + c). (Set up see 3))

f) Observe Concrete Stand (for Cracks)

CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS RYERSON JOSEPH T. RYERSON & SON, INC.

DATE 2/2/67

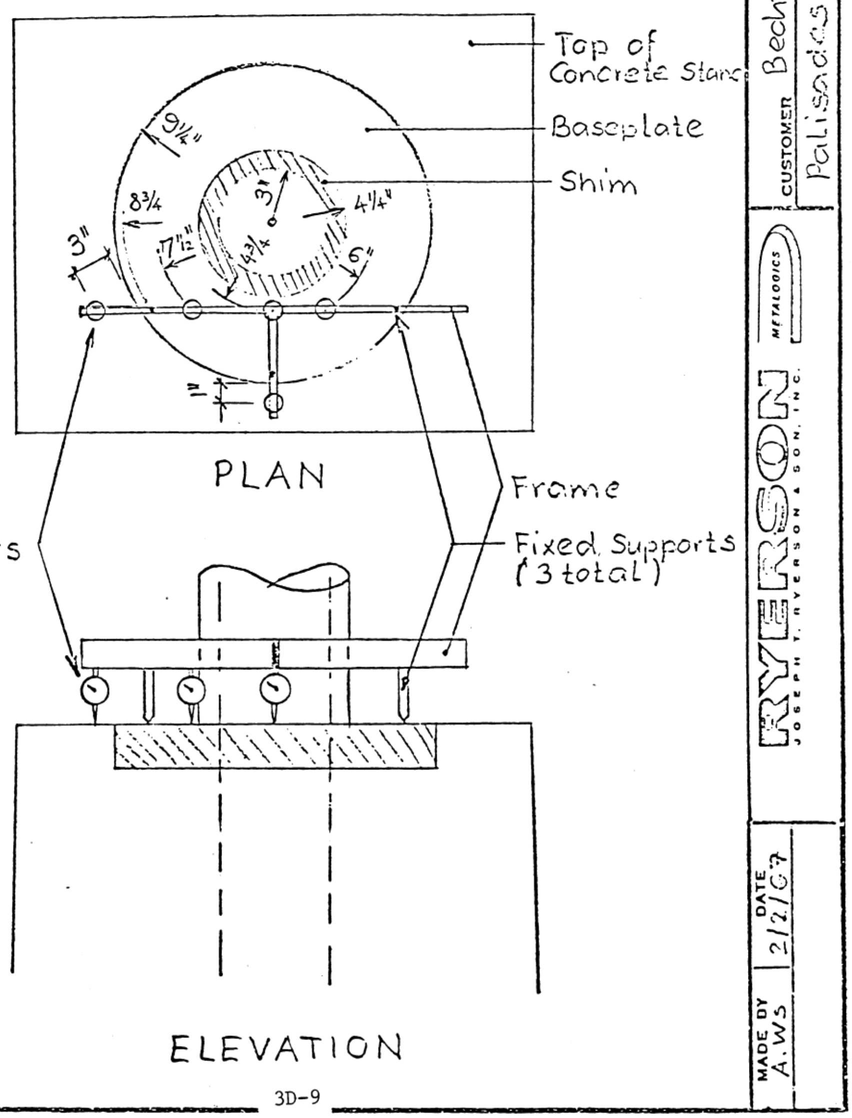
MADE BY A.W.S.

Sheet 5 of Notes

GINNA/UFSAR

3) Deformation - Measurements

The instrumentation is shown only to illustrate the required readings.



Sheet 6 of Notes

GINNA/UFSAR

4) Anticipated Test Results

a) Obervation of Concrete Stand

It is anticipated, that the Concrete Stand does not crack (other than Hairline Cracks) up to the design load of 742^K. The Hairline cracks to close after removing of the Load. Spalling of the unreinforced (and non structural) Concrete around the Baseplate may occur and is insignificant.

b) Observation of Baseplate

It is anticipatet, that the Plate-Material is not subjected to stresses greater than the Yield strength up to the design Load of 742^K. The deformation measurements should therefore vary Linear with the Load and indicate complete (90%) recovery during unloading. The amount of the deformation measurements to be determined later. (max. Reading < 1/16")

CUSTOMER Bechtel Company Palisades 5935-C-51

METALOGICS RYERSON JOSEPH T. RYERSON & SON, INC.

DATE 2/2/67

MADE BY A.Ws

Sheet 7 of Notes

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The edge of the Baseplate should stay flush with the edge of the Concrete. Slight seating in is permissible, "curling up" indicates undesirable uneven bearing stress distribution.

5) Concrete Mix.

See attached Letter from Bechtel Corporation to Ryerson dated 1/27/67.

Because of the Specimen Size Limit the max. aggregate to 1½".

Perform the Test if Concrete Test Cylinders indicate a strength greater than 4000 psi Test Cylinders shall be broken on the same day as the bearing plate test is performed.

CUSTOMER Bechtel Company Palisades 5935-C-51

METALOGICS RYERSON JOSEPH T. RYERSON & SON, INC.

MADE BY A.W. DATE 2/2/67 21PT-34-114

Sheet 8 of Notes

GINNA/UFSAR

6) Baseplate - Material

See attached Heat Test Report regarding the chemical Composition. (which meets ASTM-A36) The pysical Test-Report of representitiv Samples will follow.

CUSTOMER Bechtel Company
Palisades 5935-C-51

RYERSON
JOSEPH T. RYERSON & SON, INC.
METALOGICS

MADE BY A.Ws.
DATE 2/2/67

3D-12

Compression Tests of 90-Wire Anchor Head Assembly

GINNA/UFSAR

107 REV.

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
PITTSBURGH, PA.

LABORATORY No. 646099

October 24,1966

ORDER No. CH-9583

CLIENT'S No. 212341903-18

REPORT

Report of: Compression Tests of 90-Wire Anchor Head Assembly

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

We received two (2) 90-wire anchor head assemblies for compression tests in accordance with Drawing 90-PT-1A, 90-PT-2A and addendum dated 10/11/66.

The shims and anchor heads were assembled, loaded for two minutes and disassembled for examination in accordance with the drawings. The following observations were recorded.

ANCHOR HEAD ASSEMBLY 90-PT-1

Load	Remarks
742,000 lbs.	Button headed wires deformed anchor head. The 1/16" and 1/8" shims deformed slightly. Anchor head loosens by hand from adaptor lock nut.
848,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
954,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,007,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,060,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,200,000 lbs.	Deformations from the shim plates visible on adaptor. Anchor head no longer loosens by hand.

Sheet 2 of Report

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 646099

CLIENT'S No. 212341903-18

October 24, 1966

ORDER No. CH-9583

REPORT

ANCHOR HEAD ASSEMBLY 90-PT-2

Load	Remarks
742,000 lbs.	Button headed wires deformed anchor head. The 1/16" and 1/8" shims deformed slightly.
848,000 lbs.	No apparent deformations except as noted above.
954,000 lbs.	No apparent deformations except as noted above.
1,007,000 lbs.	No apparent deformations except as noted above.
1,060,000 lbs.	No apparent deformations except as noted above.
1,200,000 lbs.	No apparent deformations except as noted above.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Joseph T. Ryerson & Son, Inc.
Attn: Mr. Richard H. [illegible]
1-PTL Chicago

3D-14

Compression Tests of 90-Wire Anchor Head Assembly

GINNA/UFSAR

COMPRESSION TEST PROCEDURE

TEST OF 90 WIRE ANCHOR HEAD ASSEMBLY

SET UP TEST IN MACHINE PER DRAWING 90-PT-1A

APPLY COMPRESSION TO DESIGNATED LOAD (SEE TABLE BELOW)

(THIS IS A STATIC TEST, APPLY + RELEASE LOADS ACCORDINGLY)

HOLD EACH LOAD FOR A PERIOD OF TWO MINUTES

RELEASE LOAD AND DISASSEMBLE

CHECK AND REPORT ON ALL DEFORMATIONS, CRACKS, OR OTHER SIGNS OF FAILURE IN THE ANCHOR HEAD, ADAPTOR LOCK NUT, AND/OR TUBE SHIMS.

REASSEMBLE AND REPEAT AT NEXT HIGHER LOAD.

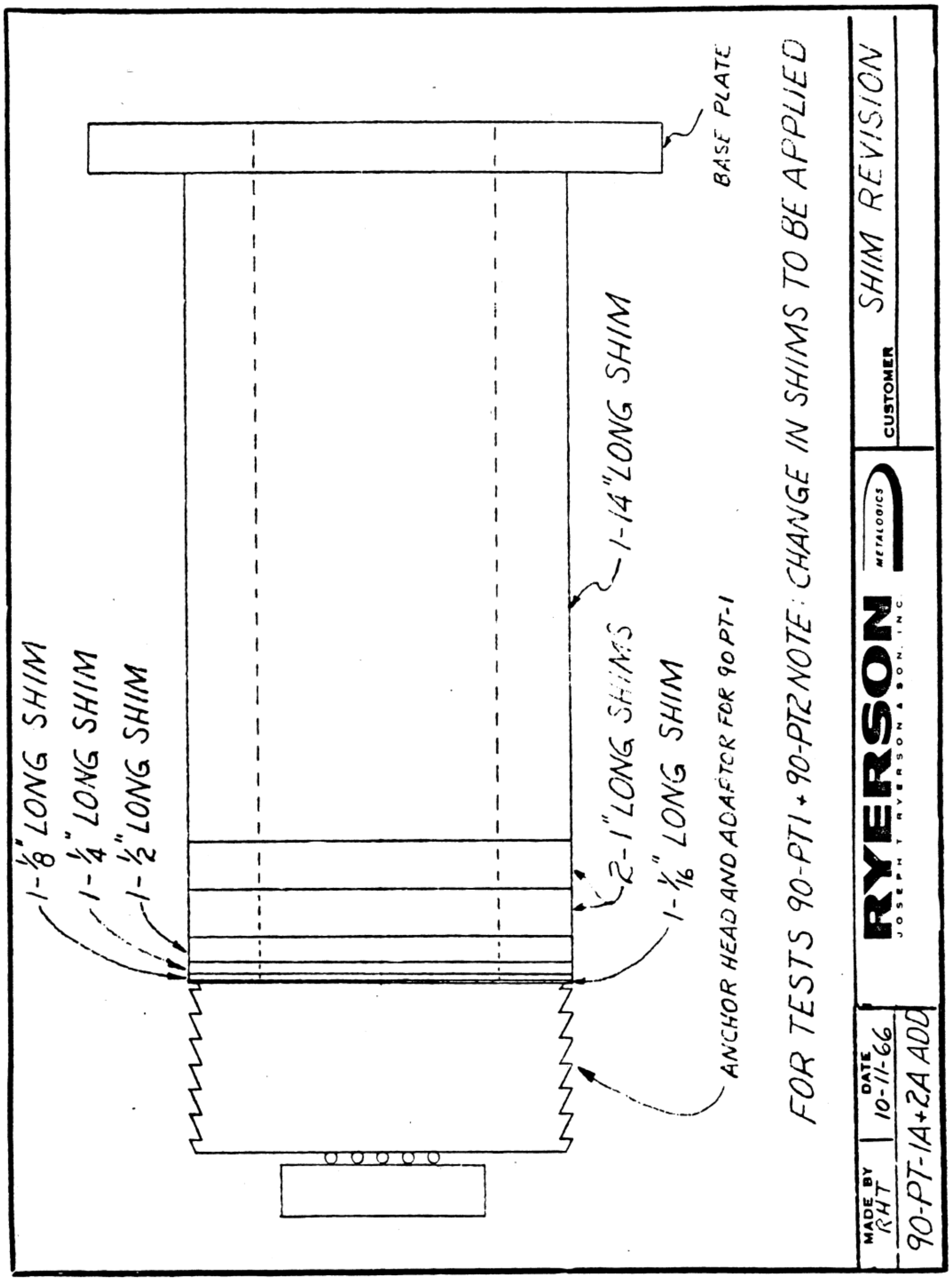
LOAD TABLE

742,000 LBS.	1,007,000 LBS.
848,000 "	1,060,000 "
954,000 "	MACHINE MAXIMUM

MADE BY	DATE	RYERSON METALOGICS JOSEPH T. RYERSON & SON, INC.	CUSTOMER	WIRE TEST
RHT	7-25-66			ANCHOR HEAD
90-PT-1				

Sheet 2 of Notes

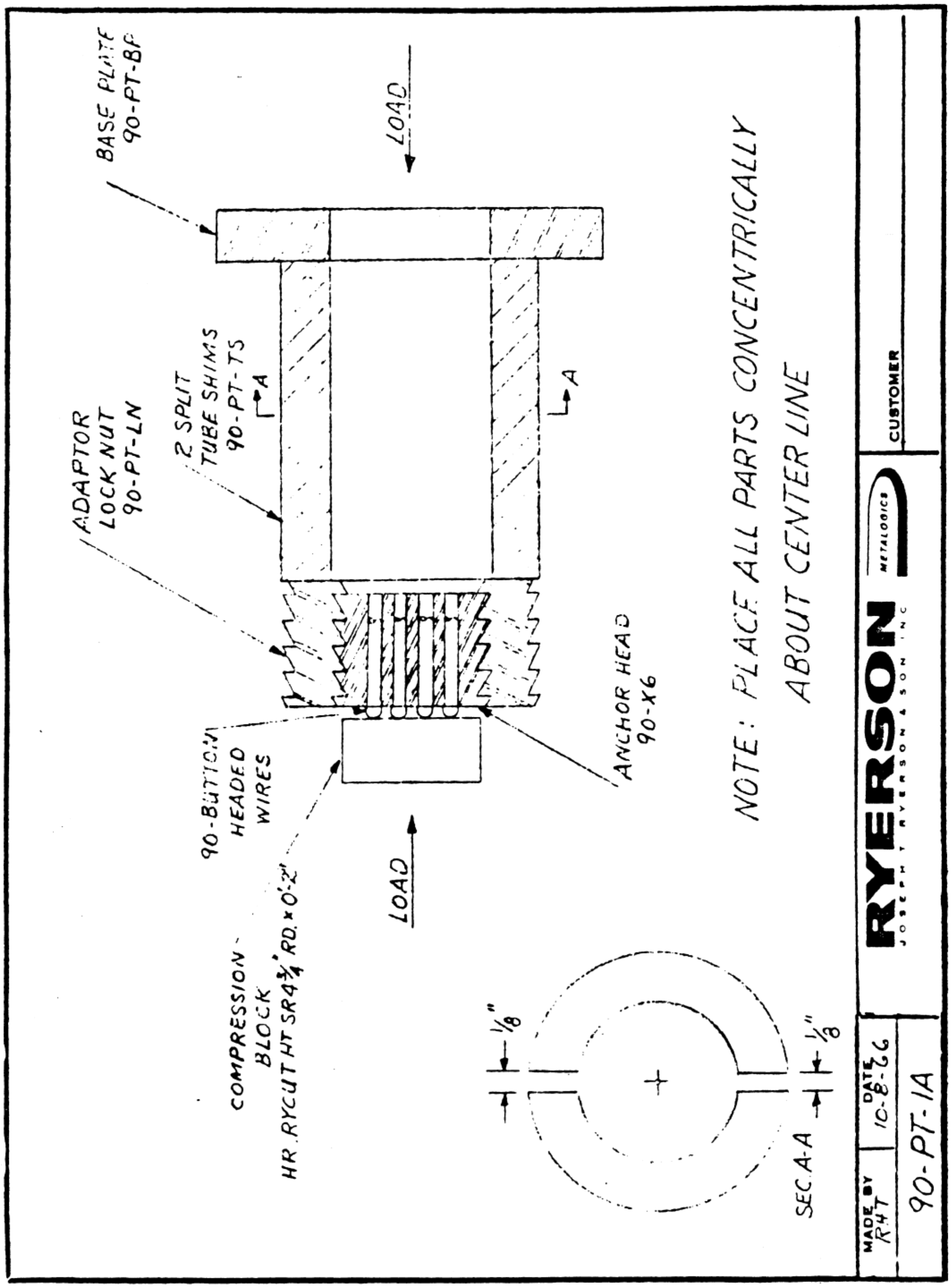
GINNA/UFSAR



3D-16

Sheet 3 of Notes

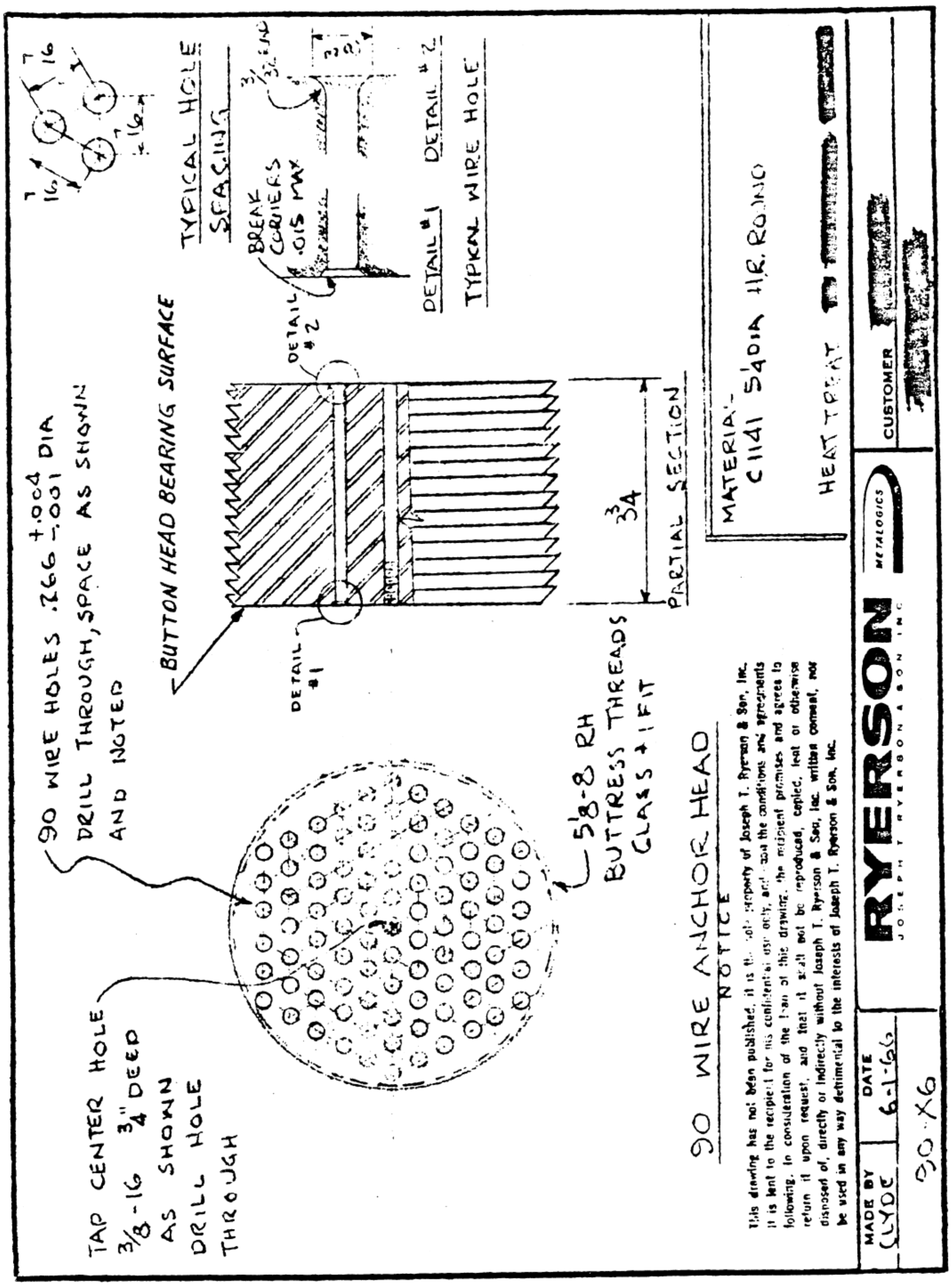
GINNA/UFSAR



3D-17

Sheet 4 of Notes

GINNA/UFSAR



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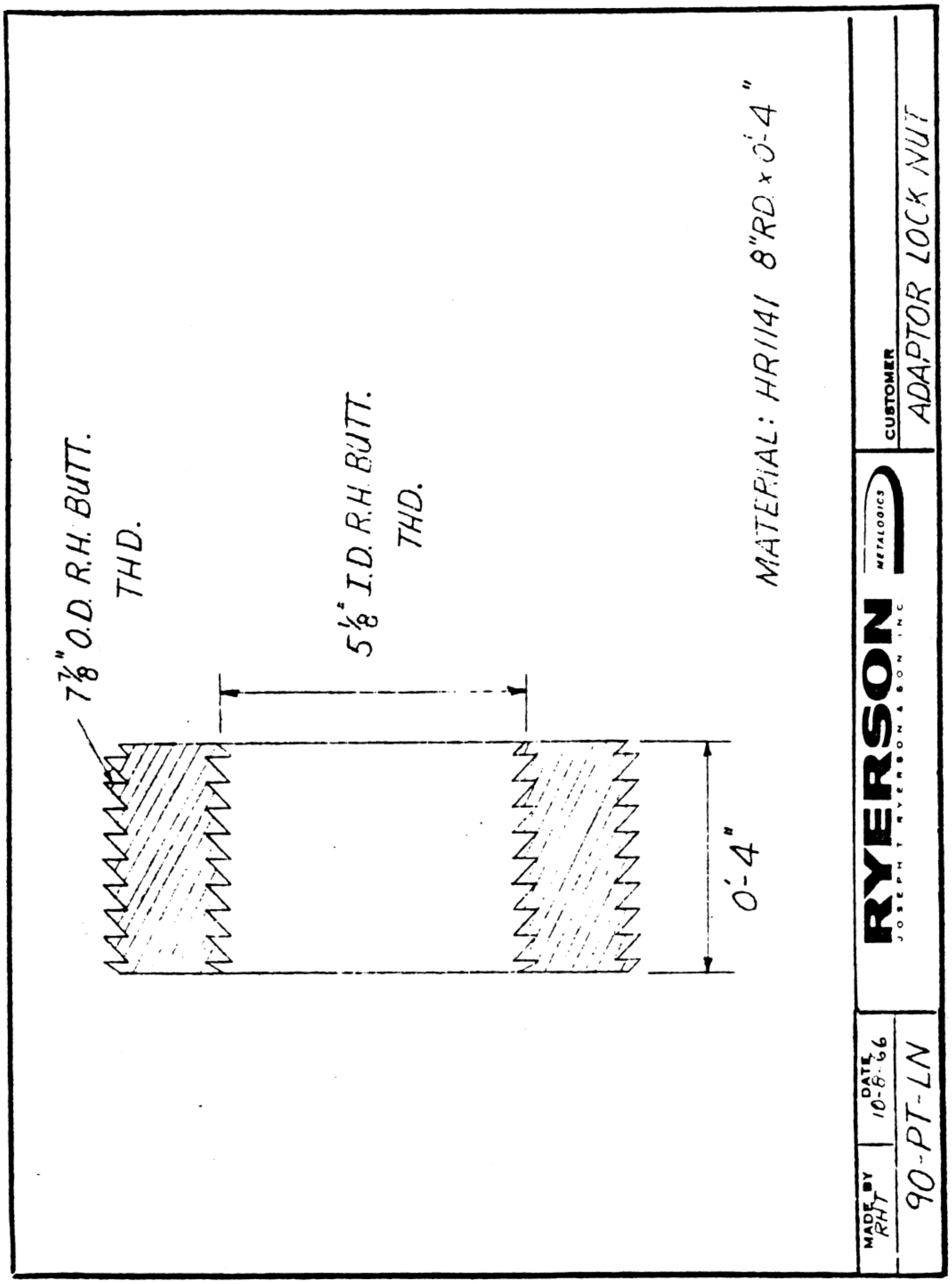
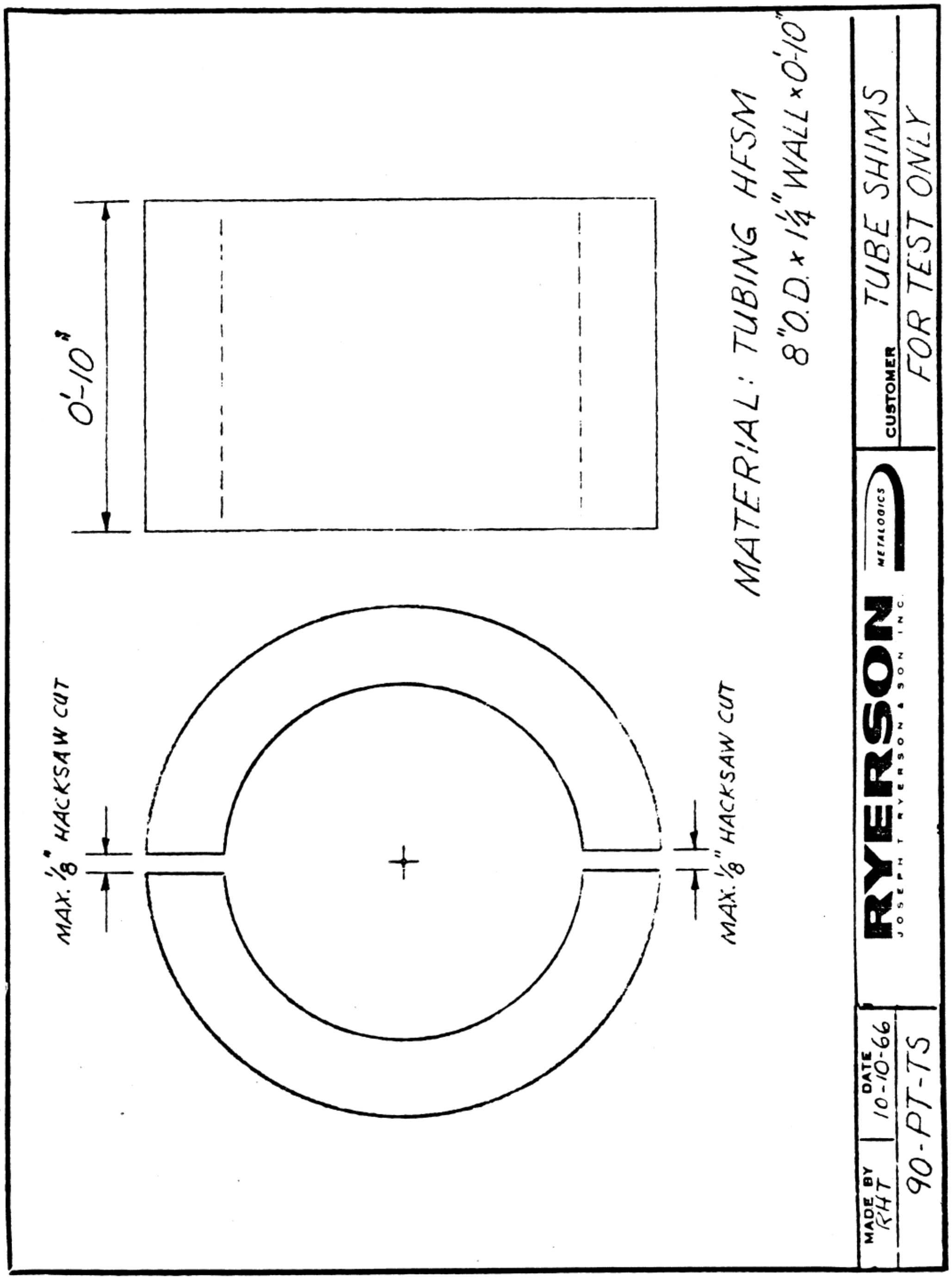


Sheet 6 of Notes



Sheet 7 of Notes

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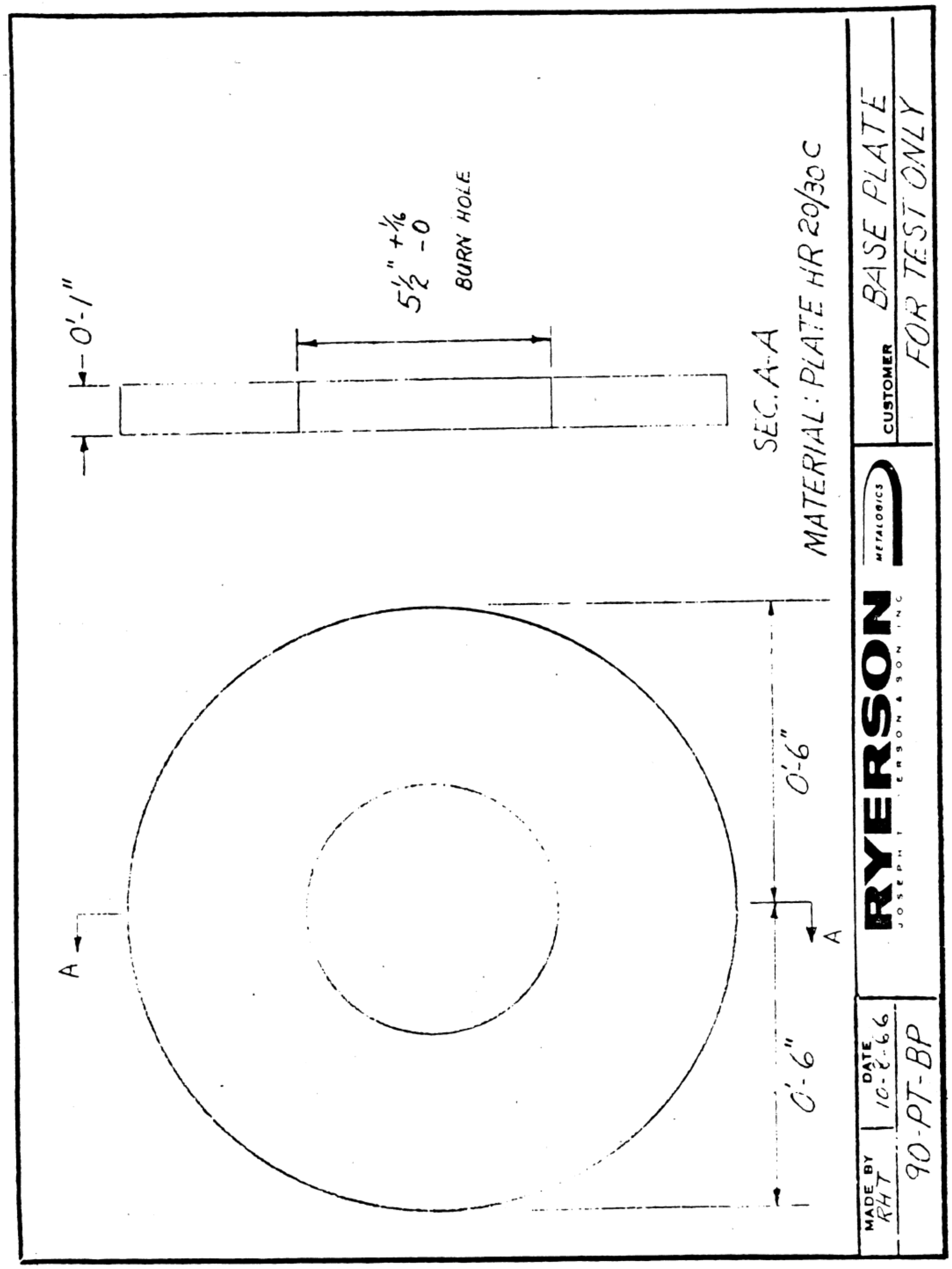

3D-21

Load Tests of Coupler and Adaptor 90-11

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 642438

Augest 25, 1966

ORDER No. CH-9583

CLIENT'S No. 21T341891-60

REPORT

Report of: Load Tests of Coupler and Adaptor 90-11

Report to: Joseph I. Ryerson & Son, Inc.
P.O. Box 8000-A
Chicago, Illinois, 60680

Attention: Mr. W. A. Corson

We received at our laboratory one bushing measuring 11" long, 7-7/8" x 8 buttress threads on the O. D. and 5-1/8" x 8 buttress threads on the I. D., along with a pulling rod measuring 18" long with 3-3/4" of 5-1/8" x 8 buttress threads. This bushing was to be used in conjunction with the coupling identified in our Laboratory Report No. 640730. The set up was made as shown on Ryerson drawing, that is, the bushing was threaded into the 10-1/2" diameter coupling with a 5-1/4" pull rod on one end and the 8" pull rod on the other end. The assembly was then loaded and tensioned to the required loads, then released and disassembled and the threads checked both inside and outside the bushing for visible defects. It was also checked whether or not the pulling rods turned easily or with difficulty.

The results of these tests are as follows:

Load Lbs.		Remarks
742,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
848,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
954,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.

Sheet 2 of Report

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PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 642438

Augest 25, 1966

ORDER No. CH-9583

CLIENT'S No. 21T341891-60

REPORT

Load Lbs.		Remarks
1,007,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
1,060,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
1,200,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher/Manger
Physical Testing Department

cc: 3-Client
Attn: W. A. Corson
1-PTL Chicago

3D-23

Load Tests of Coupler and Adaptor 90-11

TEST PROCEDURE

TEST OF COUPLER 90-X9

SET UP TEST IN MACHINE PER DRAWING

APPLY TENSION TO DESIGNATED LOAD (SEE TABLE BELOW)
(THIS IS A STATIC TEST, APPLY & RELEASE LOADS ACCORDINGLY)

RELEASE LOAD AND DISASSEMBLE

CHECK THREADS AT BOTH END OF COUPLER

CHECK FOR VISIBLE DEFECTS

CHECK FOR TORQUE REQUIRED TO TURN RODS IN COUPLER

MEASUREMENTS ARE NOT NECESSARY, QUALITATIVE REMARKS (TURNS EASILY, DIFFICULT TO TURN, ETC) ARE SUFFICENT

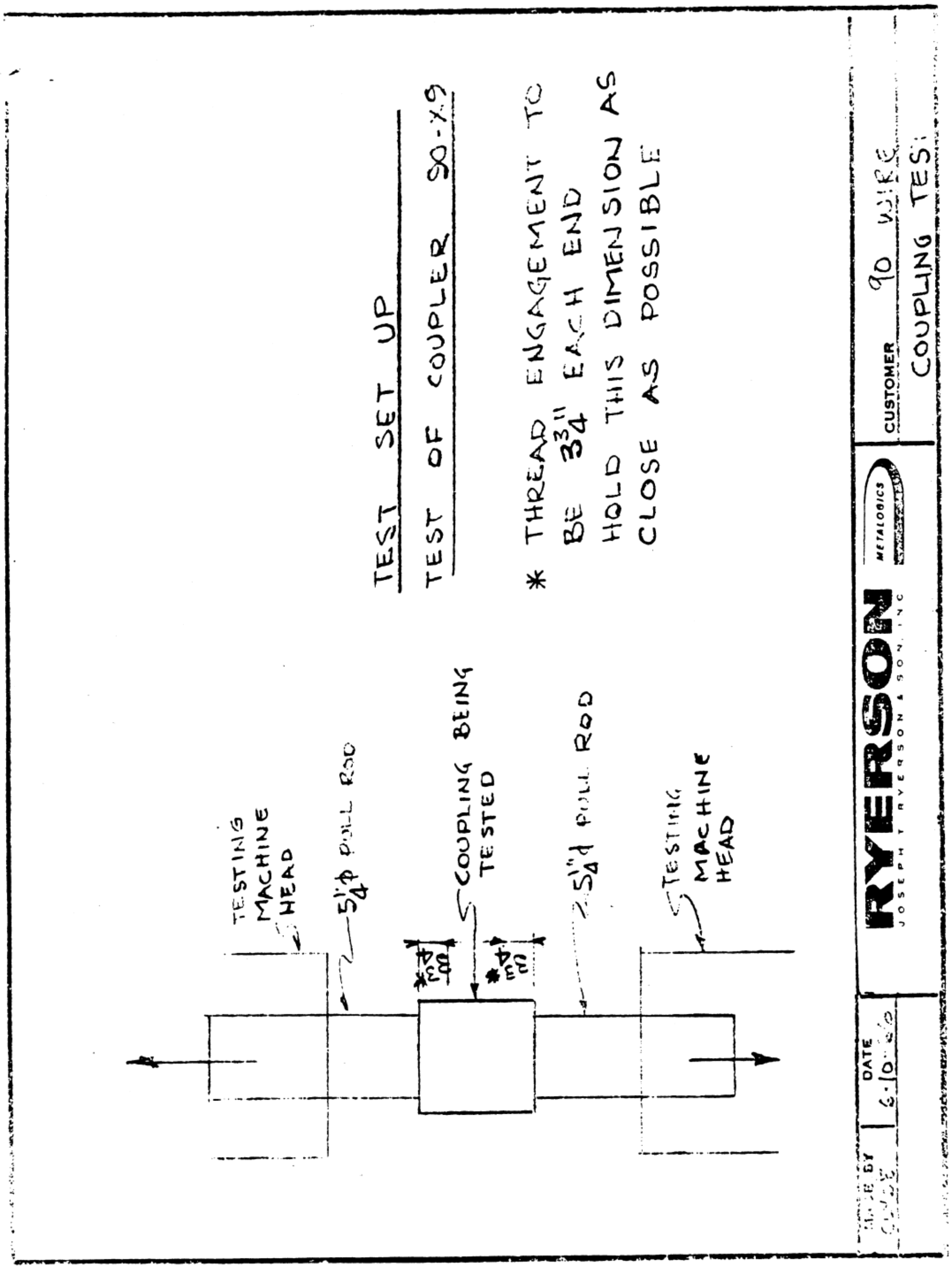
REASSEMBLE AND REPEAT AT NEXT HIGHER LOAD

LOAD TABLE

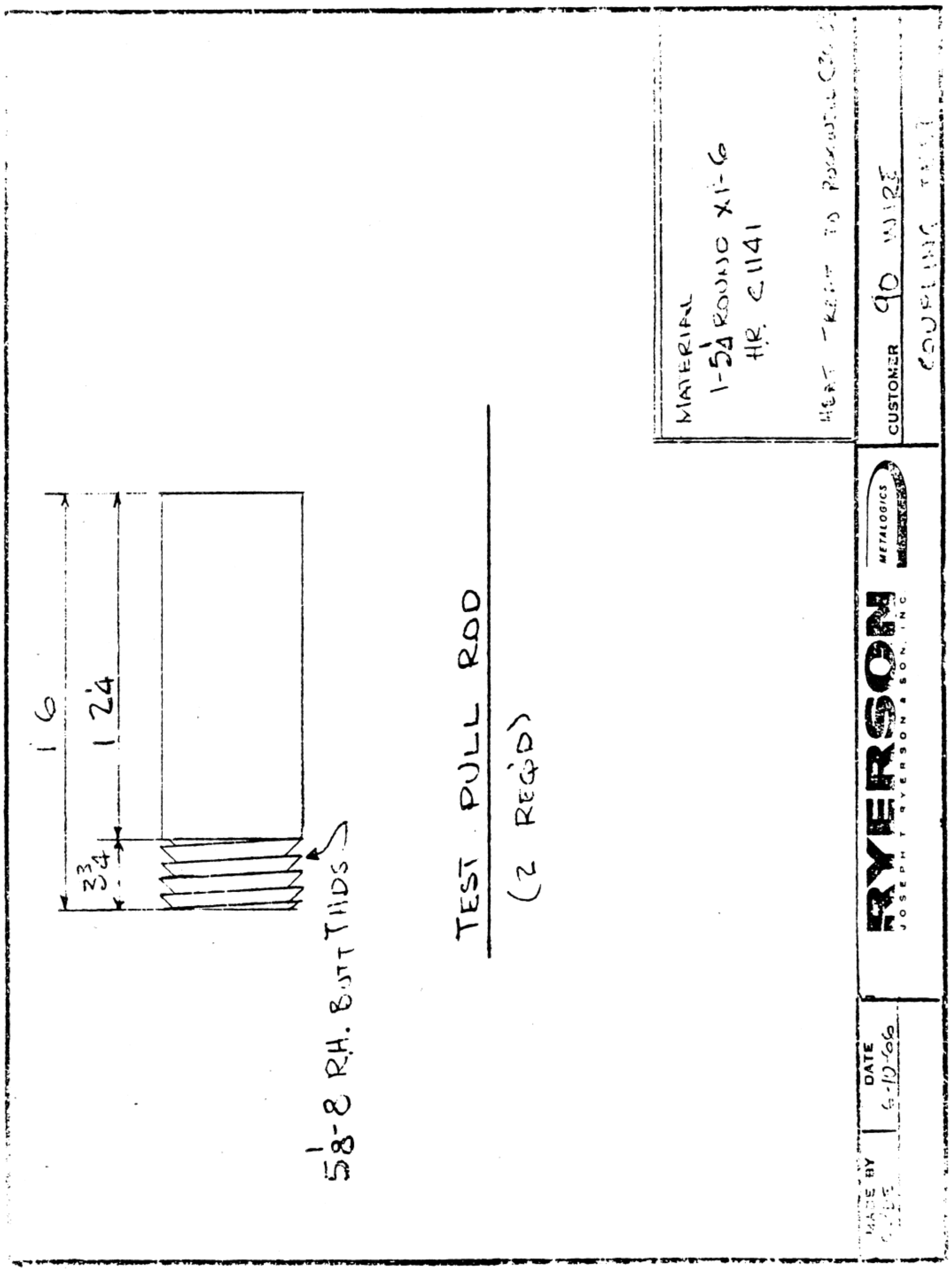
742,000 lbs
848,000 "
954,000 "
1,007,000 "
1,060,000 "
MACHINE MAXIMUM

MADE BY	DATE	RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS	CUSTOMER 90 WIRE COUPLING TEST
[illegible]	6-10-[illegible]		

Sheet 2 of Notes



Sheet 3 of Notes



3D-26

Sheet 4 of Notes

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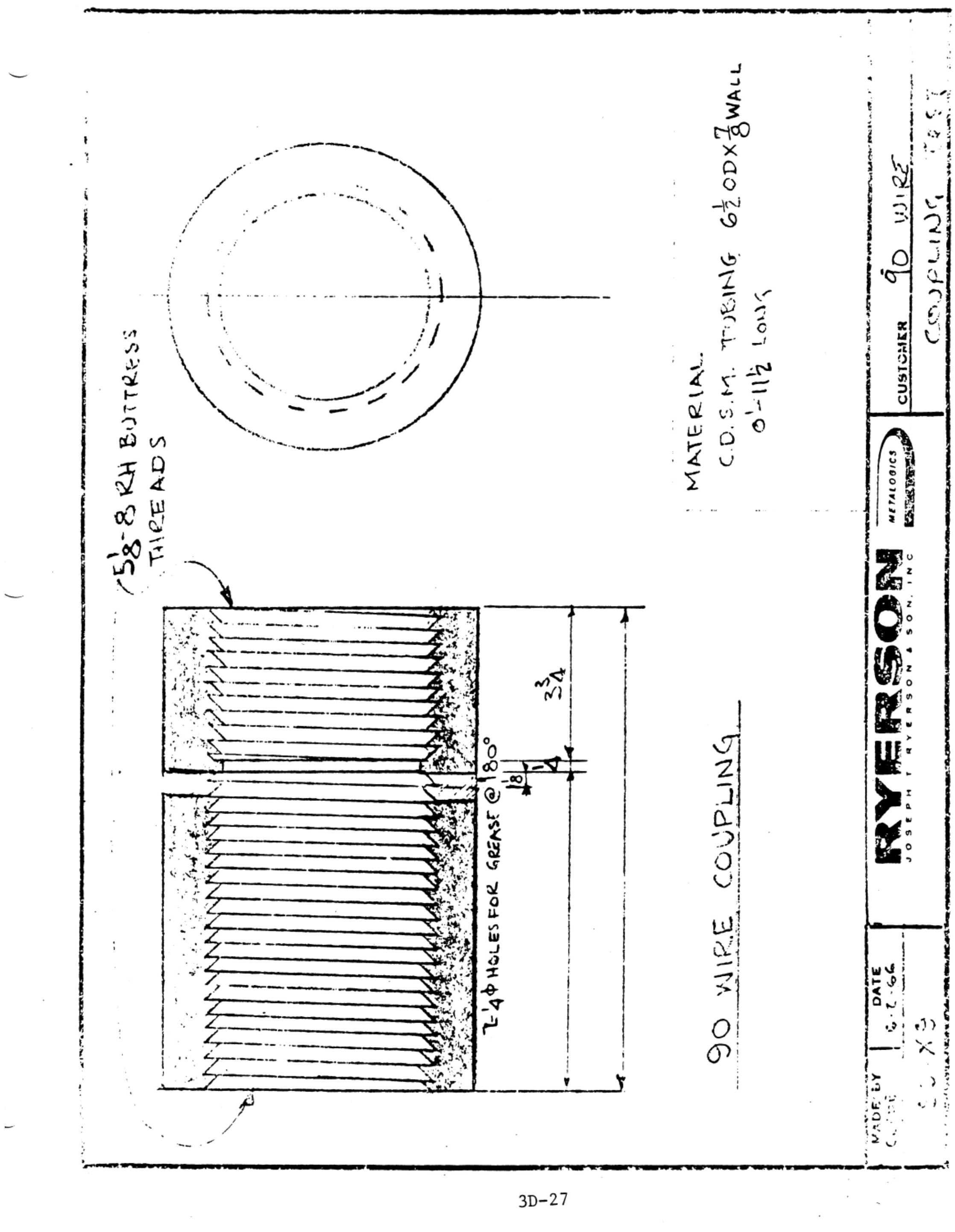


90 Wire Tendon Test

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 655506

ORDER No. PG-18619

June 5, 1937

CLIENT'S No. Ltr. 3/13/67

REPORT

Report of: 90 Wire Tondon Test

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

We received a sample which was identified to us as a 90 wire tondon. We were requested to test the sample in tension measuring alongation over a 120" gage length.

The sample consisted of 90 wires, 1/4" in diameter, with anchor heads on each end. The anchor heads were held on the wire by the wire button heads. The anchor head had external threads which threaded into a coupler. The coupler then threaded onto pull rods, 8" in diameter, which were installed in the upper and lower cross heads of our 1,200,000# testing machine.

An extensometer, modified to give a 120" gage length, was used to record sufficient data to plot the attached curve.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Client
1-PTL Chicago

3D-28

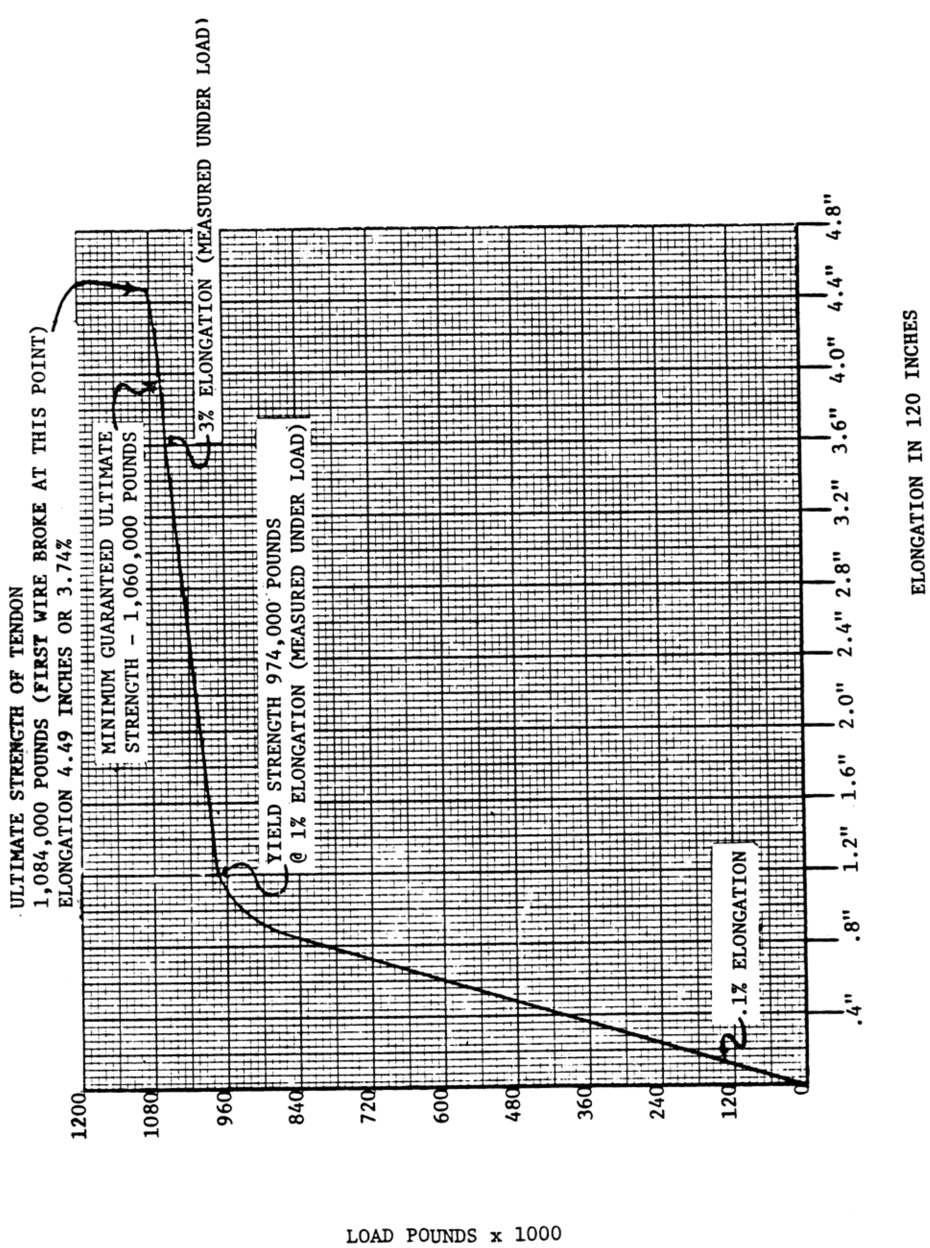
90 Wire Tendon Test

GINNA/UFSAR

TENSION TEST OF 90 WIRE TENDON

J.T. RYERSON & SONS, INC. PG-18619

5-26-67 65506



3D-29

90 Wire Tendon Test

GINNA/UFSAR

90 WIRE TENDON TEST

The purpose of this Test is to verify, that a Tendon, consisting of 90 wires, the wires anchored at each end in anchorheads by means of Buttonheads, is 90 times as strong as one wire. The Test further allows to measure the Tendon-elongation.

The complete Endanchors have been previously tested beyond the ultimate strength of the Tendon.

(see Ryerson 90-PT-1 dated 7/25/66
90-PT-2 7/25/66

and the corresponding Test report from PTL dated 10/24/66.)

CUSTOMER Bechtel Company

Palisades 5935-C-51

RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS

MADE BY A.W.S. DATE 2/3/67

21 PT-34-114

3D-30

Sheet 2 of Notes

GINNA/UFSAR

A. LOAD

Min. guarantied ultimate strength of ϕ 1/4" wire (see ASTM - 421)

240'000 psi

Min guarantied ultimate strength of 90 wire Tendon

90 · 0,04909 · 240'000 = 1'060'000 #

Min. Yield strength of 90 wire Tendon, measured under Load at 1,0% extension.

80% × ult. strength = 848'000 #

Anticipated Test Result:
No wirebreak will occur before the Load of 1060K is reached.

B. ELONGATION

Min. Tendon elongation 3%
measured under Load
in min. Gauge length of 10 ft
(See PCI, proposed Post-tensioning Material Specifications)

CUSTOMER Bechtel Company
Palisades 5935-C-31

METALOGICS

RYERSON
JOSEPH T. RYERSON & SON, INC.

DATE 2/3/67

MADE BY AWS

Sheet 3 of Notes

GINNA/UFSAR

The Elongation is to be measured as movement between Anchorheads.

The Wirelength for the Test tendon is 10'-0" → 120"

The methode of measuring elongation shall be similar to the one specified in ASTM -421.

Initial elongation 0.1% → 0.12" → 1/8"
Initial Stress 29'000 psi → 128 K

Yield at 1% extension → 1,20" → 1 3/16"
min yield strength 848 K

Min. Elongation 3% → 3,60" → 3 5/8"
is to be reached before the first wire breaks.

CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS
RYERSON JOSEPH T. RYERSON & SON, INC.

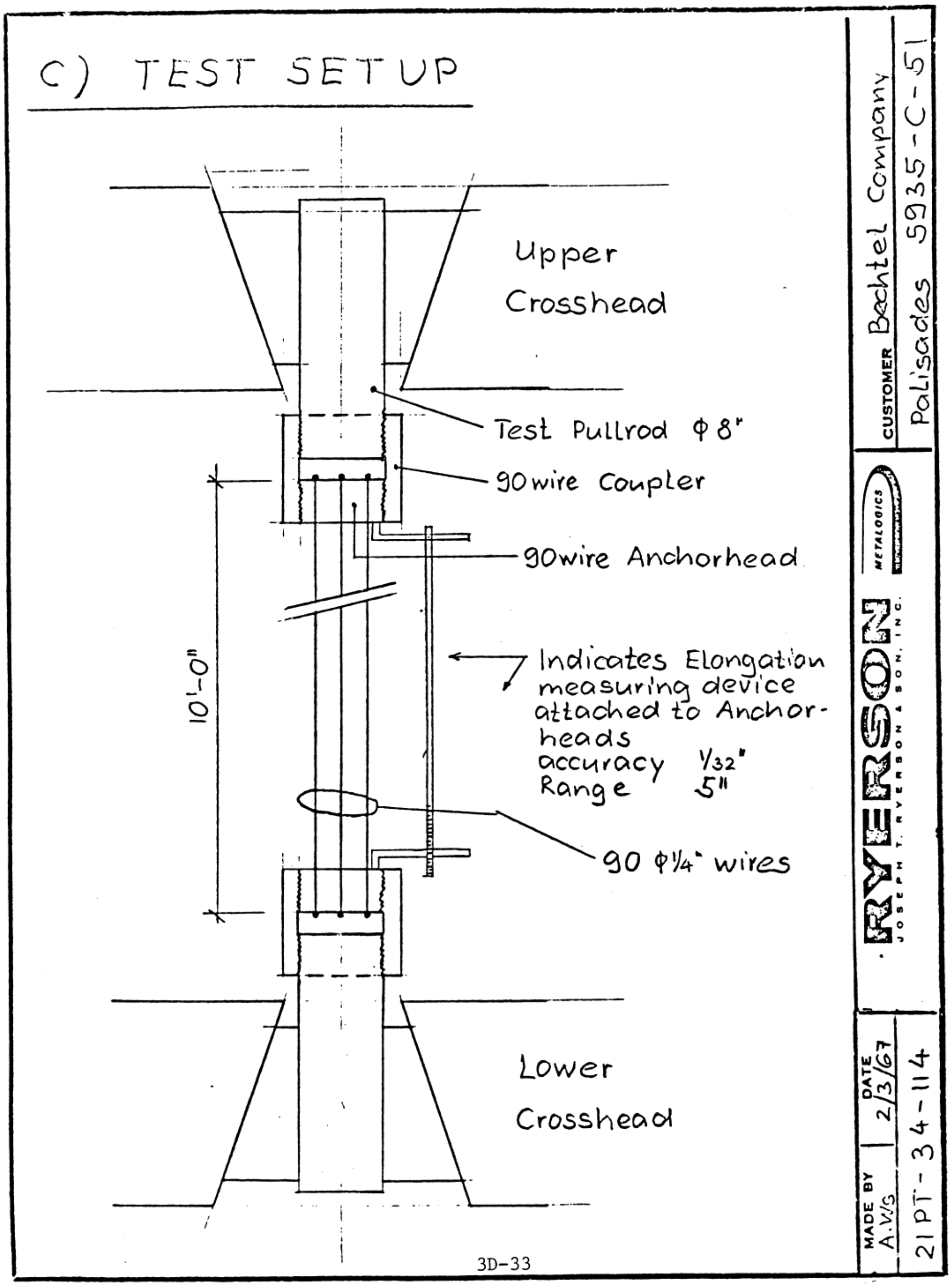
DATE 2/3/67

MADE BY A.W.S.

3D-32

Sheet 4 of Notes

GINNA/UFSAR



Load Tests of 90-X7 Coupler

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 640730

CLIENT'S No. 21T341891-48

August 25, 1966

ORDER No. CH-9583

REPORT

Report of: Load Tests of 90-X7 Coupler

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

Attention: Mr. W. A. Corson

Submitted to our laboratory for load tests was an assembly identified as 90-X7 coupler. We were instructed to set up the coupler assembly as shown on Ryerson drawing that showed the coupler that measured 10-1/2" O.D., 8" long, with a 7-7/8"-8 buttress thread and two 8" diameter pulling rods at either end threaded into the coupler. The thread engagement at each end was 3-1/2". After the assembly was complete, we were to apply designated tension loads and release the loads accordingly. After releasing the loads we were to disassemble the assembly and check threads at both ends of the coupler for visible defects and check whether or not the pulling rods would turn easily or with difficulty from the coupler.

The results of these tests are as follows:

Load Lbs.	Remarks
	Threads lubricated with oil.
142,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
848,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
954,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
1,007,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod. Approximately 3 turns, strap wrenches used from then on. Evidence of thread cutting on rod. Threads on bottom rod dressed with file. Threads lubricated with "Fluoro Glide" dry lubricant.

Sheet 2 of Report

GINNA/UFSAR

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

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LABORATORY No. 640730

ORDER No. CH-9583

August 25, 1966

CLIENT'S No. 21T341891-48

REPORT

Load Lbs.	Remarks
1,060,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
1,200,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Client
Attn: Mr. W. A. Corson
1-PTL Chicago

3D-35

APPENDIX 3E

CONTAINMENT LINER INSULATION PREOPERATIONAL TESTS

by JOHNS-MANVILLE SALES CORPORATION

BM Containment Insulation SP-5290 Ginna Plant

JOHNS-MANVILLE
SALES CORPORATION
INDUSTRIAL INSULATIONS DIVISION
22 EAST 40th STREET • NEW YORK, N. Y. 10016 • TELEPHONE: 532-7600 • AREA CODE 212

JM

December 22, 1967

Gilbert Associates, Inc.
525 Lancaster Avenue
Reading, Pa. 19603

Attention: Mr. K. T. Momose

Re: BM Containment Insulation
SP-5290 Ginna Plant

Dear Mr. Momose:

On November 29, at your request Mr. E. D. Cox sent to your attention the following reports:

Report E 455-T-258 Vinylcel - Resistance to Flame Exposure

Report E 455-T-266 Vinylcel (4pcf) Effect of Heat and Pressure

Subsequent to this you requested engineering data on the 4 pcf Vinylcel similar to that previously furnished for 6 pcf Vinylcel. This is as follows:

2:07.2 Based on pressure cycling tests of nominal 6 pcf Vinylcel (Report E 455-T-238) and the relative elastic moduli of 6 pcf and 4 pcf Vinylcel, we estimate the maximum deflection of 4 pcf Vinylcel to be 2.8% and the residual deformation to be 0.8%.

3:01.2

a. Thermal conductivity (BTU/hr sq ft/F/in) per ASTM C-518 Heat Flow Meter calibrated per ASTM C-177 Guarded Hot Plate.

Mean Temperature, F.	75	100	125	150
BTU-in	0.22	0.23	0.25	0.27

b. Compressive yield strength per ASTM D1621 - 140 psi at the 0.2 percent point on stress-strain curve.

c. Maximum operating temperature for continuous service - 175F, but may vary with specific application requirements.

d. Maximum allowable temperature for specified time interval. See attached Report No. E455-T-266, Appendix I, Compression Under Combined Heat and Pressure Test.

3E-1

Sheet 2 of Cover Letter

e. Moisture vapor permeability per ASTM C-355. See attached Report No. E455-T-268, Appendix I, Table 3.

f. Shear strength per ASTM C-273 - 68 psi ultimate.

g. Shear modulus per ASTM C-273 - 3510 psi.

h. Compressive modulus per ASTM D-1621 - 2300 psi.

i. Density per ASTM D-1622 - 4.0 lbs/cu ft. nominal, 3.7 lbs/cu ft. minimum.

j. Average coefficient of linear expansion - 9.4×10^{-6} in/in/F.

k. Curves for the Case III showing temperature before and after accident plotted against time. See Report No. E 455-T-266, Analogue Study of Vinylcel used as Containment Insulation.

l. Test results of permeability tests per ASTM C-355. See attached Report E 455-T-268,

Predicted curve for 6 month test as required under 2:07.9. See attached Report No. E455-T-268. Dimensional rather than weight change is given as explained under Humid Aging (Results) of the report.

m. Radiation exposure of 8×10^6 roentgens within 6 hours will not change the physical properties of Vinylcel significantly but 10^8 roentgens within 10 hours will cause some progressive deterioration.

The 4 pcf Vinylcel will be supplied 44" x 84" x 1-1/4" thick. Length and width tolerance will be ± 1/32".

Very truly yours,

C. E. ERNST
Chief Engineer

CEE/ca

P.S. As I advised your secretary on Wednesday, Research is sending 6 copies of report E455-T-238 directly to you.

Report No. E455-T-268, VINYLCEL (4 pcf) - Water Vapor Permeability and Humid Aging Tests

GINNA/UFSAR

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-268

Date December 20, 1967

Title: VINYLCEL (4 pcf) - Water Vapor Permeability and Humid Aging Tests

SUMMARY

VINYLCEL of 4 pcf nominal density has been tested for water vapor permeability and for dimensional changes under high humidity conditions.

The water vapor permeability of 1-in. thick specimens was 0.06 perm-in. at 3.2 pcf and 0.04 at 4.9 pcf density; both values are very low.

In 6 months at 120F, 100 per cent RH, the volume change was only 1.2 per cent, and length and width changes only 0.3 per cent.

Contents: Summary, Discussion, Results, and Appendixes.

Reported by E. J. Davis

E. J. Davis
Materials Evaluation Section

3E-3

Sheet 2 of Report No. E455-T-268

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JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-268
Page 1

DISCUSSION

Test Methods:

Density - ASTM C303

Water Vapor Permeability - ASTM C355 (Desiccant Method)

Humid Aging - Specimens 4 x 4 x 0.65 in. were measured to ±0.001 in. in all dimensions and placed above the water level in a glass vessel containing water. The vessel was closed and placed in an oven (circulating type) which was controlled at 120 ±3F. The specimens were removed periodically and measured.

Results:

Water Vapor Permeability (Table 1) - When tested at 1-in. thickness according to the ASTM C355 desiccant method, VINYLCEL at 3.2 pcf density had a permeability of 0.06, and at 4.9 pcf density of 0.04 perm-in.

Humid Aging (Figure 1) - These tests of VINYLCEL have been conducted for 6 months at 120F, 100 per cent RH. Initially there was a slight expansion (about +0.2 per cent), both linearly and volumetrically. With increasing time, the specimens began to shrink; the shrinkage levelled off at about -0.3 per cent (average, length and width) and -1.2 per cent (volume).

Sheet 3 of Report No. E455-T-268

APPENDIX I

TABLE 1. VINYLCEL, 4 PCF NOMINAL DENSITY, 1-IN. THICK

	Actual Test Density	
	3.2 pcf	4.9 pcf
Water Vapor Permeability, (perm-in.) by: ASTM C355, Desiccant Method	0.06	0.04

Temperature: 90F

Relative Humidity: 52%

Vapor Pressure Difference: 0.73 in. Hg

Test Area: 100 sq in.

Duration of Test: 8 days

Sheet 4 of Report No. E455-T-268

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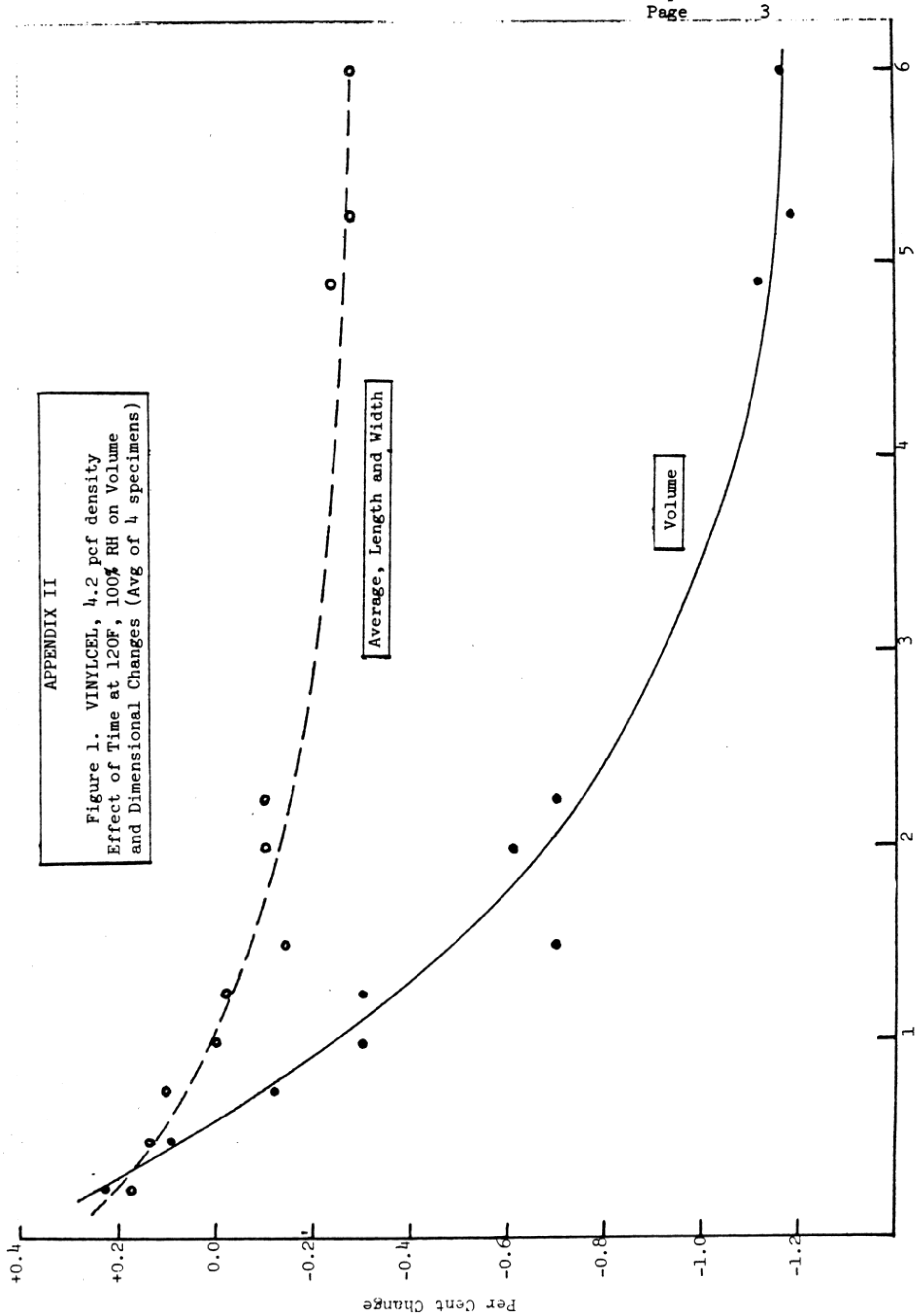
Report No. E455-T-268
Page 3



3E-6

Report No. E455-T-266, VINYLCEL (4 pcf) - Effect of Heat and Pressure

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-266

Date November 3, 1967

Title: VINYLCEL (4 pcf) - Effect of Heat and Pressure

Requested by: C. E. Ernst

SUMMARY

VINYLCEL of 4 pcf nominal density, 1-1/4-in. thick, has been subjected to a combined heat and compression test to simulate an "incident" in a nuclear reactor containment vessel.

Two tests, according to Gilbert Associates' SP-5920 (Case III), resulted in thickness decreases of 37 per cent and 38 per cent at the critical time of 5-1/2 minutes; the corresponding permanent thickness losses were 29 per cent and 22 per cent.

A network analog simulation of the insulation system under Case III conditions showed that after 5-1/2 minutes the temperature rise of the steel liner was 1F.

Contents: Summary, Discussion, and Appendixes.

Reported by E. J. Davis
E. J. Davis
Materials Evaluation Section

W. F. Gulick
W. F. Gulick
Basic Physics Research Section

rs

Sheet 2 of Report No. E455-T-266

DISCUSSION

Test Method:

A 4 x 4-in. electrically heated hot plate was used with a compression testing machine. The 4 x 4-in. specimen of VINYLCEL, with a thermocouple in a groove on its hot side, was placed on the hot plate; its temperature was read with a direct reading potentiometer. The temperature of the hot plate was raised by means of a variable resistance in series with it. As the specimen was simultaneously heated and loaded by the testing machine, its deflection was measured with a dial gage, accurate to 0.001 in., mounted in a compression rig.

The pressure and temperature conditions of Gilbert Associates Specification SP-5290 (November 30, 1966), Case III, were followed as closely as possible.

Results:

Two tests were run (see Table). In the first, the pressure curve was followed very closely. The temperature lagged by as much as 98F at 30 seconds, but had caught up at 3 minutes; it was then over the desired temperature, by as much as 29F, for the next 7 minutes. At the critical time of maximum pressure (5-1/2 minutes) the specimen had deflected approximately 37 per cent. The maximum deflection was 58.5 per cent at 10 minutes, after which the sample began to regain its thickness and the test was ended. After the test, the permanent thickness loss was about 29 per cent.

A second test was run, because of the high temperatures encountered in the first test. This time the test temperature started a little higher than desired, lagged by as much as 88F at 30 seconds, and reached the desired temperature at 4 minutes; thereafter it remained close to the desired curve.

At 5-1/2 minutes, the specimen had compressed about 38 per cent; the maximum was 49 per cent at 20 minutes, and the permanent loss of thickness about 22 per cent.

Sheet 3 of Report No. E455-T-266

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-266
Page 2

APPENDIX I

VINYLCEL, 1-1/4 in. THICK, 4 pcf NOMINAL DENSITY
Compression Under Combined Heat and Pressure Test
(Gilbert Associates, Case III Conditions)

Time	Desired Conditions Case III		Test A (1.251-in. thick)				Test B (1.248-in. thick)			
	Pressure psi	Temp. °F	Pressure psi	Temp. °F	Sample Deformation in.	Sample Deformation Per Cent	Pressure psi	Temp. °F	Sample Deformation in.	Sample Deformation Per Cent
1 sec.	<20	155	0	156	0	0	1.2	166	0	0
5 sec.	77	205	77	160	0.046	3.7	77	--	0.053	4.2
30 sec.	77	268	77	170	0.057	4.6	77	180	0.065	5.2
1 min.	79	273	79	194	0.103	8.2	79	194	0.118	9.4
2	82	282	82	251	0.191	15.3	82	248	0.193	15.4
3	84	290	84.5	288	0.257	20.6	84	276	0.245	19.6
4	87	295	87	315	0.352	28.2	87	292	0.338	27.0
5	89	302	89	330	0.390	31.2	89	300	0.428	34.2
5-1/2	90 (max)	305	90	334	0.460	36.8	90	304	0.479	38.3
6	89	306	88	332	0.492	39.4	89	306	0.502	40.2
7	87	308	87	330	0.555	44.4	87	312	0.542	43.4
10	82	310 (max)	82	313	0.732	58.5	82	312	0.611	48.9
20	68	305	68	298	0.713	57.0	69	306	0.612	49.0
30	55	298	(ended)				56	297	0.588	47.0
			(after test thickness, 0.890 in.; loss, 0.361 in., or 28.9 per cent)				(ended) (after test thickness, 0.928 in.; loss, 0.280 in., or 22.4 per cent)			

3E-9

Sheet 4 of Report No. E455-T-266

APPENDIX II

ANALOG STUDY OF VINYLCEL USED AS CONTAINMENT INSULATION

The transient rise in temperature of the insulated cross-section of a nuclear reactor has been measured, based on a temperature profile at the hot surface (hypothetical incident) provided by Gilbert Associates. The measurements were made on an electrical network analog set up to simulate the insulation system.

The cross-section consisted of a VINYLCEL (PVC foam) layer, a steel liner, and a surrounding concrete barrier. The following properties of the layers were assumed:

	VINYLCEL	Steel	Concrete
Thermal conductivity, $\frac{\text{Btu - in.}}{\text{hr - sq ft - F}}$	0.25	312.0	10.0
Specific heat, $\frac{\text{Btu}}{\text{lb - F}}$	0.3	0.106	0.21
Density, lb/ft^3	6.67	480	140
Thickness, in.	0.75*	0.375	42

*Compressed thickness. Uncompressed thickness was 1.25 in.

From these parameters the thermal resistances and capacitances of the system were computed:

	VINYLCEL	Steel	Concrete
Resistance, sec - ft^2 - F/Btu	10,800	4.3	15,100
Capacitance, Btu/ft^2 - F	0.125	1.6	103

Air to surface resistances at the hot and cold surfaces were assumed to be 180 and 900 sec - ft^2 - F/Btu.

Although the hypothetical incident lasts several hours, the temperature rise of the steel liner after 5-1/2 minutes was the information sought. The hot surface temperature profile (Case III) near the beginning of the incident was therefore programmed in detail. The 2-minute running time of the electrical analog was set equal to 16 minutes of thermal time. The VINYLCEL insulation layer was represented by six

Sheet 5 of Report No. E455-T-266

network sections, the steel layer by one section, and the concrete barrier by twenty sections resistively and by the first five sections capacitatively.* The analog was allowed to come into equilibrium with 120F on the hot face and -10F on the cold face.

Curves of temperature vs time for the hot surface of the VINYLCEL insulation, the mid-point of the insulation, and the steel liner are shown in Figure 1.

To obtain a more conservative measure of the temperature rise at the steel liner, an analog run was made in which the hot surface temperature at the start of the incident was raised immediately to 310F (the peak temperature) and held there for 16 (thermal) minutes, the duration of the run. The curves for this experiment are shown in Figure 2.

After 5-1/2 minutes, the temperature rise of the steel liner was 1F (Figure 1) and 1-1/2F (Figure 2).

*Only enough capacitance was available to fill the first five out of twenty network sections. Tests showed, however, that in 16 minutes the temperature wave had barely penetrated into the concrete.

Sheet 6 of Report No. E455-T-266

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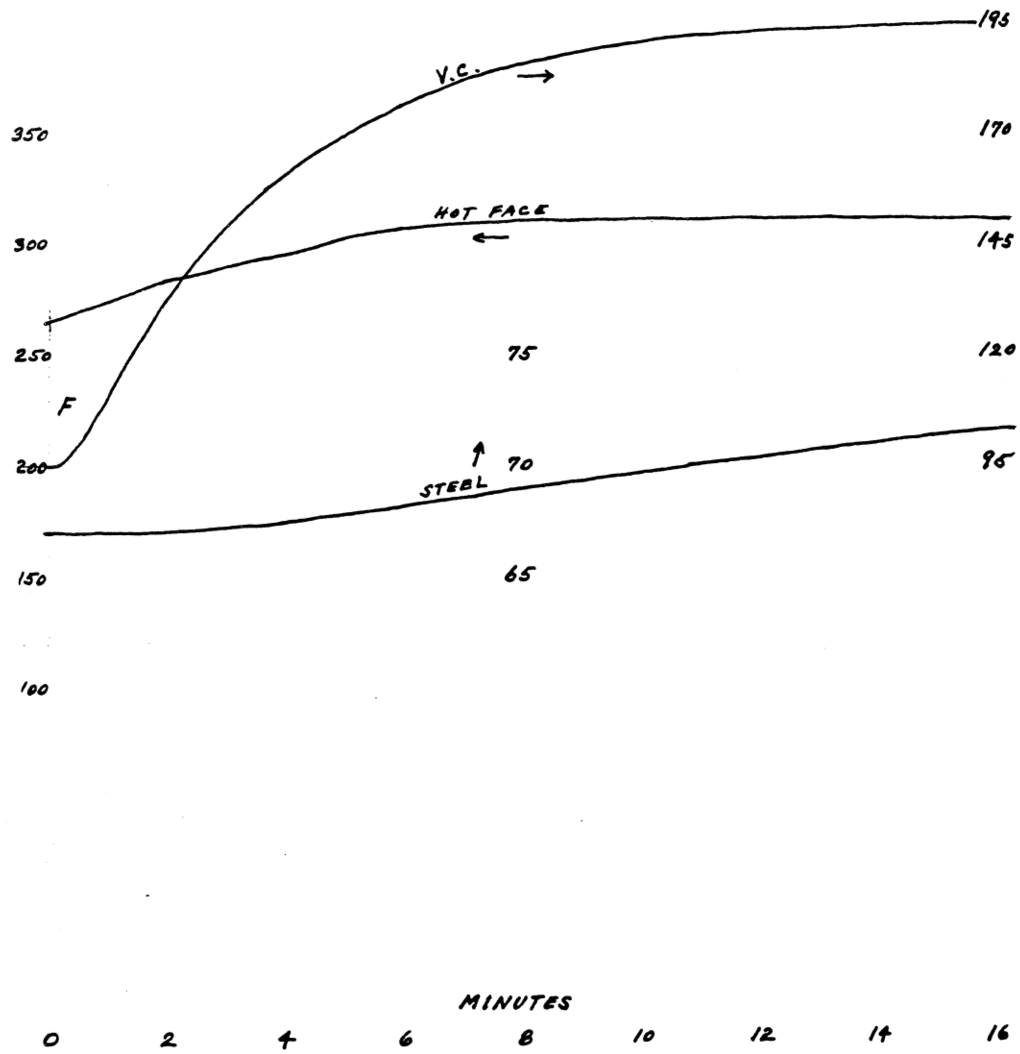
FIG. 1

Report E455-T-266
Page 5

1.25" VINYLCEL COMPRESSED TO 0.75"



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Sheet 7 of Report No. E455-T-266

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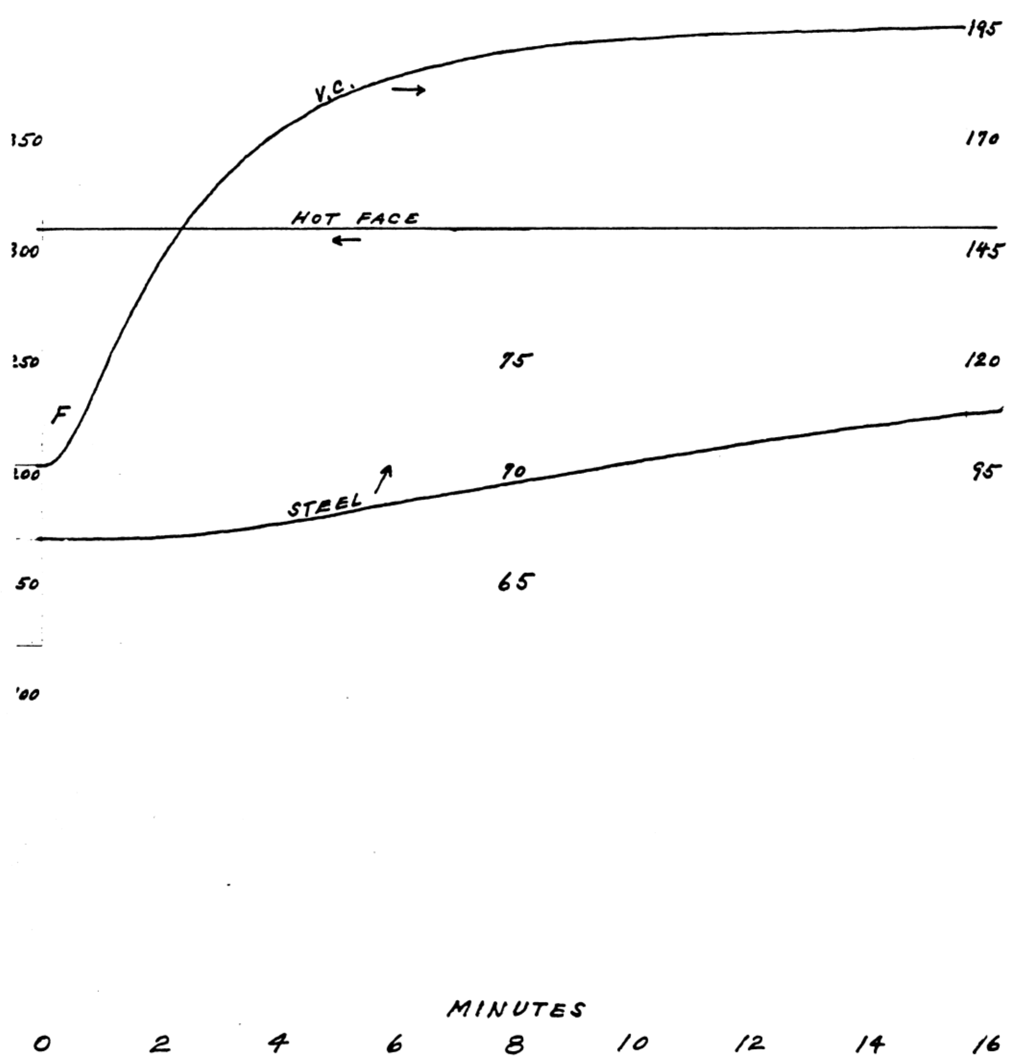
FIG. 2

Report E455-T-266

1.25" VINYLCEL COMPRESSED TO 0.75"



Report No. E455-T-258, VINYLCEL - Resistance to Flame Exposure

GINNA/UFSAR

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-258

Date September 21, 1967

Title: VINYLCEL - Resistance to Flame Exposure

SUMMARY

Johns-Manville VINYLCEL foam has been subjected to fire exposure to simulate conditions which might occur during the construction period or during operation when used as insulation for containment shells in nuclear generating plants. The tests were designed and conducted to answer questions raised by the Nuclear Energy Property Insurance Association.

The tests included flame exposures of VINYLCEL, plain, and faced with 24-gage steel, as installed against steel plate; surface burning characteristics of plain and faced sheets as measured by the ASTM E 84-61 Tunnel Test; and the fire resistance characteristics of VINYLCEL faced with 1/2-inch MARINITE 36 when exposed to 30 minutes of the standard time-temperature curve.

Test results obtained on the Bureau of Mines Flame Penetration Test and results of a Thermogravimetric Analysis are also presented.

The test results indicate:

1. VINYLCEL is a product with good flame resistance, low combustibility, and very low surface flame spread.
2. The release of combustible gases is negligible. Weight loss occurs at temperatures in the ranges of 460°F to 572°F and from 572°F to 1112°F. Weight loss of 8 per cent is recorded at 460°F and increases to 38 per cent at 572°F while 1112°F is required to reach a weight loss of 95 per cent.
3. Facing VINYLCEL with 24-gage steel provides improvement in flame resistance. That protection, or facing with 1/2-inch MARINITE 36, greatly reduced heat flow through the construction.

These tests demonstrate that VINYLCEL will offer significant protection against fire exposure. It will not propagate fire nor suffer damage beyond the area of exposure. Flammable gases will not be emitted nor are the gases emitted an explosion hazard.

Contents: Summary, Discussion, Appendix I (Tables 1 & 2), Appendix II (Figures 1-3), and Appendix III (Photographs).

Reported by E. J. Davis
E. J. Davis
Materials Evaluation Section

K. N. Smith
K. N. Smith
Materials Evaluation Section

R. H. Neisel
R. H. Neisel, P.E.
Materials Evaluation Section
N. J. License No. 9316

Sheet 2 of Report No. E455-T-258

DISCUSSION

Material

VINYLCEL of nominal 4-pcf density, 1-1/4 and 3/4-in. thick, was supplied by the VINYLCEL Production Department on September 6, 1967, and September 11, 1967. Other materials were taken from laboratory stocks.

Witnesses

The Building Fire tests, Tunnel tests, and the Vertical Panel Fire test were witnessed by the following:

R. M. L. Russell	-	Factory Insurance Association
P. H. Dobson	-	Factory Mutual Engineering Corporation
R. R. Koprowski	-	Rochester Gas & Electric Company
R. S. Brown	-	Ebasco Services, Incorporated
L. F. Picone	-	Westinghouse Electric
P. Mitchell	-	Westinghouse Electric
K. T. Momose	-	Gilbert Associates, Incorporated

Test Methods

Building Fire Tests: A concrete block building, 16 x 8 x 8 ft high, was used. (See Photograph A). To its back wall (8 x 8 ft) were bolted two 4 x 8-ft x 3/8-in. steel plates, long dimensions vertical. The joint was sealed with JM No. 450 Insulating Cement. On the exposed surface of these plates were welded 1/8-in. x 2-in. long securement pins, 24-in. on center. Sheets of 4-pcf VINYLCEL, 1-1/4-in. thick, were impaled on the pins; a full sheet (84 x 42-in.) at the top center with long dimension vertical, and cut sheets at the sides and bottom. For Test "A" the VINYLCEL was covered with 24-gage, 4 x 8-ft galvanized steel sheets, also impaled on the pins, with the long dimension horizontal and a 1-in. overlap between the two sheets. For Test "B" the VINYLCEL was left uncovered. In both cases, speed clips were placed over the pins to retain the sheets.

The ceiling was covered with 1-1/4-in. thick VINYLCEL, screwed to the 1/2-in. MARINITE overhead. Poultry wire was secured under the VINYLCEL. The same ceiling insulation was used for both tests. A draft shield of FLEXBOARD, 2-ft deep extended down form the ceiling across the 8-ft width of the room, 8 ft away from the test wall.

Nine chromel-alumel thermocouples were used in each test. Four were between the VINYLCEL and the 3/8-in. steel wall plate, at levels 1, 3, 5, and 7 ft from the floor, and alternated about 9 in. from the vertical center line of the wall. Four were on the exposed surface, at the center line, and at the same levels. One was 3 in. below the ceiling, directly above the center of the fire source. All couples were connected to a switch and a direct reading potentiometer.

The fire source was a steel bucket, 11 in. in diameter and 13 in. deep (area 0.66 sq ft) placed in a hole 6 in. deep in the floor and 2 in. from the center of the test wall. A 6-in. depth of water was in the bottom of [illegible], and a sufficient quantity of heptane was floated on the water to ensure that the fire would last at

Sheet 3 of Report No. E455-T-258

least 30 minutes. The surface of the heptane was about 6 in. above floor level. The peak burning rate of the heptane is estimated as greater than 10,000 Btu per minute.

The heptane was ignited, and readings of the nine thermocouples were taken as rapidly as possible. Notes were made as to flaming, smoking, cracking, or bowing of the test specimens. In addition, during Test "B" of the plain VINYLCEL, a propane torch was used in attempts to ignite the gases collected behind the draft shield.

After the tests, the condition of the VINYLCEL was noted, and the weight losses of the center sheets of VINYLCEL were calculated.

Vertical Panel Fire Test: The composite panel (4 x 8 ft) was assembled as follows: 1/2-in. thick MARINITE 36, 3/4-in. (0.71 in.), 4-pcf VINYLCEL and 3/8-in. steel plate. (The MARINITE was exposed to the fire). The panel was held in place in the furnace buck with MARINITE strips on the furnace side and with bolts and steel fixtures supporting the steel plate on the cold face.

Six thermocouples were placed as follows: two between the MARINITE and VINYLCEL, two between the VINYLCEL and steel plate, and two under asbestos felt pads on the room side of the steel plate. They were placed on the vertical center line of the composite panel, 3 ft and 5 ft from the top. One 7 x 3-1/2-ft panel of VINYLCEL and 4 x 1-ft and 7 x 1/2-ft filler pieces were used.

The test was continued for 30 minutes with the furnace temperature controlled to coincide with the standard time-temperature curve as given in ASTM E 119.

Tunnel Fire Test(s): Two tunnel tests were conducted as prescribed by ASTM E 84. In the first test, the 1-1/4-in. thick, 4-pound per cubic foot VINYLCEL in 7-ft lengths was placed in the tunnel directly exposed to the flame. For the second test, the VINYLCEL was laid on 24-gage galvanized sheet metal (the flame impinged on the sheet metal).

Bureau of Mines Flame Penetration Test: This test uses 6-in. square x 1-in. thick specimens of foam and a propane torch with a pencil-flame burner head with its brass fuel orifice replaced by a steel fuel orifice from a blowtorch head.

The specimen is backed by an 8-in. square x 1/4-in. thick piece of TRANSITE with a 1-1/2-in. diameter hole in its center. A piece of filter paper is placed between the specimen and the TRANSITE.

The burner is first adjusted to produce a temperature of 1910°F to 1960°F, at 2 in. beyond the burner head, and a 3-1/4-in. visible flame. The burner is then placed 1 in. from the vertical assembly of specimen, filter paper, and TRANSITE. The foam specimen is adjudged adequate in resistance to flame penetration if during a 10-minute test flame does not "penetrate the foam or ignite the filter paper." Charring or discoloration of the paper is disregarded.

This test was performed with and without 24-gage galvanized steel over the hot side of the VINYLCEL.

Thermogravimetric Analysis (TGA): The material was analyzed by standard TGA procedure. The data were obtained from ignition in a thermobalance, using a heating rate of 8°C (14.4°F) per minute. The air flow in the combustion chamber was adjusted to 0.5 liters per minute. (From Report No. 455-T-142).

Sheet 4 of Report No. E455-T-258

Results

I. Building Fire Tests:

Test "A", with 24-gage galvanized steel over the 4-pcf VINYLCEL, is covered in Table 1, Figure 1, and Photographs A through F. This test produced little external evidence of damage; the galvanized sheets were buckled and blackened where the flames hit them, but stayed intact and tight of joint. There was some smoking of the insulation from under the galvanized sheets (for about 25 minutes) but most of the smoke was from the heptane fuel. Relatively low temperatures (960°F maximum) were developed on the fire side, and the maximum on the 3/8-in. steel was 378°F.

When the galvanized sheets were removed, a charred or scorched area on the VINYLCEL was found, measuring 54 in. high and 27 in. wide. Within this area were several large shrinkage cracks, to a maximum width of 6 to 7 in., which extended through to the steel plate. Damage was confined to the area of flame impingement, and there was little or no spread either vertically or horizontally; the side sheets were virtually untouched. Weight loss of the central sheet was calculated as 6 per cent.

Test "B", with no covering over the VINYLCEL, (Table 2) produced higher temperatures; 1280°F maximum on the fire side, 660°F maximum on the 3/8-in. steel plate. The VINYLCEL flamed (at times) over an area of 2-1/2 x 7 ft, but mostly at the joints between sheets or at the shrinkage cracks. There was smoking from the VINYLCEL, but again most of the smoke was from the heptane fuel. The gases at ceiling level were tested with a torch at intervals and did not ignite.

After the test, the charred or scorched area was 64 in. high x 32 in. wide. Shrinkage cracks within this area had a maximum width of 8 to 10 in. and extended to the steel plate. There was little or no damage outside the area mentioned, and the side sheets were in good condition. Weight loss of the central sheet was calculated at 16 per cent.

II. Vertical Panel Fire Test:

The temperatures recorded at the various locations are graphically given in Appendix II, Figure 2. The large temperature drop through the VINYLCEL should be noted. After 30 minutes' exposure, this drop was 600°F indicating the high degree of retention of insulating value of the VINYLCEL under this severe exposure.

The MARINITE sheet had cracked horizontally near midheight and vertically from the bottom to the horizontal crack at the center line. This occurred after 23 minutes exposure to the fire. Some barely combustible gas (only flickers of flame when exposed to a torch) was emitted during the middle 10 minutes of the test.

Examination of the panel after the test showed that the VINYLCEL had shrunk and was broken into several pieces but remained in place.

Sheet 5 of Report No. E455-T-258

GINNA/UFSAR

III. Tunnel Fire Test(s):

The following results were obtained in the tunnel tests:

	Flame Spread	Fuel Contributed	Smoke Developed
Plain VINYLCEL	15.4	5	253
VINYLCEL - Sheet Metal	5.1	0	34

The flame spread results were for a 3-ft flame advance on the plain VINYLCEL and 1 ft for the sheet metal-faced VINYLCEL. Both results indicated that the insulation may be classified as non-combustible as they are less than 25.

Appendix III presents photographs M, N, and O of the VINYLCEL after the tunnel tests. The improvement due to the sheet metal facing is readily apparent. The extent of the physical degradation of the VINYLCEL can be seen (each piece is 7 ft long and 21 in. wide). The VINYLCEL was severely distressed only where the flame actually impinged on it.

IV. Bureau of Mines Flame Penetration Test:

(A). 4-pcf VINYLCEL, with no protection: Two tests were run. The flames penetrated the 1-in. thick specimens and ignited the paper in 40 and 45 seconds, respectively. The penetration appeared to be more because of heat shrinkage than by burning.

(B). Two tests were also run with 24-gage galvanized steel over the VINYLCEL; both were successful, with no burn-through or ignition of the paper in 10 minutes. The paper showed slight discoloration in one test, none in the other. Behind the steel, there was a saucer-shaped depression in the VINYLCEL about 6 in. in diameter by 1/2 in. deep.

V. Thermogravimetric Analysis (TGA):

The VINYLCEL began to lose weight at 140°C (284°F). When 300°C (572°F) was reached, 38 per cent of the weight had been lost. In the second stage of decomposition, between 300 and 600°C (572 and 1112°F), the specimen lost a total of 94.5 per cent of its weight. A curve of weight loss versus temperature is attached (Figure 3).

A comparison of that curve with the TGA curves for other cellular, low density polyvinyl chloride materials presented in Figure 6 of the report "Thermal Decomposition Products and Burning Characteristics of Some Synthetic Low Density Cellular Materials" by Watson, Stark, et al - Bureau of Mines Investigation No. 4777, shows significant weight loss to occur at a temperature 72°F lower than that for VINYLCEL.

This difference is believed due to the cross-linked structure of VINYLCEL. The Bureau of Mines data showed that hydrogen chloride gas was released at 374°F and the similarity of the curves indicates that the same product is released from VINYLCEL but at a significantly higher temperature (446°F).

Sheet 6 of Report No. E455-T-258

APPENDIX I

Table 1

Building Fire Test of VINYLCEL (Test A)
(8-in. Concrete Block Wall, 3/8-in. Steel Plate, 1-1/4-in. VINYLCEL 4-pcf, 24-ga. Galvanized Steel Sheet)

	Temperatures, °F									
	On 3/8" Steel, Bottom to Top				Galvanized, Bottom to Top				Ceiling	
Time	1	2	3	4	5	6	7	8	9	Notes
0	57	58	58	60	61	63	64	63	63	Fuel Ignited
30 Sec	--	--	--	--	220	144	125	140	207	Flames, 3-5 ft; smoke 5-8 ft
1 Min	--	--	--	--	240	300	210	210	264	
1-1/2	--	--	--	--	320	470	270	230	280	
2	--	--	--	--	344	500	290	240	280	Flames 4 ft, jumping to 6 ft
2-1/2	--	--	--	--	374	600	292	247	308	
3	57	59	59	60	424	660	380	290	300	Galvanized buckled toward VINYLCEL, about 3 ft in diameter
4	57	59	60	60	440	700	372	300	300	
5	57	59	59	60	490	940	460	330	320	Smoke, from under galvanized
6	57	58	59	59	485	840	460	320	310	
7	62	59	59	59	540	900	460	340	330	
8	62	59	60	59	480	800	460	320	330	Buckling deeper, but pins still holding
9	62	61	62	59	490	820	440	320	310	
10	62	67	62	59	520	960	490	360	330	
11	60	68	64	59	560	880	530	360	320	
12	67	--	64	60	540	900	480	350	310	
13	64	--	66	61	525	930	520	362	326	
14	74	--	67	61	520	640	480	350	330	Joint in galvanized still tight
15	87	--	68	62	530	700	410	330	290	
16	79	--	68	61	524	820	430	320	300	
17	78	--	69	63	560	650	400	330	320	
18	83	110	69	62	530	720	420	320	320	
19	88	200	70	63	520	630	430	320	320	Smoking (less) from under galvanized; flames 5-6 ft
20	98	270	70	64	510	720	400	320	300	
21	106	310	70	64	490	620	375	300	295	
22	110	330	70	64	520	800	400	320	310	
23	114	360	72	65	500	710	390	310	285	
24	117	370	70	65	520	630	370	310	300	
25	120	378	70	65	510	580	380	310	290	
26	120	370	72	65	520	760	400	320	300	No smoke from under galvanized; (can be touched 2 ft from fire)
27	122	364	70	65	580	810	410	310	300	
28	120	360	70	65	520	740	390	300	290	
29	122	360	72	66	520	610	300	300	295	
30	125	370	72	66	480	630	360	290	280	
31	127	370	72	66	580	620	350	290	280	
32	128	370	72	67	470	600	345	282	270	
33	128	370	74	68	530	680	375	310	280	Fire extinguished

Sheet 7 of Report No. E455-T-258

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-258

Page 6

APPENDIX I

Table 2

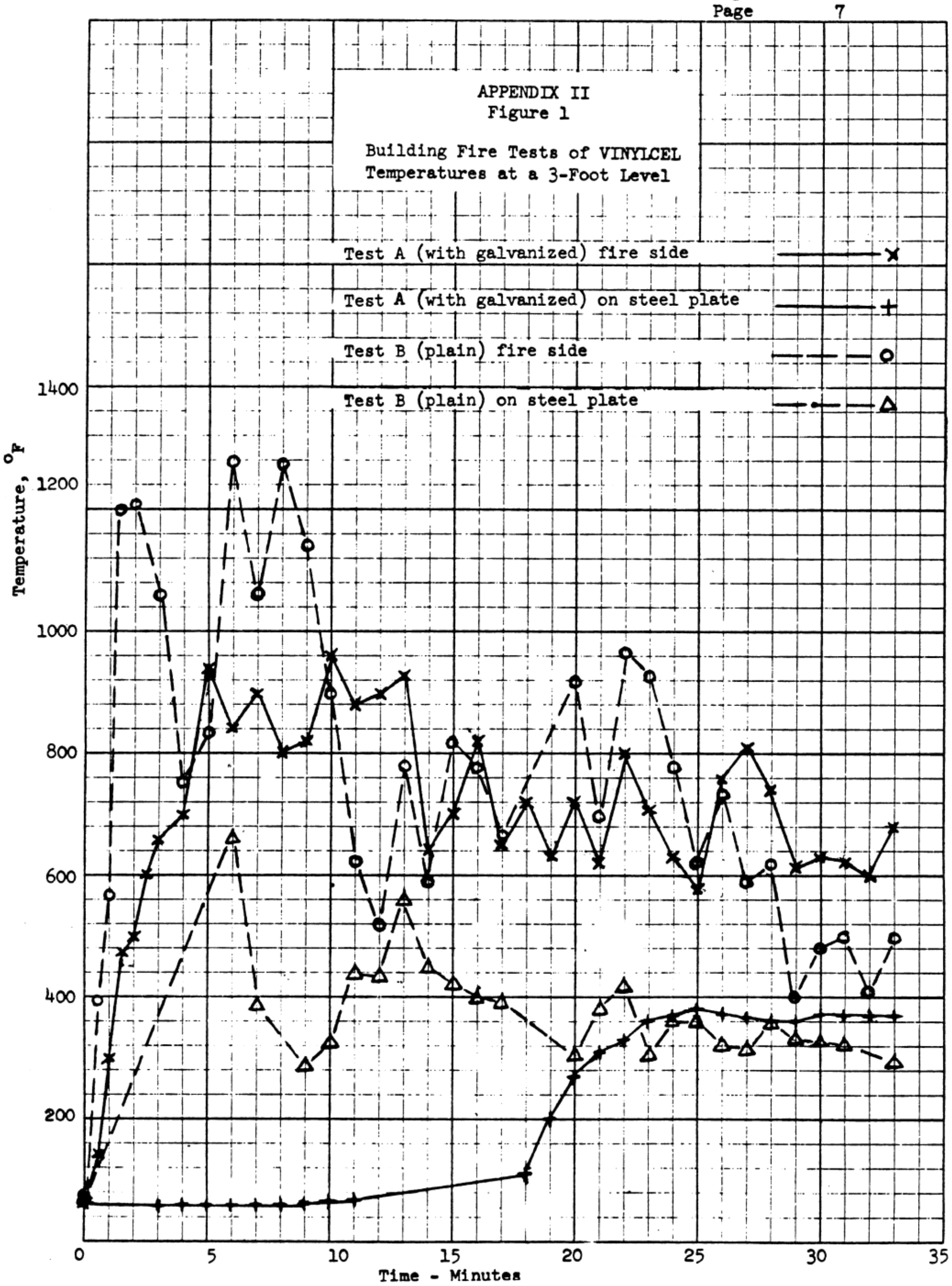
Building Fire Test of VINYLCEL (Test B)

(8 in. Concrete Block Wall, 3/8-in. Steel Plate, 1-1/4-in. VINYLCEL 4-pcf)

	Temperatures, °F									
	On Steel, Bottom to Top				VINYLCEL, Bottom to Top				Ceiling	
Time	1	2	3	4	5	6	7	8	9	Notes
0	67	72	68	68	75	76	76	76	82	Fuel ignited, flames 5 ft
30 Sec	--	--	--	--	340	390	290	200	240	
1 Min	--	--	--	--	370	570	310	240	300	VINYLCEL flaming over area 2-1/2 x 7 ft
1-1/2	--	--	--	--	580	1200	690	460	430	Smoke Level 4 ft
2	--	--	--	--	740	1210	565	420	410	
3	--	--	--	--	700	1060	560	405	425	Flaming 8 ft high;less smoke
4	--	--	--	--	700	750	550	420	420	Flaming at cracks;gases not ignitible by torch
5	--	--	--	--	840	835	640	455	420	
6	95	660	78	106	800	1280	710	440	420	Flames 6 ft high
7	124	384	82	116	740	1060	640	420	400	
8	--	--	--	--	800	1270	780	432	395	
9	166	285	88	132	900	1145	540	445	415	Gases not ignitible
10	195	325	90	126	840	900	515	425	430	Wind caused ignition of fresh area (at cracks) to 5 ft
11	190	440	92	125	820	620	440	400	370	4-5 in. opening between sheets
12	190	430	95	125	580	520	370	355	370	VINYLCEL flames out
13	190	560	100	150	480	780	380	390	380	VINYLCEL burning again, new location
14	200	450	100	150	440	590	360	350	360	Flames 6 ft high
15	202	420	100	170	470	820	370	380	350	
16	210	400	100	160	440	780	380	360	360	
17	210	390	100	150	560	660	390	350	350	
19	--	--	--	--	--	--	--	--	--	TC in flames indicated from 1200°F to 1700°F max.
20	242	310	100	165	480	920	390	360	360	
21	250	380	110	160	420	700	360	350	310	No flames from VINYLCEL
22	252	420	110	170	510	970	360	350	320	Heptane flames 4-5 ft smoke diminished.
23	265	310	110	170	430	930	370	340	320	
24	272	360	115	170	430	780	350	340	320	
25	268	360	120	170	450	620	330	320	310	
26	275	320	120	170	390	740	310	310	290	
27	290	315	120	170	390	590	290	290	290	
28	285	360	120	170	380	620	280	290	290	
29	285	340	120	160	370	400	280	280	280	
30	285	330	120	170	400	480	250	270	270	
31	292	320	120	170	400	500	300	280	280	
32	290	370	125	165	365	410	280	270	280	
33	310	295	120	170	380	500	270	275	280	Fire extinguished

3E-20

Sheet 8 of Report No. E455-T-258



Sheet 9 of Report No. E455-T-258

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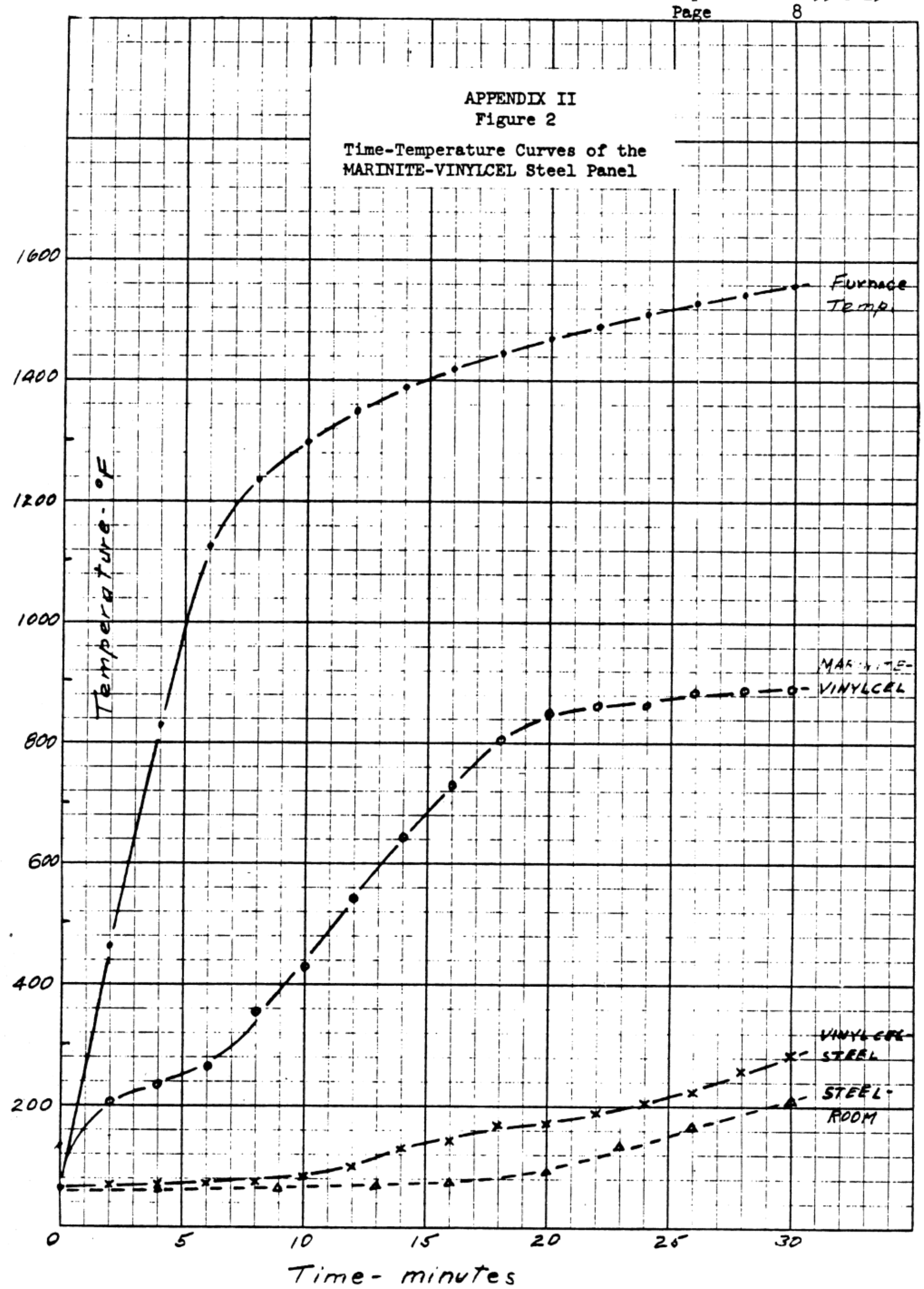
Report No. E455-T-258
Page 8

APPENDIX II
Figure 2

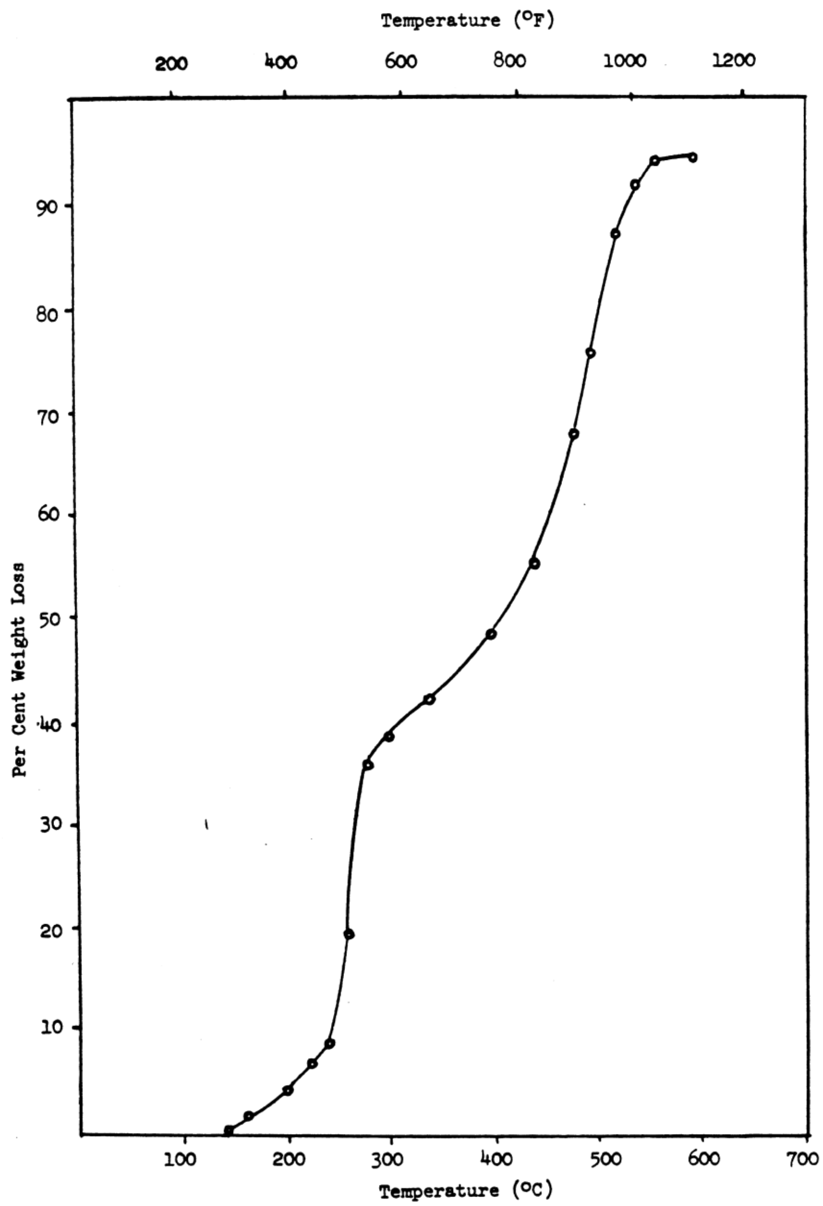
Time-Temperature Curves of the MARINITE-VINYLCEL Steel Panel



3E-22

Sheet 10 of Report No. E455-T-258

APPENDIX II
Figure 3
TGA Curve of VINYLCEL



Sheet 11 of Report No. E455-T-258

APPENDIX III

Photograph A
Fire Test Building

The test area is 16 x 8 x 8 ft with a 5 x 5-ft attached vestibule. The vestibule door was left open to admit air. A suspended ceiling of MARINITE sealed the space between the two top lines of vents. The first vent from the left on the second line from the top was left open for smoke venting. The first and second vents from the left on the bottom line were left open for air. Similar vents are on the other side of the building.

Sheet 12 of Report No. E455-T-258

BEST IMAGE AVAILABLE

APPENDIX III

Photograph B

View Through Vestibule During Test

This was taken during the test with steel facing. It is not believed the total height of flame appears in the photograph.

Sheet 13 of Report No. E455-T-258

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BEST IMAGE AVAILABLE

APPENDIX III

Photograph C
View of Test A (Steel Facing)
Prior to Light Off

Note speed-clip fasteners at 1, 3, 5, and 7 ft elevation, spaced 2 ft on centers across width. Thermocouple wires are shown attached to the surface and the ceiling. Note also the VINYLCEL supported against the ceiling with chicken mesh and clips. The edge of the asbestos-cement draft shield 8 ft from the test face may also be seen at the top.

Sheet 14 of Report No. E455-T-258

GINNA/UFSAR

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-258

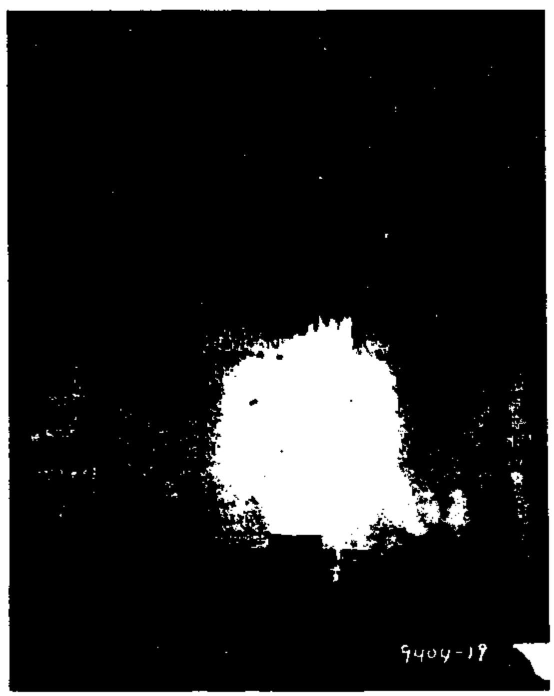
Page 13

BEST IMAGE AVAILABLE

APPENDIX III

Photograph D

Exposure Flame 50 Seconds After Light Off on Test A



The visible flame height is shown at 5 ft from the lower edge of the material at this time. Table 1 notes flame heights of 3-5 ft.

3E-27

Sheet 15 of Report No. E455-T-258

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BEST IMAGE
AVAILABLE

APPENDIX III

Photograph E
View of Test A After Fire Was Extinguished

Only smoke deposit and burn-off of zinc noted.

Sheet 16 of Report No. E455-T-258

BEST IMAGE AVAILABLE

APPENDIX III

Photograph F

Condition of VINYLCEL After Removal of Steel

The center sheet measured 7 x 3-1/2 ft and the horizontal line at 4 ft shows the position of the joint in the steel facing.

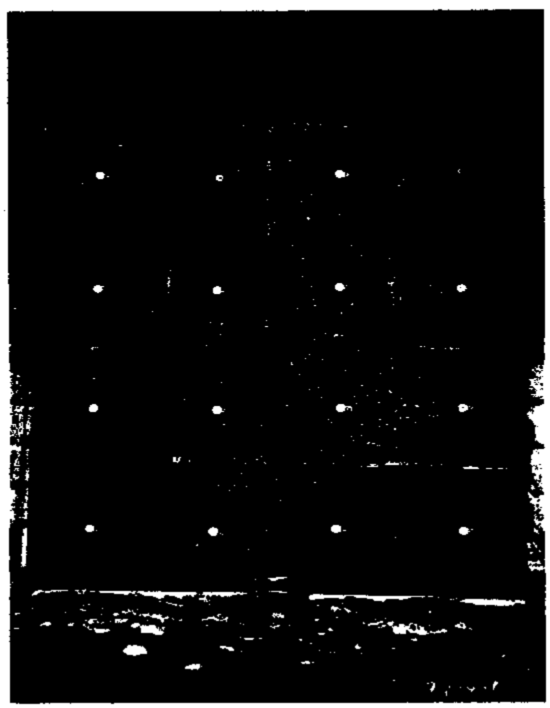
Sheet 17 of Report No. E455-T-258

BEST IMAGE AVAILABLE

APPENDIX III

Photograph G

View of Uncovered Sample for Test B



Erection was similar to that of Test A.

Note speed-clip fasteners at 1, 3, 5, and 7 ft elevation, spaced 2 ft on centers across width. Thermocouple wires are shown attached to the surface and the ceiling. Note also the VINYLCEL supported against the ceiling with chicken mesh and clips. The edge of the asbestos-cement draft shield 8 ft from the test face may also be seen at the top.

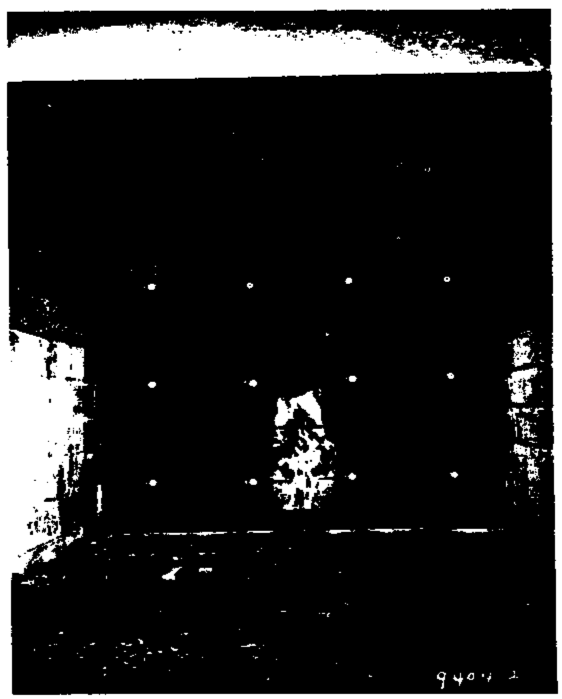
Sheet 18 of Report No. E455-T-258

BEST IMAGE AVAILABLE

APPENDIX III

Photograph H

View of Test B 30 Seconds After Light Off



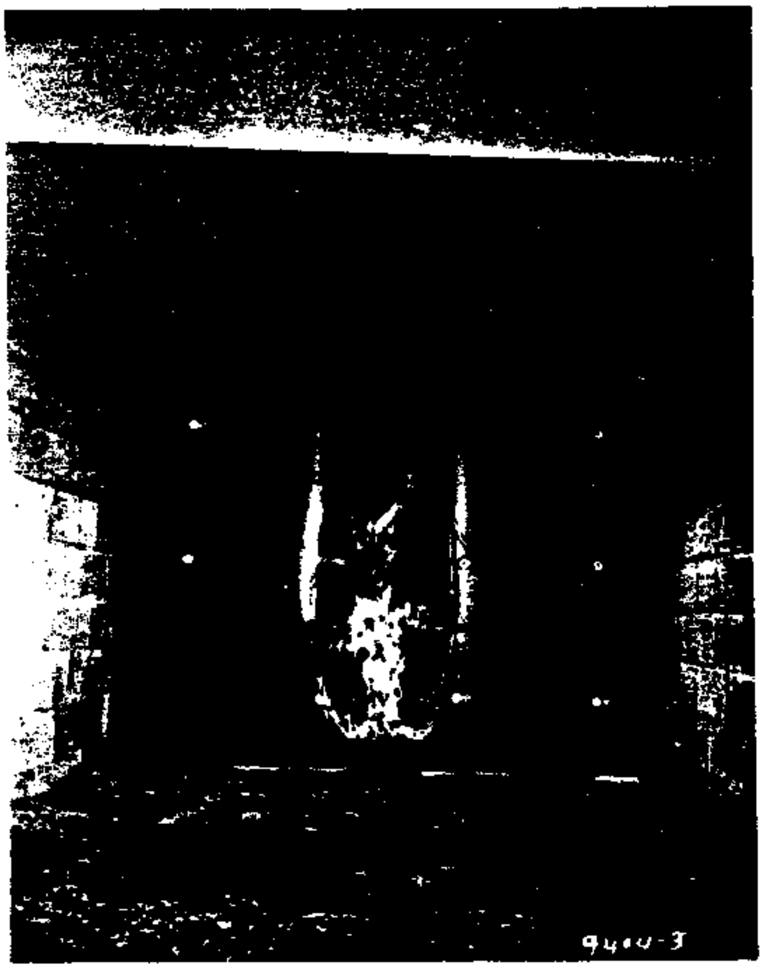
Flames 5 ft high were noted, but were not visible on the print.

3E-31

Sheet 19 of Report No. E455-T-258

Photograph J

View of Test B During Exposure



Note flames to 5 ft and cracks in material.

BEST IMAGE
AVAILABLE

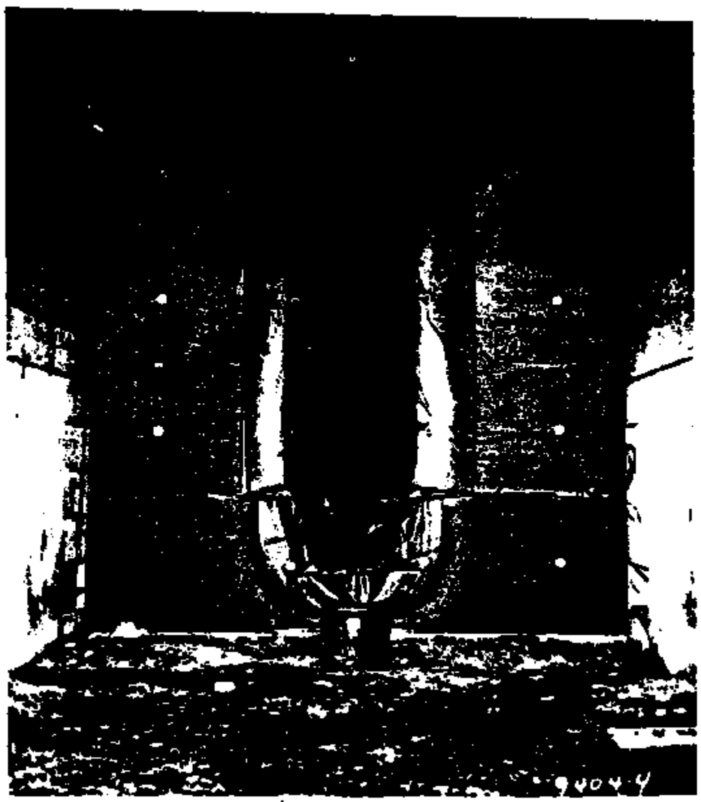
3E-32

Sheet 20 of Report No. E455-T-258

Photograph K

Bare VINYLCEL of Test B After Exposure

BEST IMAGE AVAILABLE



The charred area was 64 x 32 in. Width of through cracks was 8 to 10 in. Little lateral spread of flame noted.

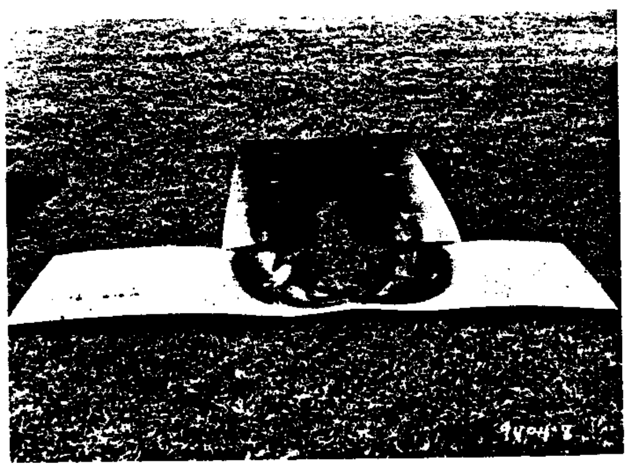
3E-33

Sheet 21 of Report No. E455-T-258

APPENDIX III

Photograph L

Sample of Test B After Removal



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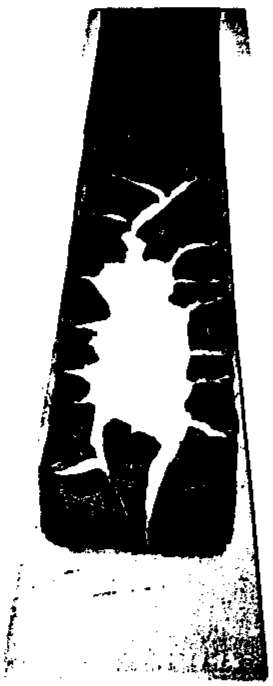
Sheet 22 of Report No. E455-T-258

BEST IMAGE AVAILABLE

APPENDIX III

Photographs M, N, O

Samples After Tunnel Exposure



Photograph M.

VINYLCEL After Tunnel Test - No sheet metal facing. (Exposed side down).

Photograph N.

VINYLCEL After Tunnel Test - Sheet metal used. (Exposed VINYLCEL shown).

Photograph O.

Reverse Side of VINYLCEL After Tunnel Test - With sheet metal.

1.

APPENDIX 3F

SUMMARY OF STRUCTURAL DESIGN CODE COMPARISON

BY THE FRANKLIN RESEARCH CENTER

TABLE OF CONTENTS

3F.1 INTRODUCTION

The Franklin Research Center, under contract to the NRC, compared the structural design codes and loading criteria used in the design of the R. E. Ginna Nuclear Power Plant against the corresponding codes and criteria currently used for licensing of new plants at the time of the Systematic Evaluation Program (SEP). The current and older codes were compared paragraph by paragraph to determine what effects the code changes could have on the load carrying capacity of individual structural members.

The scope of the review was confined to the comparison of former structural codes and criteria with counterpart current requirements. Correspondingly, the assessment of the impact of changes in codes and criteria was confined to what can be deduced solely from the provisions of the codes and criteria.

In order to carry out the code review objective of identifying criteria changes that could potentially impair perceived margins of safety, the following scheme of classifying code change impacts was used.

Where code changes involved technical content (as opposed to those which are editorial, organizational, administrative, etc.), the changes were classified according to the following scheme.

Each such code change was classified according to its potential to alter perceived margins of safety [a] in structural elements to which it applied. Four categories were established:

- Scale A Change - The new criteria have the potential to substantially impair margins of safety as perceived under the former criteria.
- Scale A_X Change - The impact of the code change on margins of safety is not immediately apparent. Scale A_X code changes require analytical studies of model structures to assess the potential magnitude of their effect upon margins of safety.
- Scale B Change - The new criteria operate to impair margins of safety but not enough to cause engineering concern about the adequacy of any structural element.
- Scale C Change - The new criteria will give rise to larger margins of safety than were exhibited under the former criteria.

This appendix is the summary of the code comparison findings. It has been reproduced directly from Appendix B to the Franklin Research Center Report, TER-C5257-322, Design Codes, Design Criteria and Loading Combinations (SEP Topic III-7.B), R. E. Ginna Nuclear Power Plant, dated May 27, 1982, which was transmitted by letter to RG&E from the NRC, dated January 4, 1983.

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a. That is, if (all other considerations remaining the same) safety margins as computed by the older code rules were to be recomputed for an as-built structure in accordance with current code provisions, would there be a difference due only to the code change under consideration.

Table 3F.2-1
AISC 1963 VERSUS AISC 1980 SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection			Comments	
AISC 1980	**AISC 1963**	**Structural Elements Potentially Affected**	**Limitations**	**Scale**
1.5.1.1	1.5.1.1	Structural members under tension, except for pin connected members		
			$F_y \leq 0.833\ F_u$	C
			$0.8333\ F_u < F_y < 0.875\ F_u$	B
			$F_y \geq 0.875\ F_u$	A
1.5.1.2.2	—	Beam and connection where the top flange is coped and subject to shear, failure by shear along a plane through fasteners, or shear and tension along and perpendicular to a plane through fasteners	See case study 1 for details.	
1.5.1.4.1 Subpara.6	1.5.1.4.1	Box-shaped members (subject to bending) of rectangular cross section whose depth is not more than 6 times their width and whose flange thickness is not more than 2 times the web thickness	New requirement in the 1980 Code	
1.5.1.4.1 Subpara.7	1.5.1.4.1	Hollow circular sections subject to bending	New requirement in the 1980 Code	
1.5.1.4.4	—	Lateral support requirements for box sections whose depth is larger than 6 times their width	New requirement in the 1980 Code	
1.5.2.2	1.7	Rivets, bolts, and threaded parts subject to 20,000 cycles or more	Change in the requirements	
1.7 & Appendix B	1.7	Members and connections subject to 20,000 cycles or more	Change in the requirements	

1.9.1.2 & Appendix C	1.7	Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2	New provisions added in the 1980 Code, Appendix C. See case study 10 for details.	
1.9.2.3 & Appendix C	—	Circular tubular elements subject to axial compression	New requirement in the 1980 Code	
1.10.6	1.10.6	Hybrid girder - reduction in flange stress	New requirements added in the 1980 Code. Hybrid girders were not covered in the 1963 Code. See case study 9 for details.	
1.11.4	1.11.4	Shear connectors in composite beams	New requirements added in the 1980 Code regarding the distribution of shear connectors (eqn. 1.11-7). The diameter and spacing of the shear connectors are also introduced.	
1.11.5	—	Composite beams or girders with formed steel deck	New requirement in the 1980 Code	
1.15.5.2 1.15.5.3 1.15.5.4	—	Restrained members when flange or moment connection plates for and connections of beams and girders are welded to the flange of I or H shaped columns	New requirement in the 1980 Code	
1.13.3	—	Roof surface not provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water (ponding)		
1.14.2.2	—	Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members	New requirement in the 1980 Code	
2.4 1st Para.	2.3 1st Para.	Slenderness ratio for columns. Must satisfy: $\frac{l}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}}$	See case study 4 for details. $F_y \leq 40$ ksi $40 < F_y < 44$ ksi $F_y \geq 44$ ksi	<u>Scale</u> C B A

2.7	2.6	Flanges of rolled W, M, or S shapes and similar built-up single-web shapes subject to compression	See case study 6 for details.	<u>Scale</u>
			$F_y \leq 36$ ksi	C
			$36 < F_y < 38$ ksi	B
			$F_y \geq 38$ ksi	
2.9	2.8	Lateral bracing of members to resist lateral and torsional displacement	See case study 7 for details.	
Appendix D	—	Web tapered members	New requirement in the 1980 Code	

<u>Scale B</u>

1.9.2.2	1.9.2	Flanges of square and rectangular box sections of uniform thickness, of stiffened elements, when subject to axial compression or to uniform compression due to bending	The 1980 Code limit on width-to-thickness ratio of flanges is slightly more stringent than that of the 1963 Code.
1.10.1	—	Hybrid girders	Hybrid girders were not covered in the 1963 Code. Application of the new requirement could not be much different from other rational method.
1.11.4	1.11.4	Flat soffit concrete slabs, using rotary kiln produced aggregates conforming to ASTM C330	Lightweight concrete is not permitted in nuclear plants as structural members (Ref. ACI-349).
1.13.2	—	Beams and girders supporting large floor areas free of partitions or other source of damping, where transient vibration due to pedestrian traffic might not be acceptable	Lightweight construction not applicable to nuclear structures which are designed for greater loads
1.14.6.1.3	—	Flare type groove welds when flush to the surface of the solid section of the bar	
1.16.4.2	1.16.4	Fasteners, minimum spacing, requirements between fasteners	
1.16.5	1.16.5	Structural joints, edge distances of holes for bolts and rivets	

1.15.5.5	—	Connections having high shear in the column web	New insert ion the 1980 Code
2.3.1 2.3.2	—	Braced and unbraced multi-story frame - instability effect	Instability effect on short buildings will have negligible effect.
2.4	2.3	Members subject to combined axial and bending moments	Procedure used in the 1963 Code for the interaction analysis is replaced by a different procedure. See case study 8 for details.
Scale C			
1.3.3	1.3.3	Support girders and their connections - pendant operated traveling cranes	
		The 1963 Code requires 25% increase in live loads to allow for impact as applied to traveling cranes, while the 1980 Code requires 10% increase.	The 1963 Code requirement is more stringent, and, therefore, conservative.
1.5.1.5.3	1.5.2.2	Bolts and rivets - projected area - in shear connections	
		$F_p = 1.5\ F_u$ (1980 Code) $F_p = 1.35\ F_y$ (1963 Code)	Results using 1963 Code are conservative.
1.10.5.3	1.10.5.3	Stiffeners in girders - spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes	New design concept added in 1980 Code giving less stringent requirements. See case study 5 for details.
1.11.4	1.11.4	Continuous composite beams; where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions	New requirement added in the 1980 Code

Table 3F.3-1
ACI 318-63 VERSUS ACI 349-76 SUMMARY OF CODE COMPARISON

Scale A

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
7.10.3	805	Columns designed for stress reversals with variation of stress from f_y in compression to 1/2 f_y in tension	Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions.
Chapter 9 9.1, 9.2, & 9.3 most specifically	Chapter 15	All primary load-carrying members or elements of the structural system are potentially affected	Definition of new loads not normally used in design of traditional buildings and redefinition of load factors and capacity reduction factors has altered the traditional analysis requirements.*
10.1 & 10.10	—	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.1	—	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.13	—	Short brackets and corbels which are primary load-carrying members	As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.15	—	Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear-type cracks	Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.16	—	All structural walls - those which are primary load-carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects	Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements.
18.1.4 & 18.4.2	—	Prestressed concrete elements	New load combinations here refer to Chapter 9 load combinations.*

Chapter 19	—	Shell structures with thickness equal to or greater than 12 inches	This chapter is completely new; therefore, shell structures designed by the general criteria of older codes may not satisfy all aspects of this chapter. Additionally, this chapter refers to Chapter 9 provisions.
Appendix A	—	All elements subject to time-dependent and position-dependent temperature variations and which are restrained such that thermal strains will result in thermal stresses	New appendix; older Code did not give specific guidelines on temperature limits for concrete. The possible effects of strength loss in concrete at high temperatures should be assessed.
Appendix B	—	All steel embedments used to transmit loads from attachments into the reinforced concrete structures	New appendix; therefore, considerable review of older designs is warranted.**
Appendix C	—	All elements whose failure under impulsive and impactive loads must be precluded	New appendix; therefore, considerations and review of older designs is considered important.**

<u>Scale B</u>

1.3.2	103(b)	Ambient temperature control for concrete inspection - upper limit reduced 5° (from 100°F to 95°F) applies to all structural concrete	Tighter control to ensure adequate control of curing environment for cast-in-place concrete.
1.5	—	Requirement of a "Quality Assurance Program" is new. Applies to all structural concrete	Previous codes required inspection but not the establishment of a quality assurance program.
Chapter 3	Chapter 4	Any elements containing steel with f_y > 60,000 psi or lightweight concrete	Use of lightweight concrete in a nuclear plant not likely. Elements containing steel with f_y > 60,000 psi may have inadequate ductility or excessive deflections at service loads.
3.2	402	Cement	This serves to clarify intent of previous code.
3.3	403	Aggregate	Eliminated reference to lightweight aggregate.
3.3.1	403	Any structural concrete covered by ACI 349-76 and expected to provide for radiation shielding in addition to structural capacity	Controls of ASTM C637, "Standard Specifications for Aggregates for Radiation Shielding Concrete," closely parallel those for ASTM C33, "Standard Specification for Concrete Aggregates."

3.3.3	403	Aggregate	To ensure adequate control.
3.4.2	404	Water for concrete	Improve quality control measures.
3.5	405	Metal reinforcement	Removed all reference to steel with $f_y > 60{,}000$ psi.
3.6	406, 407, & 408	Concrete mixtures	Added requirements to improve quality control.
4.1 & 4.2	501 & 502	Concrete proportioning	Proportioning logic improved to account for statistical variation and statistical quality control.
4.3	504	Evaluation and acceptance of concrete	Added provision to allow for design specified strength at age > 28 days to be used. Not considered to be a problem, since large cross sections will allow concrete in place to continue to hydrate.
5.7	607	Curing of very large concrete elements and control of hydration temperature	Attention to this is required because of the thicker elements encountered in nuclear-related structures.
6.3.3	—	All structural elements with embedded piping containing high temperature materials in excess of 150°F, or 200°F in localized areas not insulated from the concrete	Previous codes did not address the problem of long periods of exposure to high temperature and did not provide for reduction in design allowables to account for strength reduction at high (> 150°F) temperatures.
7.5, 7.6, & 7.8	805	Members with spliced reinforcing steel	Sections on splicing and tie requirements amplified to better control strength at splice locations and provide ductility.
7.9	805	Members containing deformed wire fabric	New sections to define requirements for this new material.
7.10 & 7.11	—	Connection of primary load-carrying members and at splices in column steel	To ensure adequate ductility.
7.12.3 7.12.4	—	Lateral ties in columns	To provide for adequate ductility.
7.13.1 through 7.13.3	—	Reinforcement in exposed concrete	New requirements to conform with the expected large thicknesses in nuclear related structures.
8.6	—	Continuous nonprestressed flexural members.	Allowance for redistribution of negative moments has been redefined as a function of the steel percentage.

9.5.1.1	—	Reinforced concrete members subject to bending - deflection limits	Allows for more stringent controls on deflection in special cases.
9.4	1505	Reinforcing steel - design strength limitation	See comments in Chapter 3 summary.
9.5.1.2 through 9.5.1.4	—	Slab and beams - minimum thickness requirements	Minimum thickness generally would not control this type of structure.
9.5.2.4	909	Beams and one-way slabs	Affects serviceability, not strength.
9.5.3	—	Non-prestressed two-way construction	Immediate and long time deflections generally not critical in structures designed for very large live loadings; however, design by ultimate requires more attention to deflection controls.
9.5.4 & 9.5.5	—	Prestressed concrete members	Control of camber, both initial and long time in addition to service load deflection, requires more attention for designs by ultimate strength.
10.2.7	—	Flexural members - new limit on B factor	Lower limit on B of 0.65 would correspond to an f'_c of 8,000 psi. No concrete of this strength likely to be found in a nuclear structure.
10.3.6	—	Compression members, with spiral reinforcement or tied reinforcement, non-prestressed and prestressed.	Limits on axial design load for these members given in terms of design equations.
			See case study 2.
10.6.1 10.6.2 10.6.3 10.6.4	1508	Beams and one-way slabs	Changes in distribution of reinforcement for crack control.
10.6.5	—	Beams	New insert
10.8.1 10.8.2 10.8.3	912	Compression members, limiting dimensions	Moment magnification concept introduced for compression members. Results using column reduction factors in ACI 318-63 are reasonably the same as using magnification.

10.11.1 10.11.2 10.11.3 10.11.4 10.11.5 10.11.5.1 10.11.5.2 10.11.6 10.11.7 10.12	915 916	Compression members, slenderness effects	For slender columns, moment magnification concept replaces the so-called strength reduction concept but for the limits stated in ACI 318-63 both methods yield equal accuracy and both are acceptable methods.
10.15.1 10.15.2 10.15.3 10.15.4 10.15.5 10.15.6	1404 - 1406	Composite compression members	New items - no way to compare; ACI 318-63 contained only working stress method of design for these members.
10.17	—	Massive concrete members, more than 48 in. thick	New item - no comparison.
11.2.1 11.2.2	—	Concrete flexural members	For non-prestressed members, concept of minimum area of shear reinforcement is new. For prestressed members, Eqn. 11-2 is the same as in ACI 318-63. Requirement of minimum shear reinforcement provides for ductility and restrains inclined crack growth in the event of unexpected loading.
11.7 through 11.8.6	—	Non-prestressed members	Detailed provisions for this load combination were not part of ACI 318-63. These new sections provide a conservative logic which requires that the steel needed for torsion be added to that required for transverse shear, which is consistent with the logic of ACI 318-63. This is not considered to be critical, as ACI 318-63 required the designer to consider torsional stresses; assuming that some rational method was used to account for torsion, no problem is expected to arise.

11.9 through 11.9.6	—	Deep beams	Special provisions for shear stresses in deep beams is new. The minimum steel requirements are similar to the ACI 318-63 requirements of using the wall steel limits. Deep beams designed under previous ACI 318-63 criterion were reinforced as walls at the minimum and therefore no unreinforced section would have resulted.
11.10 through 11.10.7	—	Slabs and footings	New provision for shear reinforcement in slabs or footings for the two-way action condition and new controls where shear head reinforcement is used. Logic consistent with ACI 318-63 for these conditions and change is not considered major.
11.11.1	1707	Slabs and footings	The change which deletes the old requirement that steel be considered as only 50% effective and allows concrete to carry 1/2 the allowable for two-way action is new. Also deleted was the requirement that shear reinforcement not be considered effective in slabs less than 10 in. thick. Change is based on recent research which indicates that such reinforcement works even in thin slabs.
11.11.2 through 11.11.2.5	—	Slabs	Details for the design of shearhead is new. ACI 318-63 had no provisions for shearhead design. This section for slabs and footings is not likely to be found in older plant designs. If such devices were used, it is assumed a rational design method was used.
11.12	—	Openings in slabs and footings	Modification for inclusion of shearhead design. See above conclusion.
11.13.1 11.13.2	—	Columns	No problem anticipated since previous code required design consideration by some analysis.

Chapter 12	—	Reinforcement	Development length concept replaces bond stress concept in ACI 318-63. The various 1_d lengths in this chapter are based entirely on ACI 318-63 permissible bond stresses. There is essentially no difference in the final design results in a design under the new code compared to ACI 318-63.
12.1.6 through 12.1.63	918(C)	Reinforcement	Modified with minimum added to ACI 318-63, 918(C).
12.2.2 12.2.3	—	Reinforcement	New insert in ACI 349-76.
12.4	—	Reinforcement of special members	New insert. Gives emphasis to special member consideration.
12.8.1 12.8.2	—	Standard hooks	Based on ACI 318-63 bond stress allowables in general; therefore, no major change.
12.10.1 12.10.2(b)	—	Wire fabric	New insert. Use of such reinforcement not likely in Category I structures for nuclear plants.
12.11.2	—	Wire fabric	New insert. Mainly applies to precast prestressed members.
12.13.1.4	—	Wire fabric	New insert. Use of this material for stirrups not likely in heavy members of a nuclear plant.
13.5	—	Slab reinforcement	New details on slab reinforcement intended to produce better crack control and maintain ductility. Past practice was not inconsistent with this in general.
14.2	—	Walls with loads in the Kern area of the thickness	Change of the order of the empirical equation (14-1) makes the solution compatible with Chapter 10 for walls with loads in the Kern area of the thickness.

15.5	—	Footings - shear and development of reinforcement	Changes here are intended to be compatible with change in concept of checking bar development instead of nominal bond stress consistent with Chapter 12.
15.9	—	Minimum thickness of plain footing on piles	Reference to minimum thickness of plain footing on piles which was in ACI 318-63 was removed entirely.
16.2	—	Design considerations for a structure behaving monolithically or not, as well as for joints and bearings.	New but consistent with the intent of previous code.
17.5.3	2505	Horizontal shear stress in any segment	Use of Nominal Average Shear Stress equation (17-1) replaces the theoretical elastic equation (25-1) of ACI 318-63. It provides for easier computation for the designer.
18.4.1	—	Concrete immediately after prestress transfer	Change allows more tension, thus is less conservative but not considered a problem.
18.5	2606	Tendons (steel)	Augmented to include yield and ultimate in the jacking force requirement.
18.7.1	—	Bonded and unbonded members	Eqn. 18-4 is based on more recent test data.
18.9.1 18.9.2 18.9.3	—	Two-way flat plates (solid slabs) having minimum bonded reinforcement	Intended primarily for control of cracking.
18.11.3 18.11.4	—	Bonded reinforcement at supports	New to allow for consideration of the redistribution of negative moments in the design.
18.13 18.14 18.15 18.16.1	—	Prestressed compression members under combined axial load and bending. Unbonded tendons. Post tensioning ducts. Grout for bonded tendons.	New to emphasize details particular to prestressed members not previously addressed in the codes in detail.
18.16.2	—	Proportions of grouting materials	Expanded definition of how grout properties may be determined.
18.16.4	—	Grouting temperature	Expanded definition of temperature controls when grouting.

Scale C

7.13.4	—	Reinforcement in flexural slabs	

10.14	2306	Bearing - sections controlled by design bearing stresses	ACI 318-63 is more conservative, allowing a stress of 1.9 (0.25 f'_c) = 0.475 f'_c < 0.6 f'_c
11.2.3	1706	Reinforcement concrete members without prestressing	Allowance of spirals as shear reinforcement is new. Requirement, where shear stress exceeds $6\phi\sqrt{f'_c}$ of 2 lines of web reinforcement was removed.
13.0 to end	—	Two-way slabs with multiple square or rectangular panels	Slabs designed by the previous criteria of ACI 318-63 are generally the same or more conservative.
13.4.1.5	—	Equivalent column flexibility stiffness and attached torsional members	Previous code did not consider the effect of stiffness of members normal to the plane of the equivalent frame.
17.5.4 17.5.5	—	Permissible horizontal shear stress for any surface, ties provided or not provided	Nominal increase in allowable shear stress under new code.

* Special treatment of load and loading combinations is addressed in other sections of the report.

** Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

Table 3F.4-1
ACI 301-63 VERSUS ACI 301-72 (REVISED 1975) SUMMARY OF CODE COMPARISON

Scale B

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
3.8.2.1 3.8.2.3	309b	Lower strength concrete can be proportioned when "working stress concrete" is used	ACI 301-72 (Rev. 1975) bases proportioning of concrete mixes on the specified strength plus a value determined from the standard deviation of test cylinder strength results. ACI 301-63 bases proportioning for "working stress concrete" on the specified strength plus 15 percent with no mention of standard deviation. High standard deviations in cylinder test results could require more than 15 percent under ACI 301-72 (Rev. 1975)
3.8.2.2 3.8.2.3	309d	Mix proportions could give lower strength concrete	ACI 301-72 (Rev. 1975) requires more strength tests than ACI 301-63 for evaluation of strength and bases the strength to be achieved on the standard deviation of strength test results.
17.3.2.3	1704d	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) requires core samples to have an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. ACI 301-63 simply requires "strength adequate for the intended purpose." If "adequate for the intended purpose" is less than 85 percent of the specified strength, lower strength concrete could be used.

17.2	1702a 1703a	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) specifies that no individual strength test result shall fall below the specified strength by more than 500 psi. ACI 301-63 specifies that either 20 percent (1702a) or 10 percent (1703a) of the strength tests can be below the specified strength. Just how far below is not noted.
15.2.6.1	1502b1	Weaker tendon bond possible	ACI 301-72 (Rev. 1975) requires fine aggregate in grout when sheath is more than four times the tendon area. ACI 301-63 requires fine sand addition at five times the tendon area.
15.2.2.1 15.2.2.2 15.2.2.3	1502e1	Prestressing may not be as good	ACI 301-72 (Rev. 1975) gives considerably more detail for bonded and unbonded tendon anchorages and couplings. ACI 301-63 does not seem to address unbonded tendons.
8.4.3	804b	Cure of concrete may not be as good	ACI 301-72 (Rev. 1975) provides for better control of placing temperature. This will give better initial cure.
8.2.2.4	802b4	Concrete may be more nonuniform when placed	ACI 301-72 (Rev. 1975) provides for a maximum slump loss. This gives better control of the characteristics of the placed concrete.
8.3.2	803b	Weaker columns and walls possible	ACI 301-72 (Rev. 1975) provides for a longer setting time for concrete in columns and walls before placing concrete in supported elements.
5.5.2	—	Poor bonding of reinforcement to concrete possible	ACI 301-72 (Rev. 1975) provides for cleaning of reinforcement. ACI 301-63 has no corresponding section.
5.2.5.3	—	Reinforcement may not be as good	ACI 301-72 (Rev. 1975) provides for use of welded deformed steel wire fabric for reinforcement. ACI 301-63 has no corresponding section.
5.2.5.1 5.2.5.2	503a	Reinforcement may not be as good when welded steel wire fabric is used	ACI 301-72 (Rev. 1975) provides a maximum spacing of 12 in. for welded intersection in the direction of principal reinforcement.

5.2.1	—	Reinforcement may not have reserve strength and ductility	ACI 301-72 (Rev. 1975) has more stringent yield requirements.
4.6.3	406c	Floors may crack	ACI 301-72 (Rev. 1975) provides for placement of reshores directly under shores above, while ACI 301-63 states that reshores shall be placed "in approximately the same pattern."
4.6.2	—	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for reshoring no later than the end of the working day when stripping occurs.
4.6.4	—	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for load distribution by reshoring in multistory buildings.
4.2.13	—	Low strength possible if reinforcing steel is distorted	ACI 301-72 (Rev. 1975) requires that equipment runways not rest on reinforcing steel.
3.8.5	—	Possible to have lower strength floors	ACI 301-72 (Rev. 1975) places tighter control on the concrete for floors.
3.7.2 3.4.4	—	Embedments may corrode and lower concrete strength	ACI 301-72 (Rev. 1975) requires that it be demonstrated that mix water does not contain a deleterious amount of chloride ion.
3.4.2 3.4.3	—	Possible lower strength	ACI 301-72 (Rev. 1975) places tighter control on water-cement ratios for watertight structures and structures exposed to chemically aggressive solutions.
1.2	—	Possible damage to green or underage concrete resulting in lower strength	ACI 301-72 (Rev. 1975) provides for limits on loading of emplaced concrete.

Scale C

3.5	305	Better strength resulting from better placement and consolidation	ACI 301-63 gives a minimum slump requirement. ACI 301-72 (Rev. 1975) omits minimum slump which could lead to difficulty in placement and/or consolidation of very low slump concrete. A tolerance of 1 in above maximum slump is allowed provided the average slump does not exceed maximum. Generally the placed concrete could be less uniform and of lower strength.

3.6	306b	Better strength resulting from better placement and consolidation	ACI 301-63 provides for use of single mix design with maximum nominal aggregate size suited to the most critical condition of concreting. ACI 301-72 (Rev. 1975) allows waiver of size requirement if the architect-engineer believes the concrete can be placed and consolidated.
3.8.2.1	309b	Higher strength from better proportioning	ACI 301-63 bases proportioning for "ultimate strength" concrete on the specified strength plus 25%. ACI 301-72 (Rev. 1975) bases proportioning on the specified strength plus a value determined from the standard deviation of test cylinder strengths. The requirement to exceed the specified strength by 25% gives higher strengths than the standard deviation method.
4.4.2.2	404c	Better bond to reinforcement gives better strength	ACI 301-63 provides that form coating be applied prior to placing reinforcing steel. ACI 301-72 (Rev. 1975) omits this requirement. If form coating contacts the reinforcement, no bond will develop.
4.5.5	405b	Better strength and less chance of cracking or sagging	ACI 301-63 provides for keeping forms in place until the 28-day strength is attained. ACI 301-72 (Rev. 1975) provides for removal of forms when specified removal strength is reached.
4.6.2	406b	Better strength and less chance of cracking or sagging	Same as above but applied to reshoring.
4.7.1	407a	Better strength by curing longer in forms	ACI 301-63 provides for cylinder field cure under most unfavorable conditions prevailing for any part of structure. ACI 301-72 (Rev. 1975) provides only that the cylinders be cured along with the concrete they represent. Cure of cylinders could give higher strength than the in-place concrete and forms could be removed too soon.

5.2.2.1 5.2.2.2	—	Better strength, less chance of cracked reinforcing bars	ACI 301-72 (Rev. 1975) has less stringent bending requirement for reinforcing bars than does ACI 318-63.
5.5.4 5.5.5	505b	Better strength from reinforcement	ACI 301-63 provides for more overlap in welded wire fabric.
12.2.3	1201d	Better strength from better cure of concrete	ACI 301-63 provides for final curing for 7 days with air temperature above 50°F. ACI 301-72 (Rev. 1975) provides for curing for 7 days and compressive strength of test cylinders to be 70 percent of specified strength. This could allow termination of cure too soon.
14.4.1	1404	Better strength resulting from better uniformity	ACI 301-63 provides for a maximum slump of 2 in. ACI 301-72 (Rev. 1975) gives a tolerance on the maximum slump which could lead to nonuniformity in the concrete in place.
15.2.1.1	1502-c1b	Higher strength from higher yield prestressing bars	ACI 301-63 requires higher yield stress than does ACI 301-72 (Rev. 1975).
15.2.1.2	1502-c2	Higher strength from better prestressing steel	ACI 301-63 requires that stress curves from the production lot of steel be furnished. ACI 301-72 (Rev. 1975) requires that a typical stress-strain curve be submitted. The use of the typical curve may miss lower strength material.
16.3.4.3	1602-4c	Better strength resulting from better cylinder tests	ACI 301-63 requires 3 cylinders to be tested at 28 days; if a cylinder is damaged, the strength is based on the average of two. ACI 301-72 (Rev. 1975) requires only two 28-day cylinders; if one is damaged, the strength is based on the one survivor.
16.3.4.4	1602-4d	Better strength, less chance of substandard concrete	ACI 301-63 requires that less than 100 yd^3 of any class of concrete placed in any one day be represented by 5 tests. ACI 301-72 (Rev. 1975) allows strength tests to be waived on less than 50 yd^3.

17.3.2.3	1704d	Better strength could be developed	ACI 301-63 requires core strengths "adequate for the intended purposes." ACI 301-72 (Rev. 1975) requires an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. If "adequate for the intended purpose" is higher than 85 percent of the specified strength, the concrete is stronger.

Table 3F.5-1
ACI 318-63 VERSUS ASME B&PV CODE, SECTION III, DIVISION 2, 1980, SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3230	1506	Containment (load combinations and applicable load factor)*	Definition of new loads not normally used in design of traditional buildings.
Table CC-3230-1	1506	Containment (load combinations and applicable load factor)*	Definition of loads and load combinations along with new load factors has altered the traditional analysis requirements.
CC-3421.5	—	Containment and other elements transmitting in-plane shear	New concept. There is no comparable section in ACI 318-63, i.e., no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain conditions, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore a check of old designs could show some significant decrease in overall prediction of structural integrity.
CC-3421.6	1707	Peripheral shear in the region of concentrated forces normal to the shell surface	These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63, Sections 1707 (c) and (d) which address "punching" shear in slabs and footings with the ϕ factor taken care of in the basic shear equation (Section CC-3521.2.1, Eqn. 10). Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 12 for details.

CC-3421.7	921	Torsion	New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses. See case study 13 for details.
CC-3421.8	—	Bracket and corbels	New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure.
CC-3532.1.2	—	Where biaxial tension exists	ACI 318-63 did not consider the problem of development length in biaxial tension fields.
CC-3900 All sections in this chapter	—	Concrete containment*	New design criteria. ACI 318-63 did not contain design criteria for loading such as impulse or missile impact. Therefore, no comparison is possible for this section.

Scale B

CC-3320	—	Shells	Added explicit design guidance for concrete reactor vessels not stated in the previous code. Acceptance of elastic behavior as the basis for analysis is consistent with the logic of the older codes.
CC-3340	—	Penetrations and openings	Added to ensure the consideration of special conditions particular to concrete reactor vessels and containments. These conditions would have been considered in design practice even though not specifically referred to in the old code.

Table CC-3421-1	1503(c)	Containment-allowable stress for factored compression loads	ACI 318-63 allowable concrete compressive stress was 0.85 f'_c if an equivalent rectangular stress block was assumed; also ACI 318-63 made no distinction between primary and secondary stress. ACI 318-63 used 0.003 in./in. as the maximum concrete compressive strain at ultimate strength.
CC-3421.4.1	1701	Containment and any section carrying transverse shear	Modified and amplified from ACI 318-63, Section 1701.1. 1. ϕ factors removed from all equations and included in CC-3521.2.1, Eqn. 17. 2. Separation of equations applicable to sections under axial compression and axial tension. New equations added. 3. Equations applicable to cross sections with combined shear and bending modified for case where $\rho < 0.015$. 4. Modification for low values of ρ will not be a large reduction; therefore, change is not deemed to be major.
CC-3421.4.2	2610(b)	Prestressed concrete sections	ACI 318-63, Eqn. 26-13 is a straight line approximation of Eqn. 8 (the "exact" Mohr's circle solution) with the prestress force shear component "$V\rho$" added. (Ref. ACI 426 R-74) ACI 318-63, Eqn. 26-12 modified to include members with axial load on the cross section and modified to reflect steel percentage. Remaining logic similar to ACI 318-63, Section 2610. Both codes intend to control the principal tensile stress.
CC-3422.1	1508(b)	Reinforcing steel	ACI 318-63 allowed higher f_y if full scale tests show adequate crack control.

			The requirement for tests where f_y > 60 ksi was used would provide adequate assurance, in old design, that crack control was maintained.
CC-3422.1	1503(d)	All ordinary reinforcing steel	ACI 318-63 allowed stress for load resisting purposes was f_y. However, a capacity reduction factor ϕ of 0.9 was used in flexure. Therefore, allowable tensile stress due to flexure could be interpreted as limited to some percentage of f_y less than 1.0 f_y and greater than 0.9 f_y. Limiting the allowable tensile stress to 0.9 f_y is in effect the same as applying a capacity reduction factor ϕ of 0.9 to the theoretical equation.
CC-3422.1		All ordinary reinforcing steel	ACI 318-63 had no provision to cover limiting steel strains; therefore, this section is completely new. Traditional concrete design practice has been directed at control of stresses and limiting steel percentages to control ductility. The logic of providing a control of design parameters at the centroid of all the bars in layered bar arrangement is consistent with older codes and design practice.
CC-3422.2	1503(d)	Stress on reinforcing bars	ACI 318-63 allowed the compressive steel stress limit to be f_y; however, the capacity reduction factor for tied compression members was $\phi = 0.70$ and for spiral ties $\phi = 0.75$, applied to the theoretical equation. As this overall reduction for such members is so large, part of the reduction could be considered as reducing the allowable compressive stress to some level less than f_y; therefore, the 0.9 f_y limit here is consistent with and reasonably similar to the older code.
CC-3423	2608	Tendon system stresses	ACI 318-63 Section 2608 is generally less conservative.

CC-3431.3	—	Shear, torsion, and bearing	ACI 318-63 does not have a strictly comparable section; however, the 50% reduction of the ultimate strength requirements on shear and bearing stresses to get the working stress limits is identical to the ACI 318-63 logic and requirements.
Table CC-3431-1	—	Allowable stresses for service compression loads	Allowable concrete compressive stresses are less conservative than or the same as the ACI 318-63 equivalent allowables.
CC-3432.2	1003(b)	Reinforcing bar (compression)	ACI 318-63 is slightly more conservative in using 0.4 f_y up to a limit of 30 ksi. The upper limit is the same, since ACI 359-80 stipulates max f_y = 60 ksi.
CC-3432.2 (b), (c)	1004	Reinforcing bar (compression)	Logic similar to older codes. Allowance of 1/3 overstress for short duration loading.
CC-3433	2606	Tendon system stress	Limits here are essentially the same as in ACI 318-63 or slightly less conservative; ACI 318-63 limits effective prestress to 0.6 of the ultimate strength or 0.8 of the yield strength, whichever is smaller.
CC-3521	—	Reinforced concrete	Membrane forces in both horizontal and vertical directions are taken by the reinforcing steel, since concrete is not expected to take any tension. Tangential shear in the inclined direction is taken, up to V_c' by the concrete, and the rest by the reinforcing steel. In all cases, the ACI concept of ϕ is incorporated in the equation as 0.9. While not specifically indicating how to design for membrane stresses, ACI 318-63 indicated the basic premises that tension forces are taken by reinforcing steel (and not concrete) and that concrete can take some shear, but any excess beyond a certain limit must be taken by reinforcing steel.
CC-3521.2.1	1701	Nominal shear stress	Similar to ACI 318-63, with the exception of ϕ, which equals 0.85, being included in the Eqn. 17.

			Placing ϕ in the stress formula, rather than in the formulae for shear reinforcement, provides the same end result.
CC-3532	—	Where bundled bars are used	Bundled bars were not commonly used prior to 1963; therefore, no criteria were specified in ACI 318-63. In more recent codes, identical requirements are specified for bundled bars.
CC-3532.1.2	918(c)	Where tensile steel is terminated in tension zones	Similar to older code, but maximum shear allowed at cutoff point increased to 2/3, as compared to 1/2 in ACI 318-63, over that normally permitted. Slightly less conservative than ACI 318-63. This is not considered critical since good design practice has always avoided bar cutoff in tension zones.
CC-3532.1.2	1801	Where bars carrying stress are to be terminated	Development lengths derived from the basic concept of ACI 318-63 where: bond strength = tensile strength $\Sigma_0 \mu L = A_b f_y$ $L = \frac{A_b f_y}{(\mu \pi D)}$ If $\mu = 9.5 \sqrt{f'_c}$ then $L = 0.0335 \frac{A_b f_y}{\sqrt{f'_c}}$ With $\phi = 0.85$ $L = 0.0394 \frac{A_b f_y}{\sqrt{f'_c}}$ No change in basic philosophy for #11 and smaller bars.
CC-3532.3	919(h) 801	Hooked bars	Change in format. New values are similar for small bars and more conservative for large bars and higher yield strength bars. Not considered critical since prior to 1963 the use of f_y > 40 ksi steel was not common.

CC-3533	919	Shear reinforcement	Essentially the same concepts. Bend of 135° now permitted (versus 80° formerly) and two-piece stirrups now permitted. These are not considered as sacrificing strength. Other items here are identical.
CC-3534.1	—	Bundled bars - any location	Provisions for bundled bars were not considered in ACI 318-63. Bundled bars were not commonly used before the early 1960s. Later codes provide identical provisions.
CC-3536	—	Curved reinforcement	Early codes did not provide detailed information, but good design practice would consider such conditions.
CC-3543	2614	Tendon and anchor reinforcement	Similar to concepts in ACI 318-63, Section 2614 but new statement is more specific. Basic requirements are not changed.
CC-3550	—	Structures integral with containment	Statement here is specific to concrete reactor vessels. The logic of this guideline is consistent with the design logic used for all indeterminate structures. ACI 318-63 did not specifically state any guideline in this regard.
CC-3560		Foundation requirements	There is no comparable section in ACI 318-63. These items were assumed to be controlled by the appropriate general building code of which ACI 318-63 was to be a referenced inclusion. All items are considered to be part of common building design practice.

Scale C

CC-3421.9	2306 (f) and (g)	Bearing	ACI 318-63 is more conservative, allowing a stress of $1.9\ (0.25\ f'_c) = 0.475\ f'_c < 0.6\ f'_c$
CC-3431.2	2605	Concrete (allowable stress in concrete)	Identical to ACI 318-63 logic.

Appendix II	—	Concrete reactor vessels	ACI 318-63 did not contain any criteria for compressive strength modification for multiaxial stress conditions. Therefore, no comparison is possible for Section II-1100. Because of this, ACI 318-63 was more conservative by ignoring the strength increase which accompanies triaxial stress conditions. This section probably does not apply to concrete containment structures.
CC-3531	—	All	Rather conservative for service loads. Using ϕ of 0.9 for flexure, $\frac{U}{\phi} = \frac{1.5}{0.9}\ to\ \frac{1.8}{0.9} = 1.67\ to\ 2.0$ for ACI 318-63. By using the value of 2.0, the upper limit of the ratio of factored to service loads is employed.

* Special treatment of load and load combinations is addressed in other sections of the report.

Figure 3.7-1 Seismic Response Spectra, 8%g Housner Model

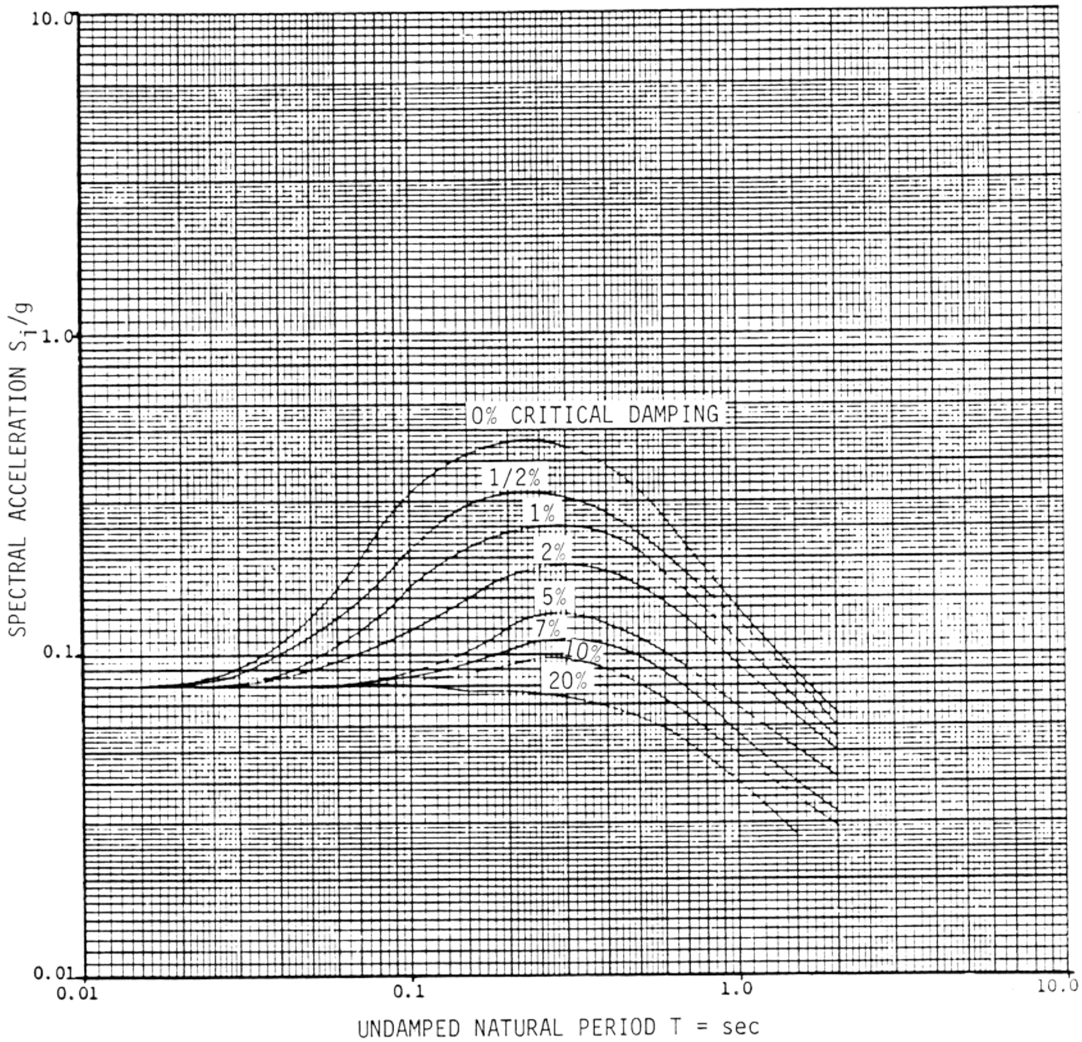

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Figure 3.7-1 Seismic Response Spectra, 8%g Housner Model

Figure 3.7-2 Seismic Response Spectra, 20%g Housner Model

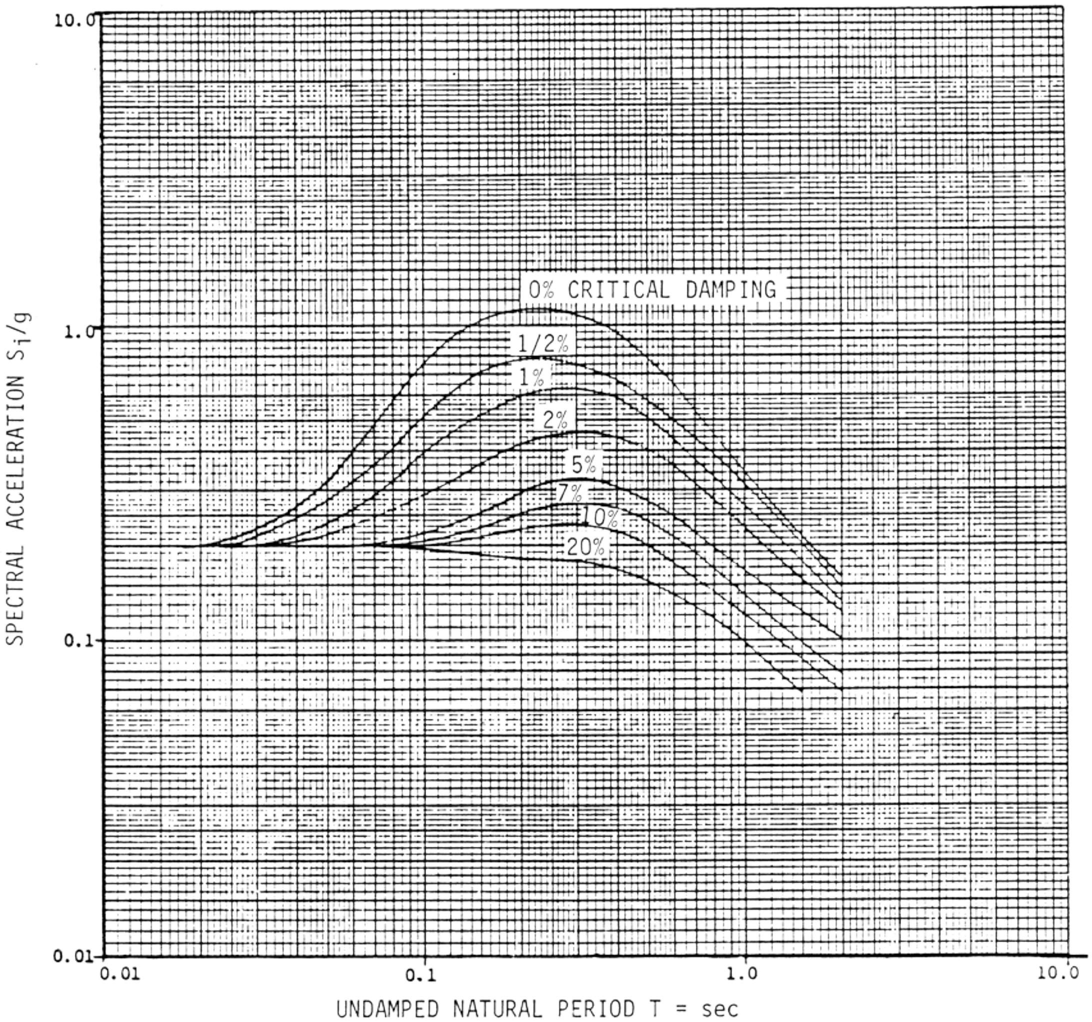

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Figure 3.7-2

Seismic Response Spectra, 20%g Housner Model

Figure 3.7-3 NRC Systematic Evaluation Program Site Specific Spectrum, Ginna Site (5% Damping)

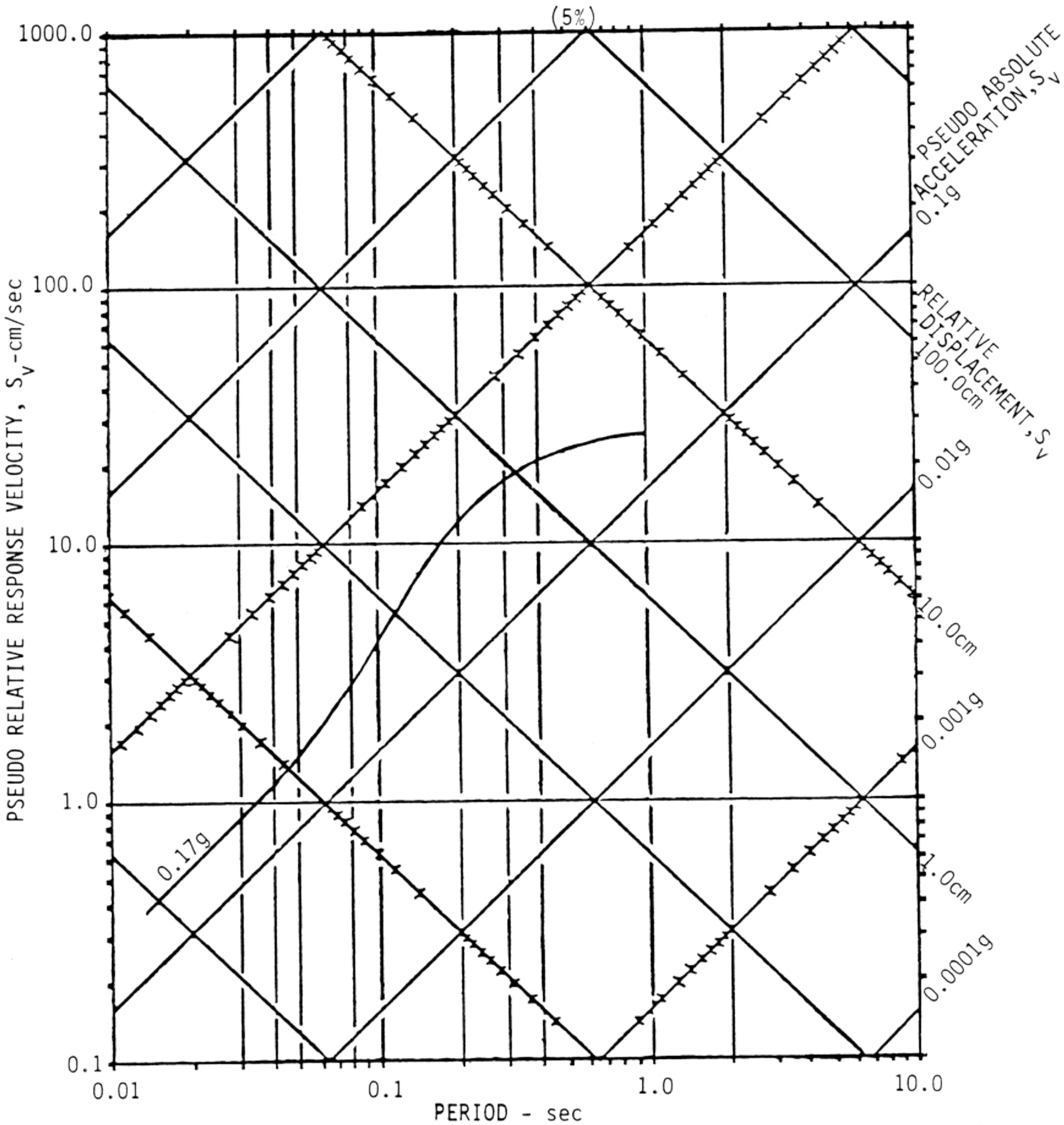

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Figure 3.7-3
NRC Systematic Evaluation Program Site Specific Spectrum, Ginna Site (5% Damping)

Figure 3.7-4 Comparison of the Housner Response Spectrum for 2% of Critical Damping with the 7% Regulatory Guide 1.60 Spectrum

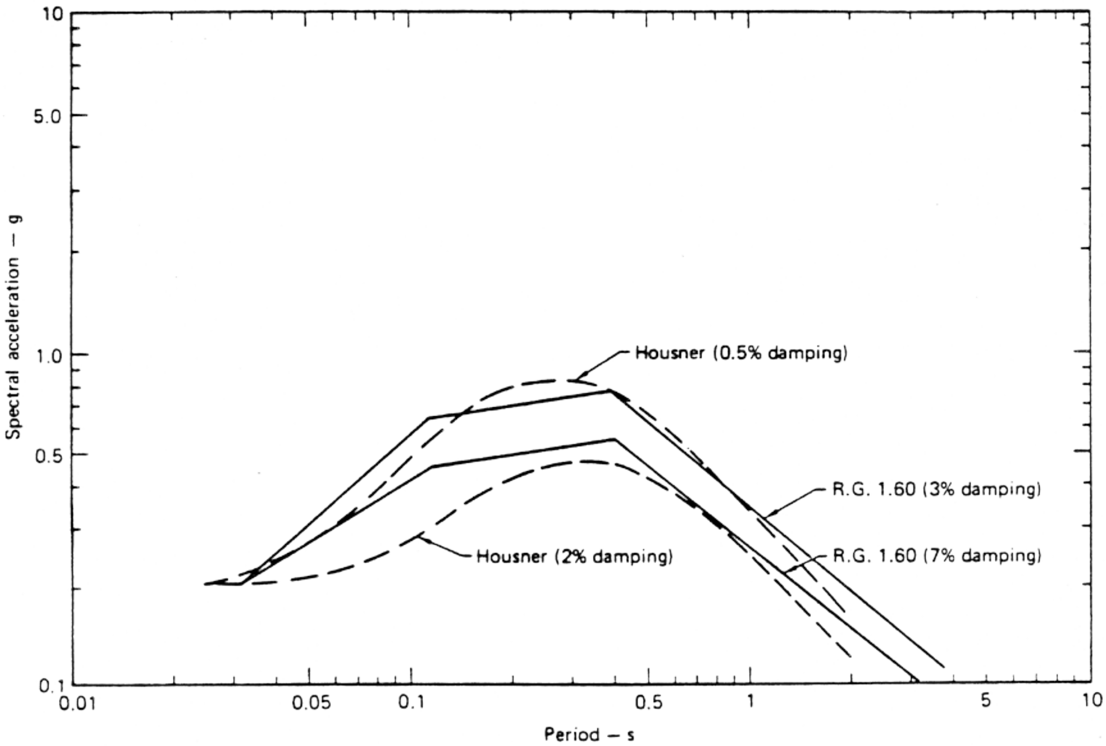

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Figure 3.7-4 Comparison of the Housner Response Spectrum for 2% of Critical Damping with the 7% Regulatory Guide 1.60 Spectrum

Figure 3.7-5 In-Structure Response Spectra for Interconnected Building, Half-Area and Full-Area Models

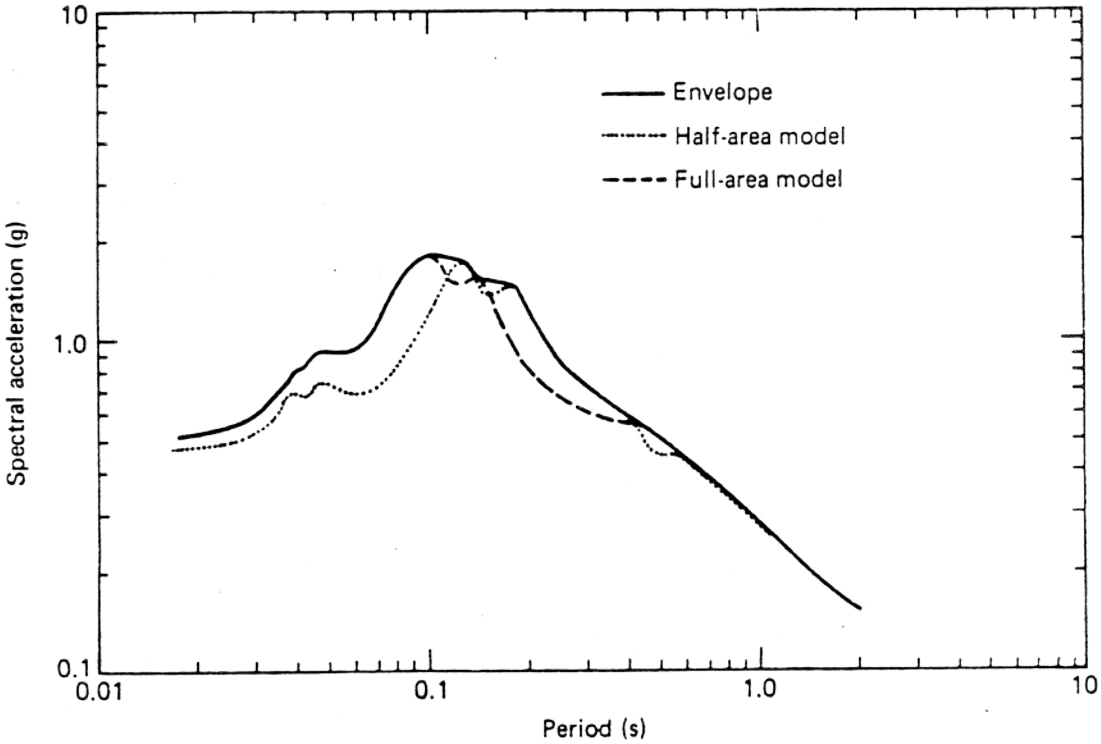

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Figure 3.7-5
In-Structure Response Spectra for Interconnected Building, Half-Area and Full-Area Models

Figure 3.7-6 Containment Building and Complex of Interconnected Seismic Category I and Nonseismic Structures, Plan View

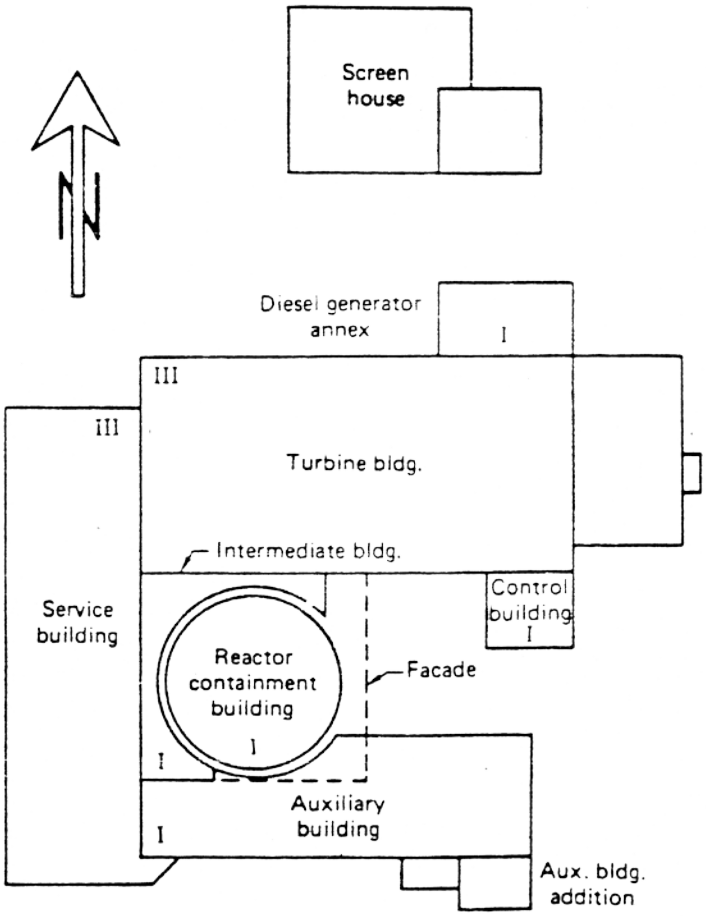

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Figure 3.7-6

Containment Building and Complex of Interconnected Seismic Category I and Nonseismic Structures, Plan View

Figure 3.7-7 Horizontal Response Spectra - SEP Systematic Evaluation Program

f	OBE	SSE
0 Hz	1 g	2 g
1	1	2
2	3	6
8	3	6
9	2	4
20	2	4
30	1	2
35	1	2

FOR VERTICAL MOTION
USE 2/3 DAMPING = 1%

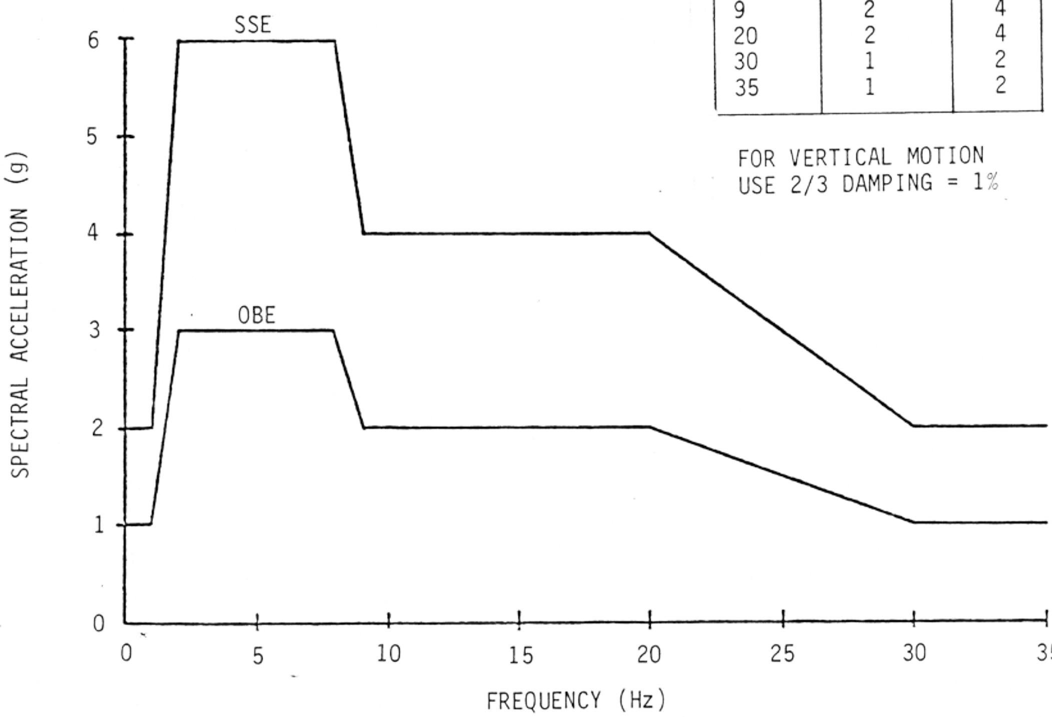

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Figure 3.7-7

Horizontal Response Spectra - SEP
Systematic Evaluation Program

Figure 3.7-8 Steam Generator Mathematical Model

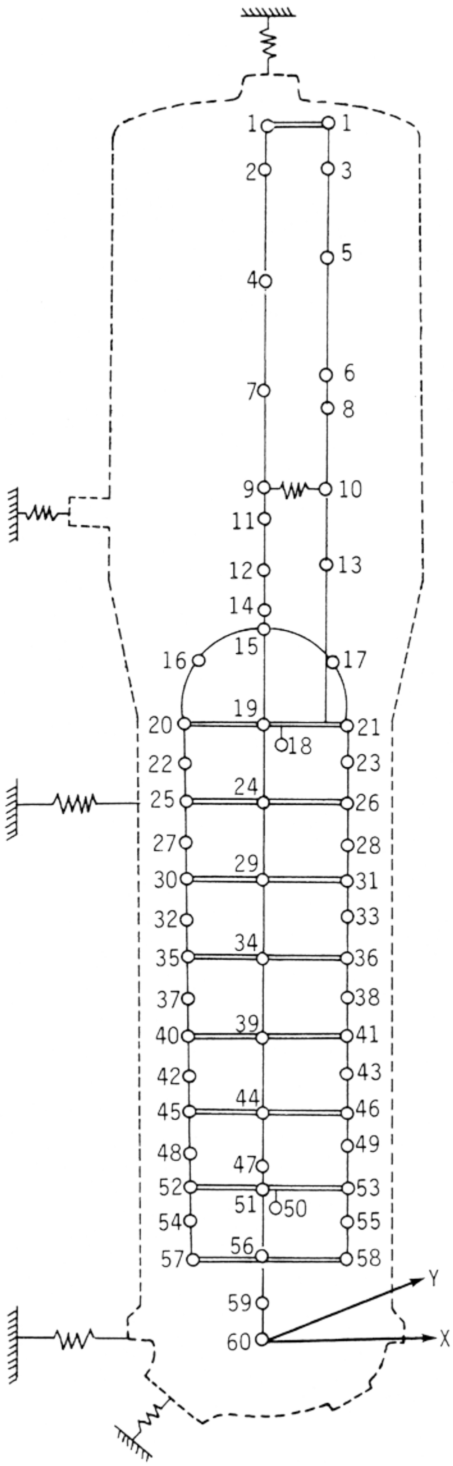

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Figure 3.7-8
Steam Generator Mathematical Model

Figure 3.7-9 Mathematical Model of Reactor Vessel

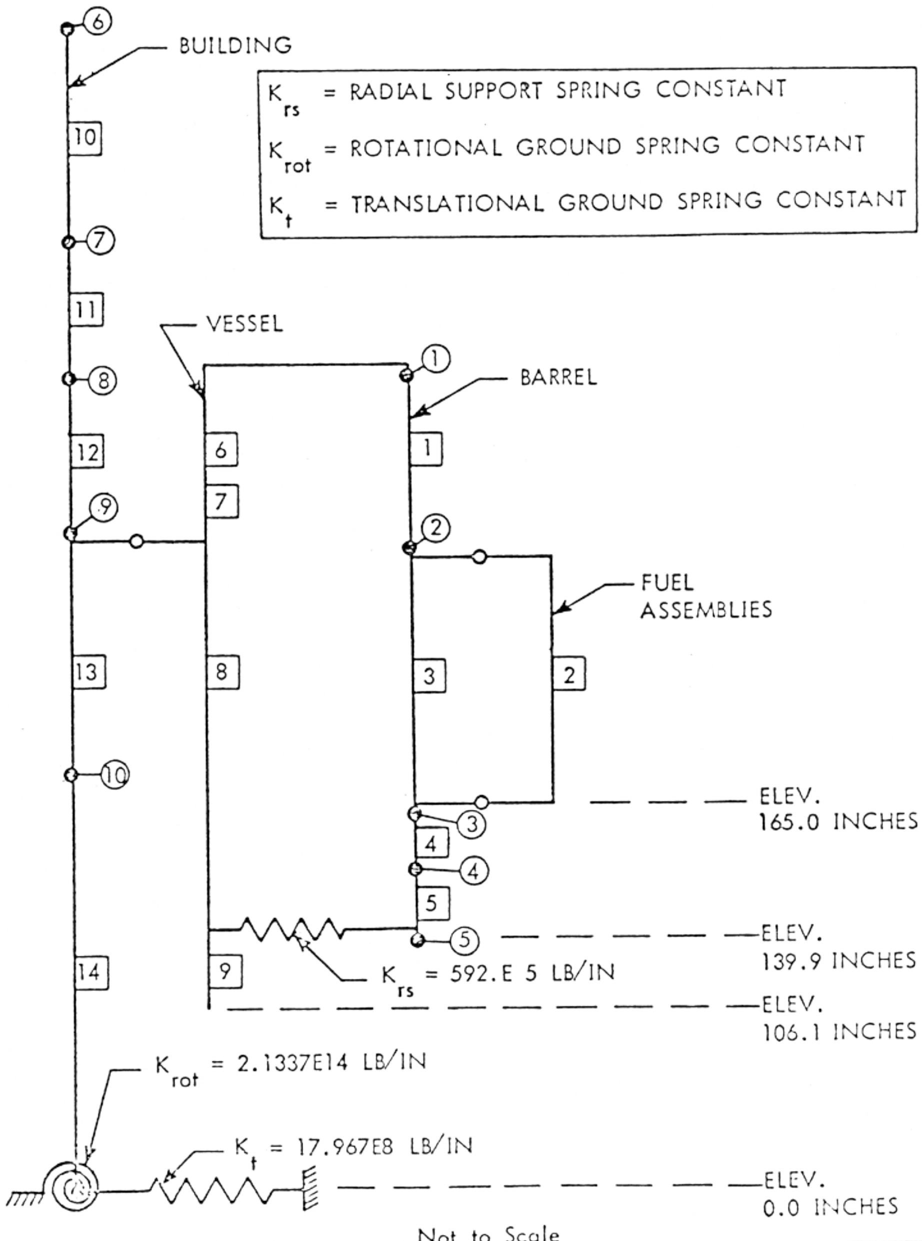

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Figure 3.7-9
Mathematical Model of Reactor Vessel

Figure 3.7-10 Seismic Average Acceleration Spectrum Design Earthquake, 1% Damping

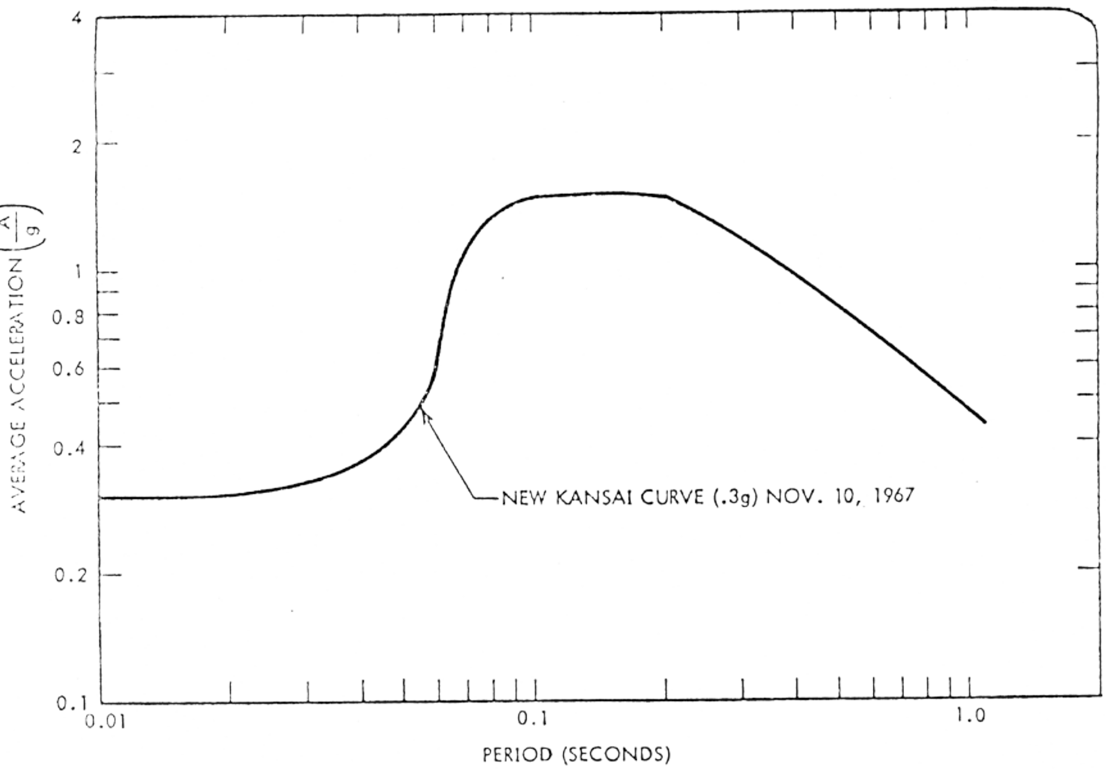

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Figure 3.7-10 Seismic Average Acceleration Spectrum Design Earthquake, 1% Damping

Figure 3.7-11 Locations Where In-Structure Response Spectra Were Generated in Interconnected Building Complex

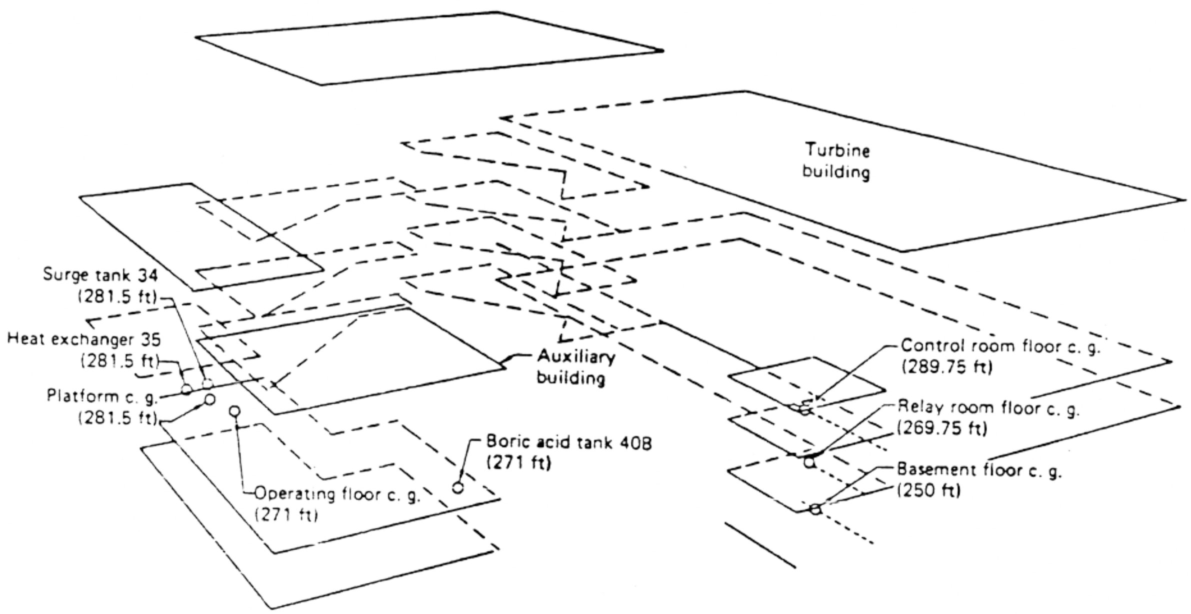

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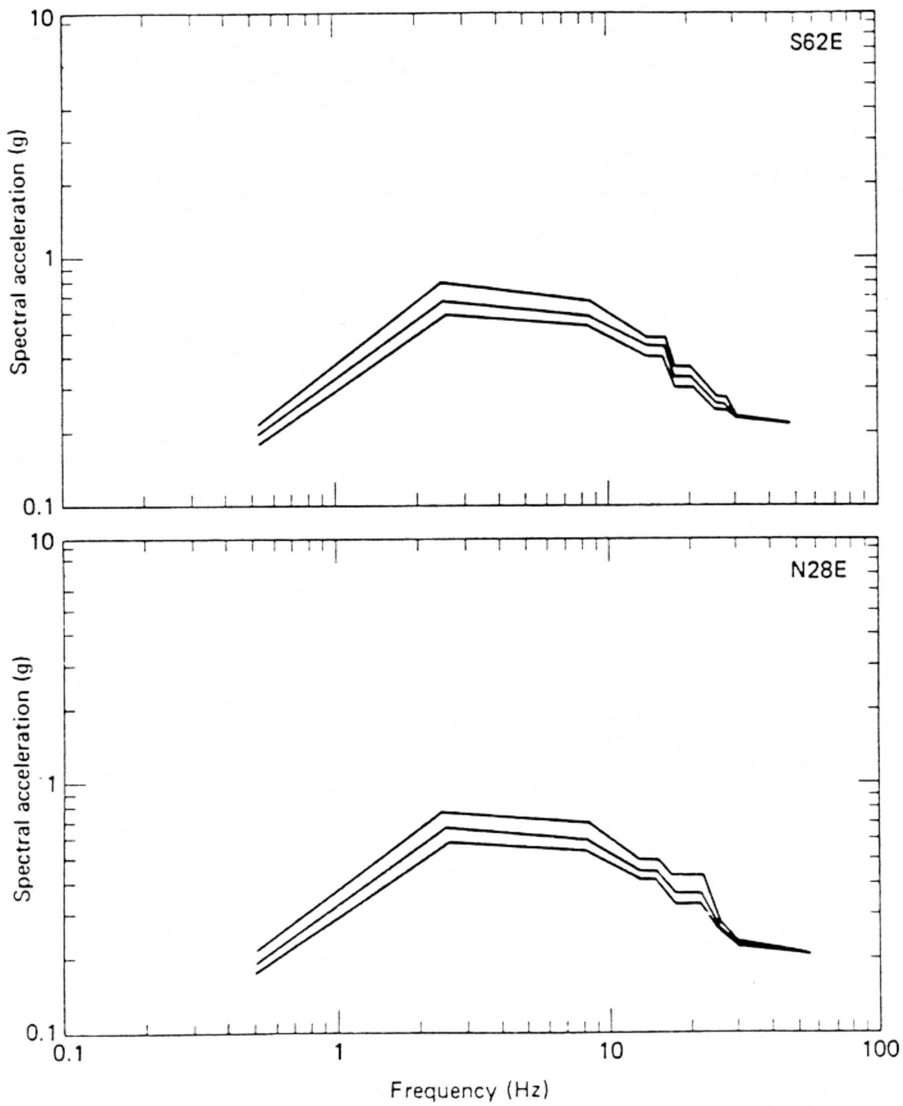
Figure 3.7-11
Locations Where In-Structure Response Spectra Were Generated in Interconnected Building Complex

Figure 3.7-12 SEP Response Spectra for Pressurizer PR-1 (Containment Building Elevation 253 ft) for 3%, 5%, and 7% Damping



TIME-HISTORY METHOD

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Figure 3.7-12

SEP Response Spectra for Pressurizer PR-1 (Containment Building Elevation 253 ft) for 3%, 5%, and 7% Damping

Figure 3.7-13 SEP Response Spectra for Control Rod Drive (Containment Building Elevation 253 ft) for 3%, 5%, 7% Damping

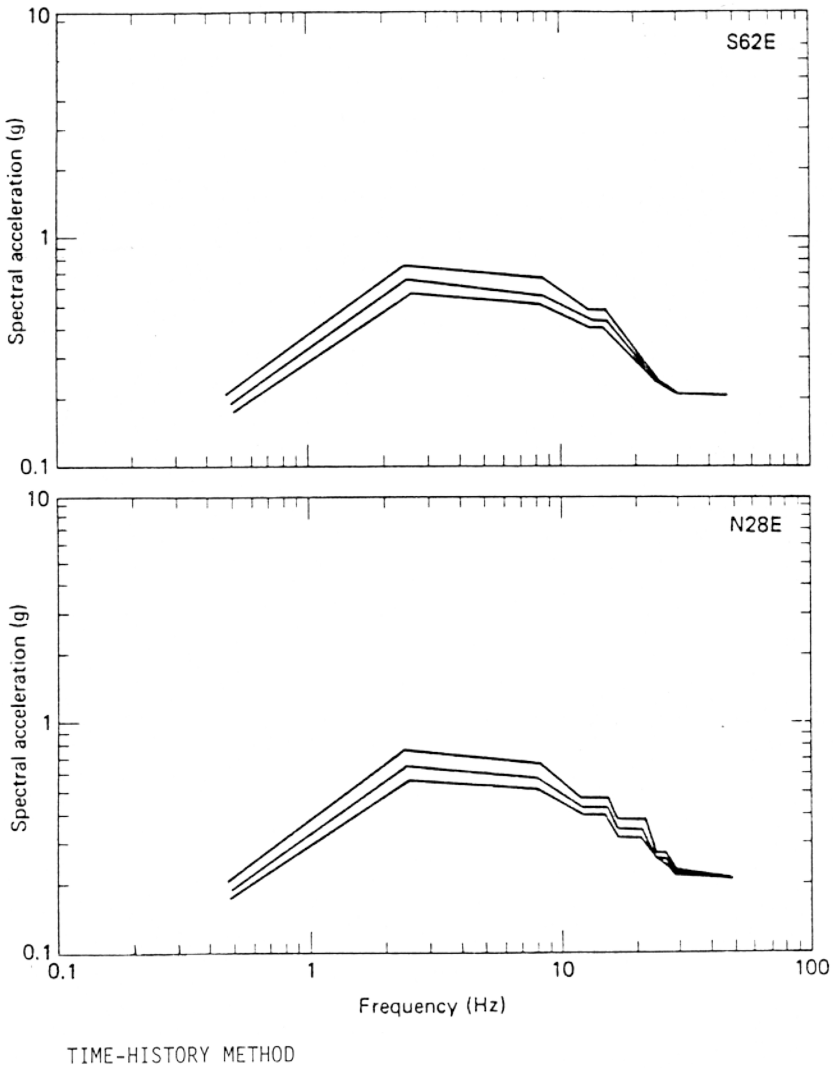

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Figure 3.7-13

SEP Response Spectra for Control Rod Drive (Containment Building Elevation 253 ft) for 3%, 5%, and 7% Damping

Figure 3.7-14 SEP Response Spectra for Control Rod Drive (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

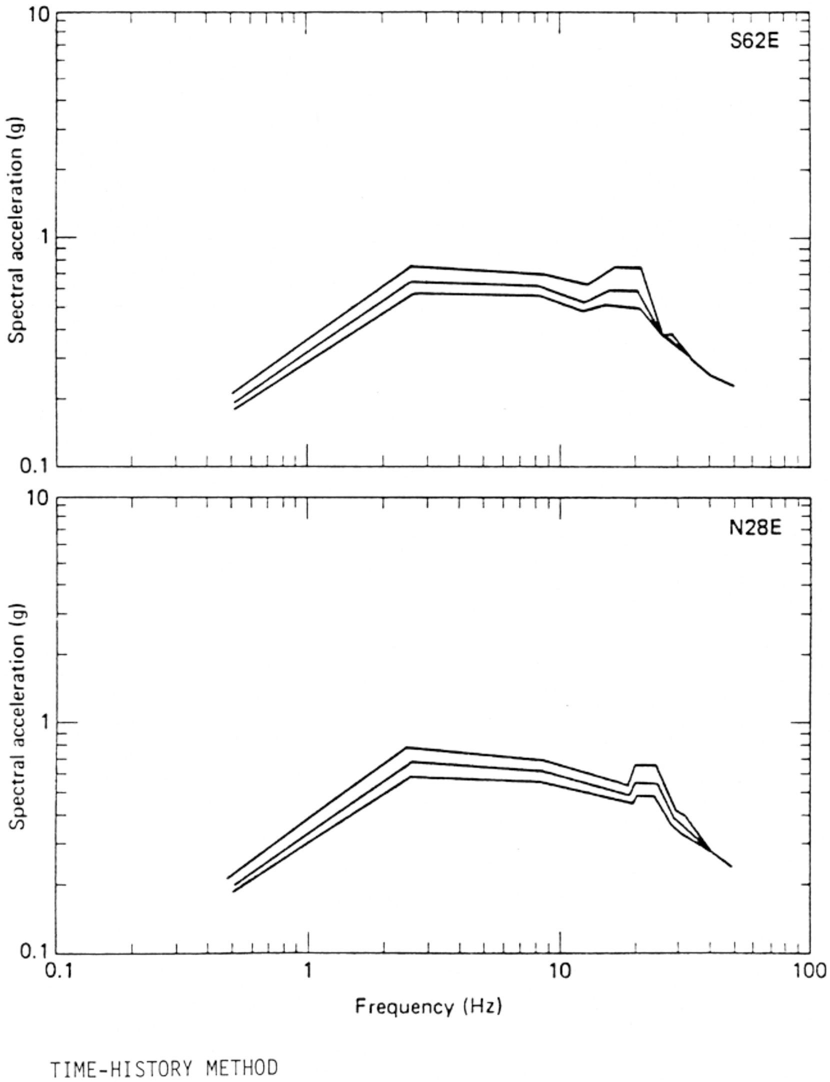

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Figure 3.7-14 SEP Response Spectra for Control Rod Drive (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

Figure 3.7-15 SEP Response Spectra for Steam Generator SG-1A (Containment Building Elevation 250 ft) for 3%, 5%, and 7% Damping

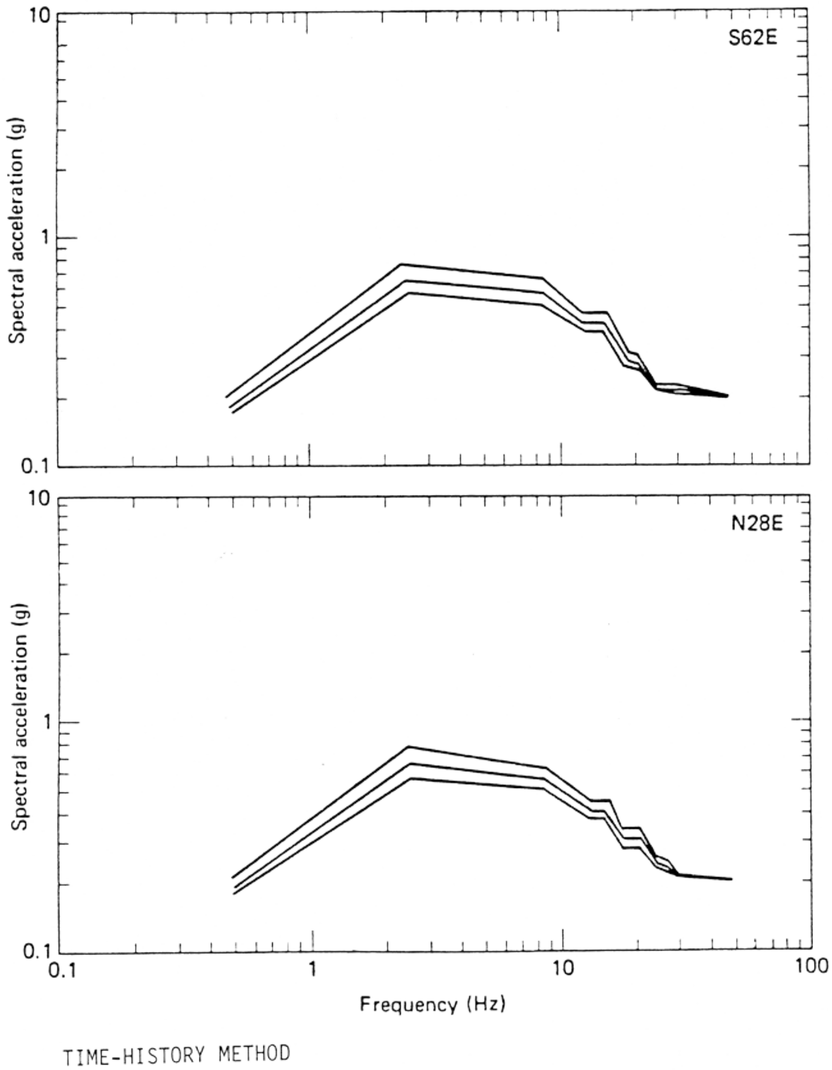

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Figure 3.7-15 SEP Response Spectra for Steam Generator SG-1A (Containment Building Elevation 250 ft) for 3%, 5%, and 7% Damping

Figure 3.7-16 SEP Response Spectra for Steam Generator SG-1A (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

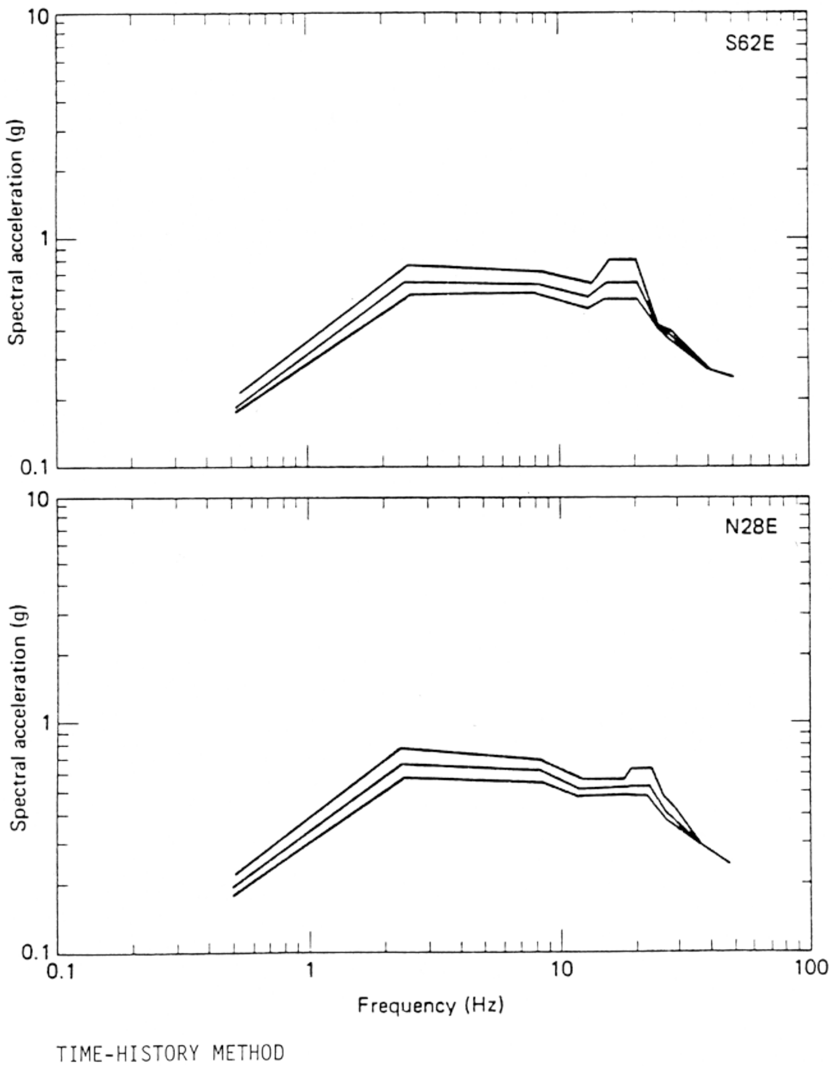

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Figure 3.7-16 SEP Response Spectra for Steam Generator SG-1A (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

Figure 3.7-17 SEP Response Spectra for Steam Generator SG-1B (Containment Building Elevation 250 ft) for 3%, 5%, and 7% Damping

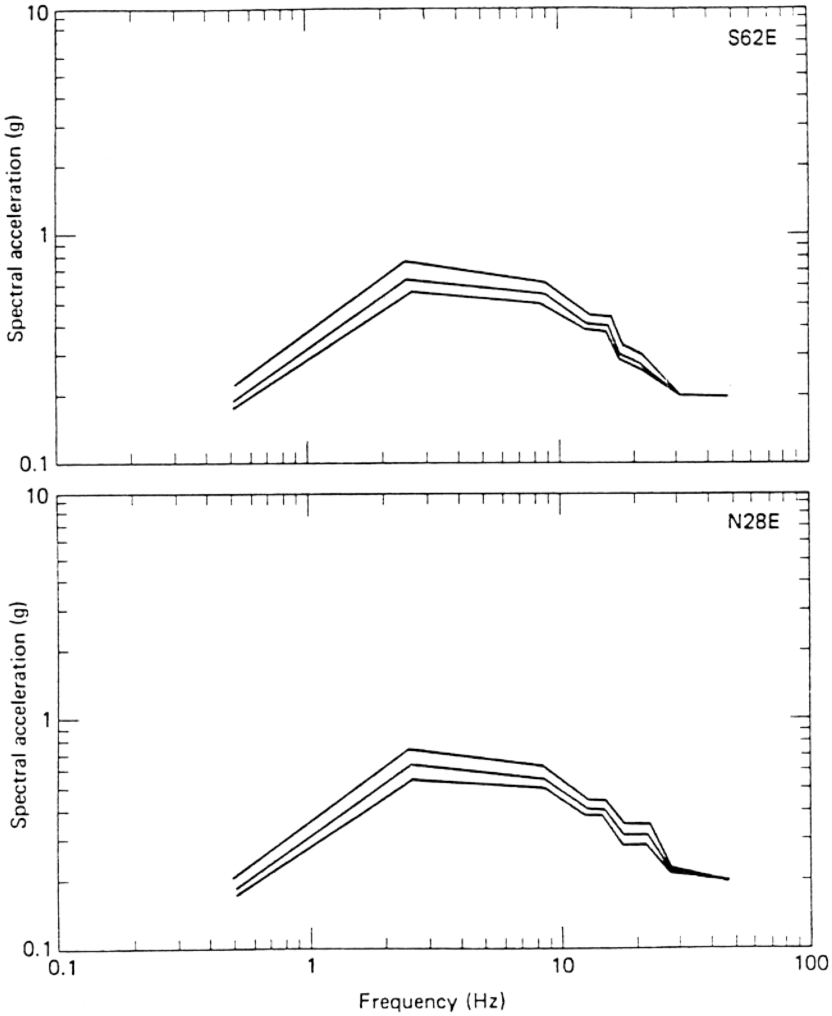

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Figure 3.7-17 SEP Response Spectra for Steam Generator SG-1B (Containment Building Elevation 250 ft) for 3%, 5%, and 7% Damping

Figure 3.7-18 SEP Response Spectra for Steam Generator SG-1B (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

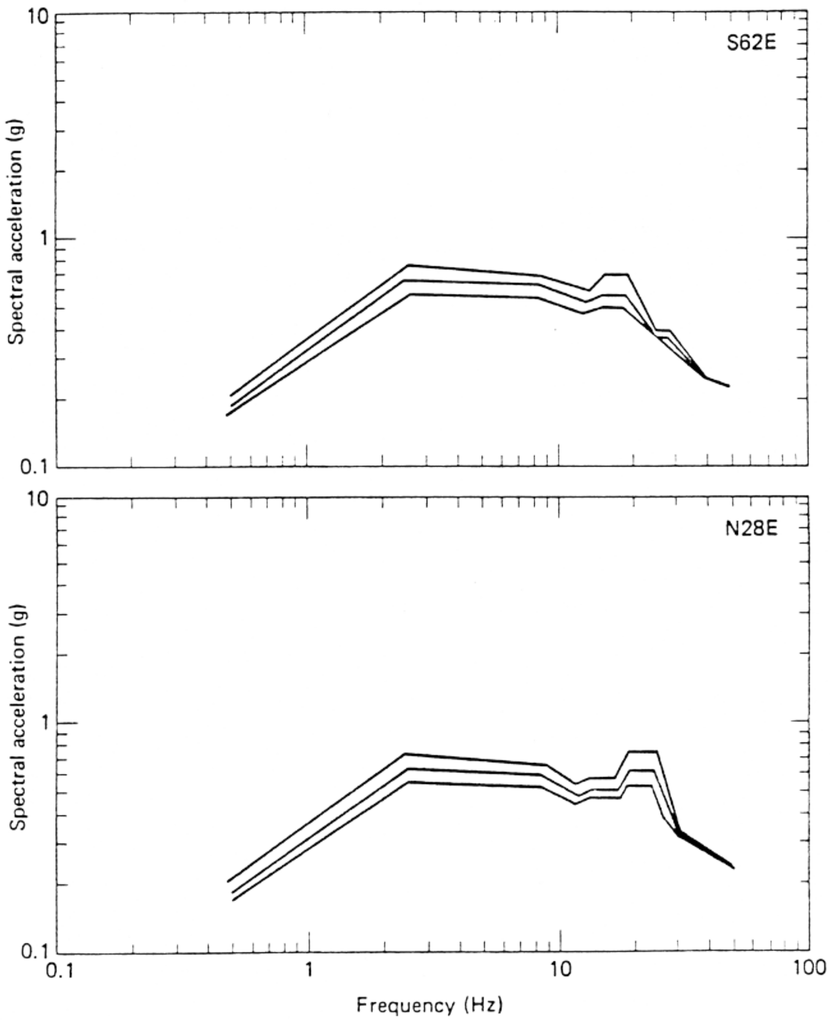

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Figure 3.7-18 SEP Response Spectra for Steam Generator SG-1B (Containment Building Elevation 278 ft) for 3%, 5%, and 7% Damping

Figure 3.7-19 SEP Response Spectra for Reactor Coolant Pump Rp-1A (Containment Building Elevation 247 ft) for 3%, 5%, and 7% Damping

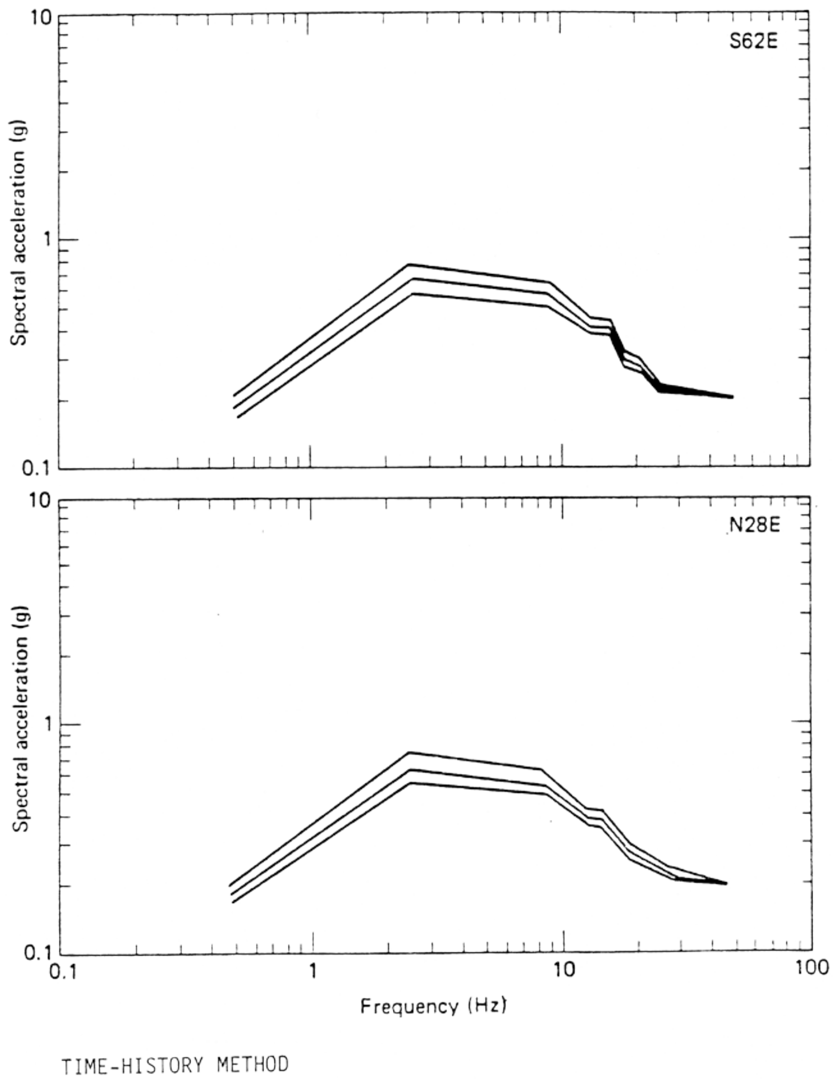

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Figure 3.7-19
SEP Response Spectra for Reactor Coolant Pump RP-1A (Containment Building Elevation 247 ft) for 3%, 5%, and 7% Damping

Figure 3.7-20 SEP Response Spectra for Reactor Coolant Pump RP-1B (Containment Building Elevation 247 ft) for 3%, 5%, and 7% Damping

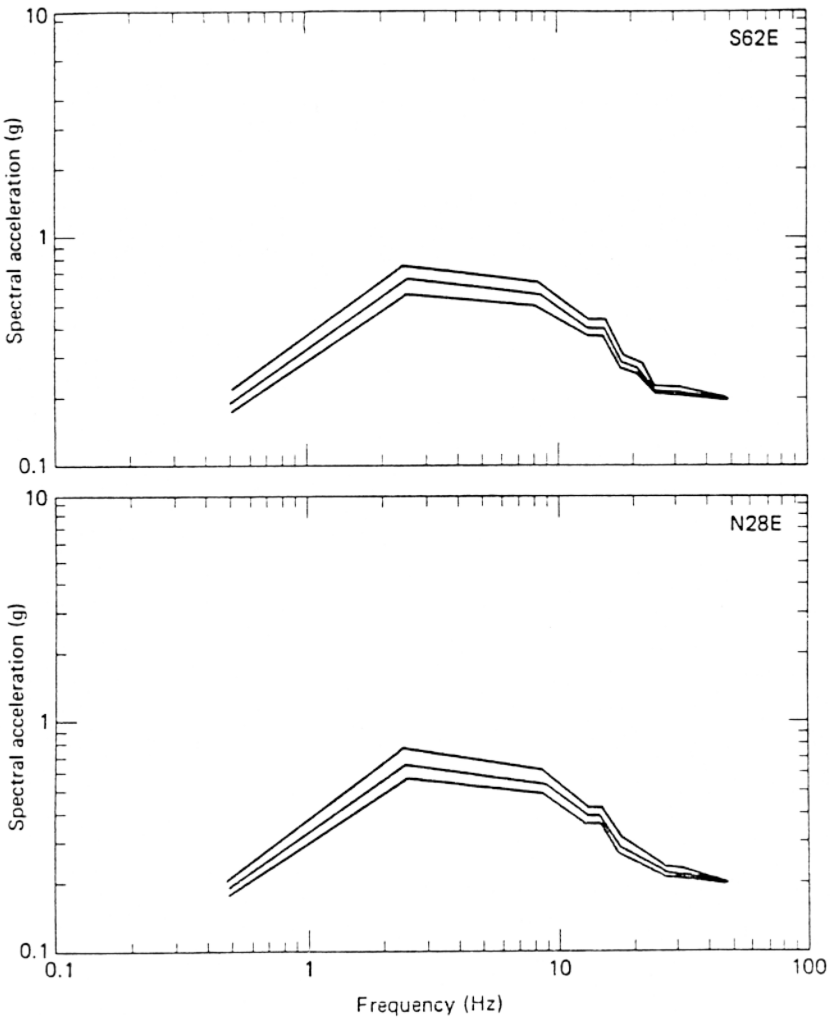

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Figure 3.7-20 SEP Response Spectra for Reactor Coolant Pump RP-1B (Containment Building Elevation 247 ft) for 3%, 5%, and 7% Damping

Figure 3.7-21 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Platform (Elevation 281 ft 6 in)

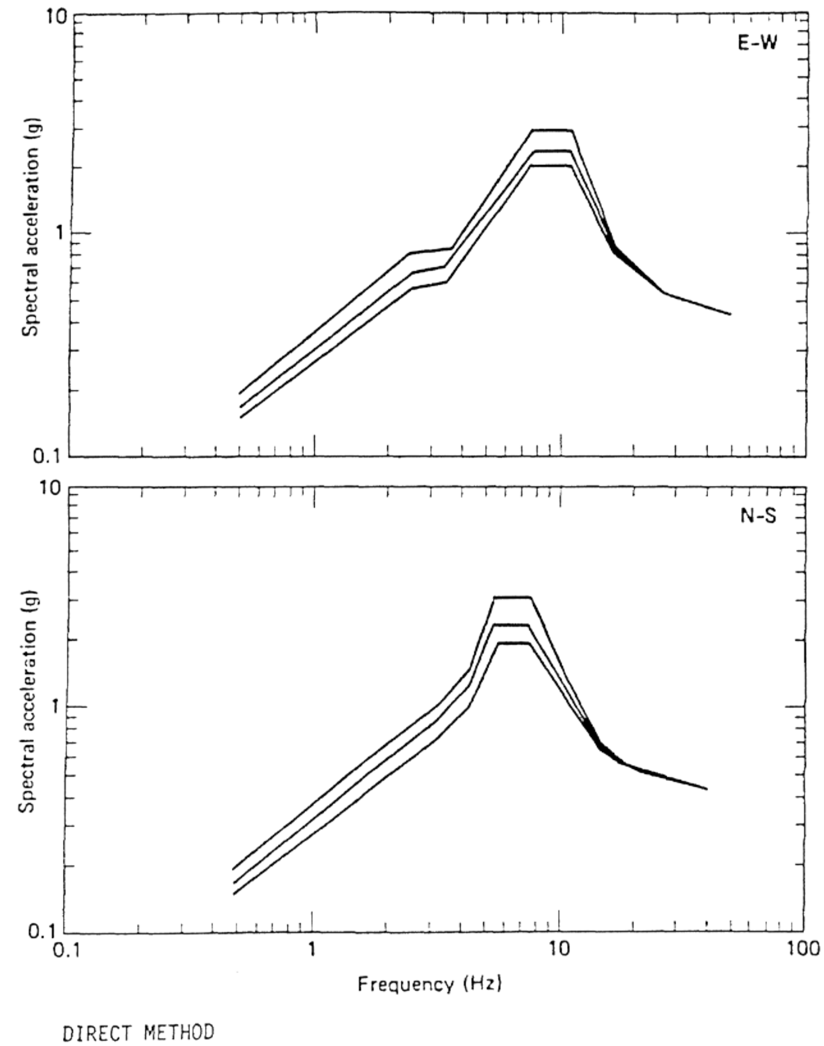

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Figure 3.7-21
SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Platform (Elevation 281 ft 6 in.)

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Figure 3.7-22 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Heat Exchanger 35 (Elevation 281 ft 6 in)

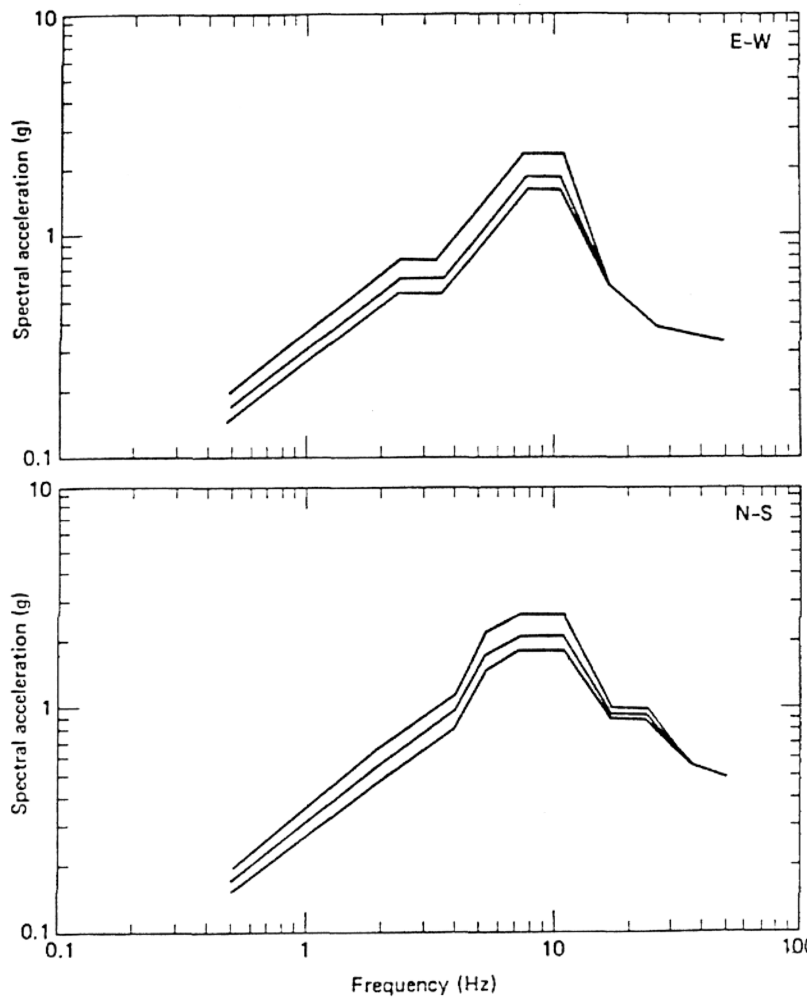

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Figure 3.7-22 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Heat Exchanger 35 (Elevation 281 ft 6 in.)

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Figure 3.7-23 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Surge Tank 34

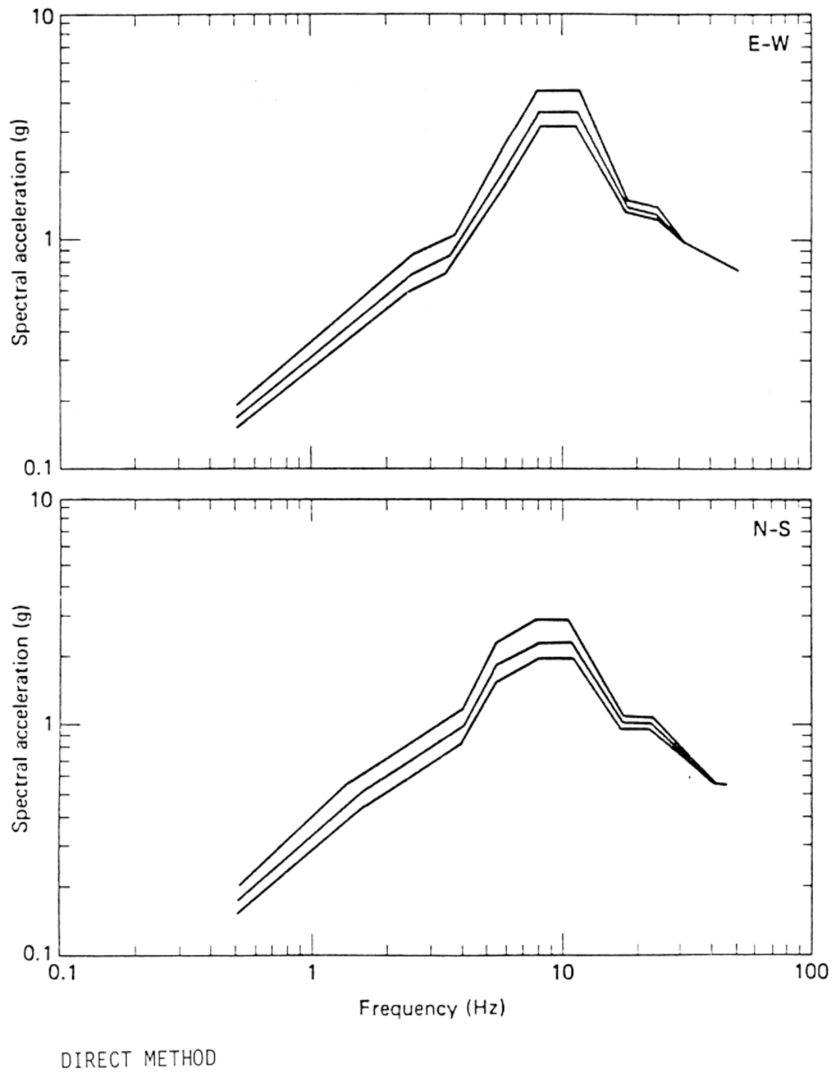

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Figure 3.7-23

SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Surge Tank 34

Figure 3.7-24 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Boric Acid Storage Tank 34

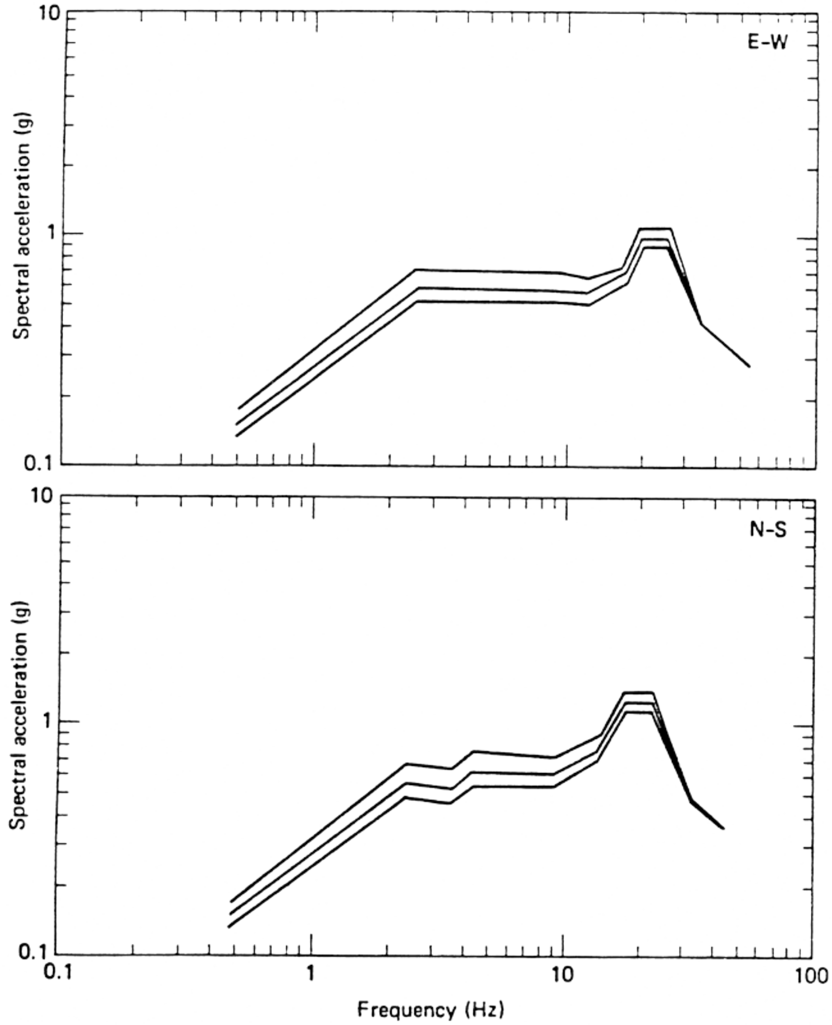

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Figure 3.7-24 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Boric Acid Storage Tank 34

REV 11 12/94

Figure 3.7-25 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Operating Floor (Elevation 271 ft 6 in)

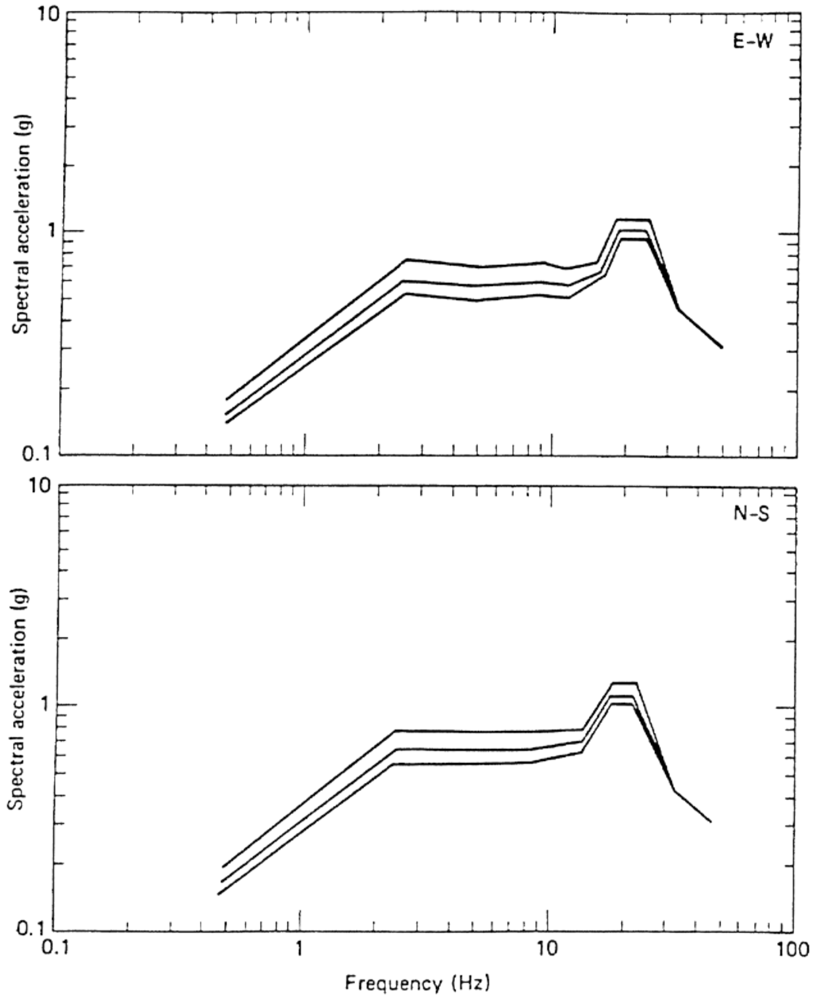

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Figure 3.7-25 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Auxiliary Building Operating Floor (Elevation 271 ft 6 in.)

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Figure 3.7-26 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Building Basement Floor (Elevation 250 ft 0 in)

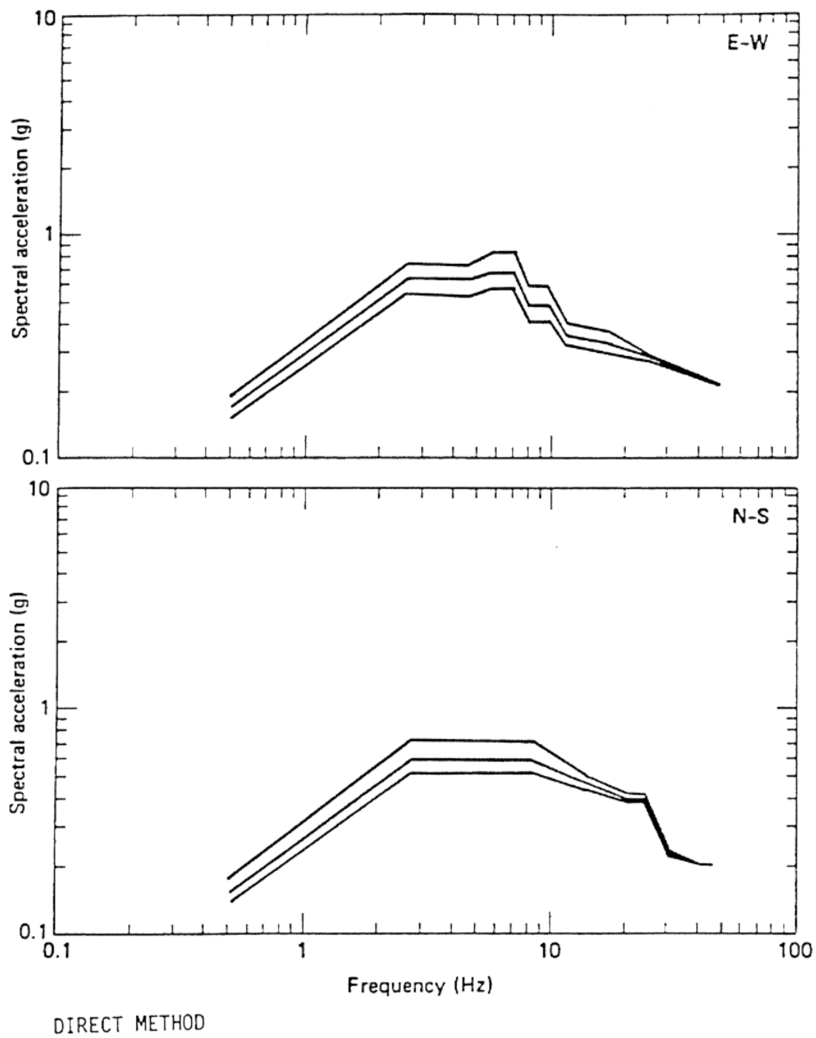

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Figure 3.7-26 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Building Basement Floor (Elevation 250 ft 0 in.)

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Figure 3.7-27 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Building Relay Room Floor (Elevation 269 ft 9 in)

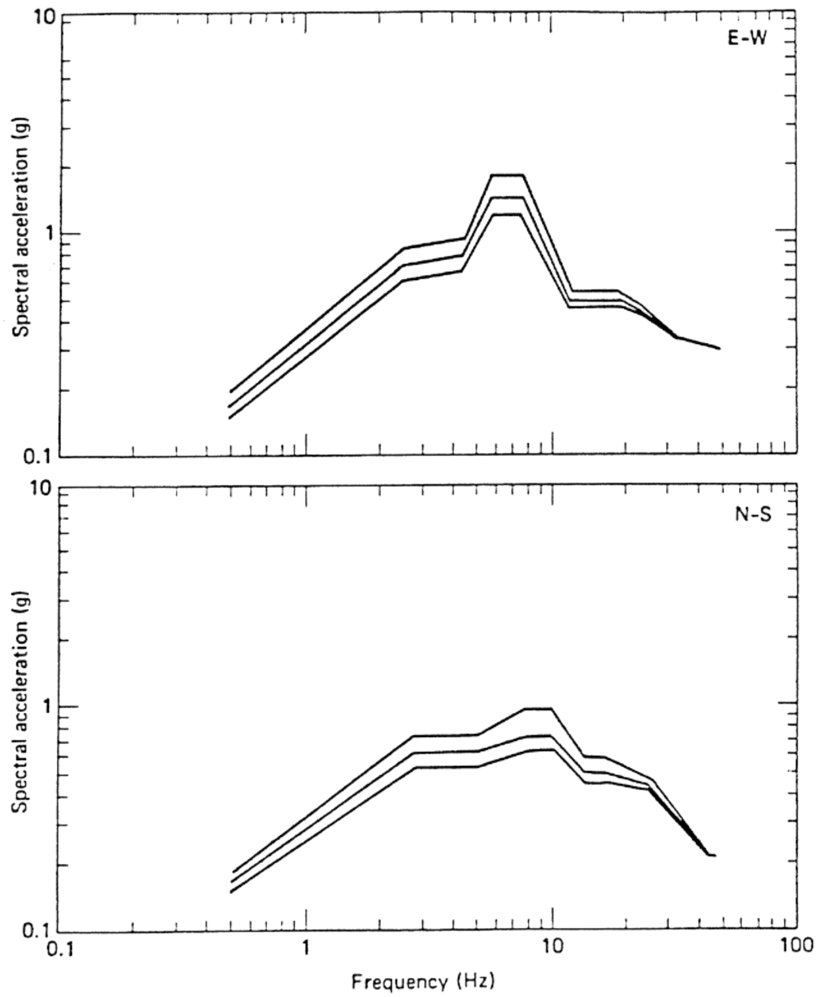

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Figure 3.7-27 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Building Relay Room Floor (Elevation 269 ft 9 in.)

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Figure 3.7-28 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Room Floor (Elevation 289 ft 9 in)

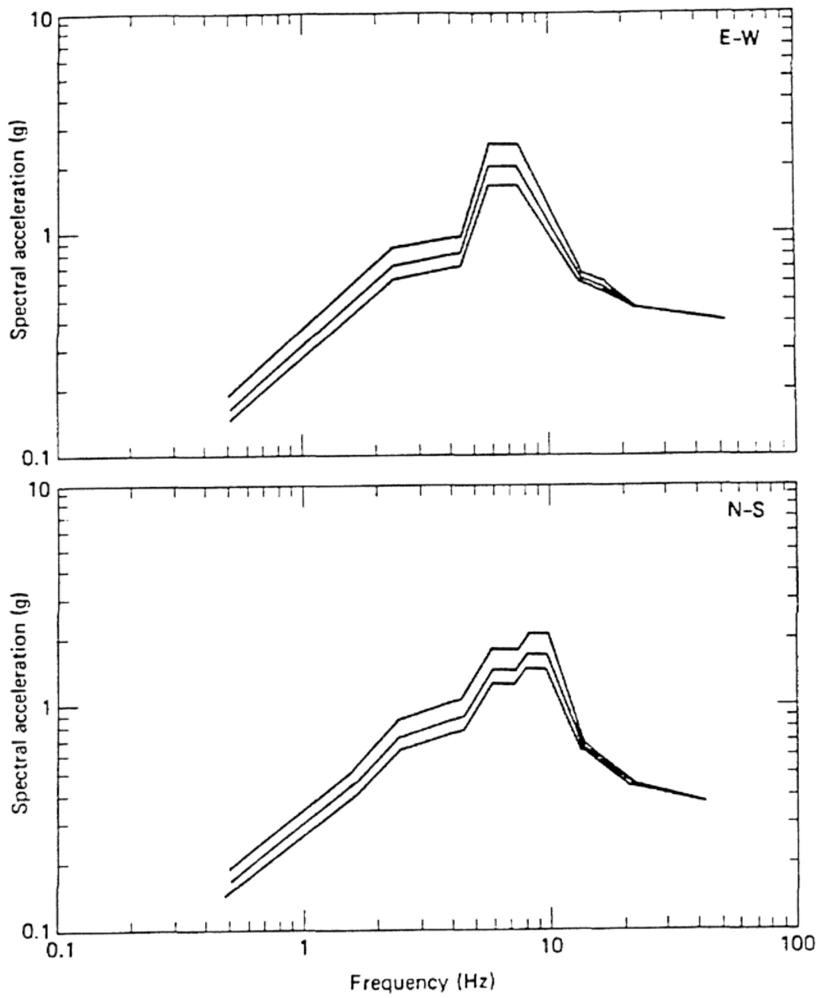

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Figure 3.7-28 SEP Equipment Response Spectra for 3%, 5%, and 7% Damping at Control Room Floor (Elevation 289 ft 9 in.)

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Figure 3.7-29 Residual Heat Removal Line Inside Containment

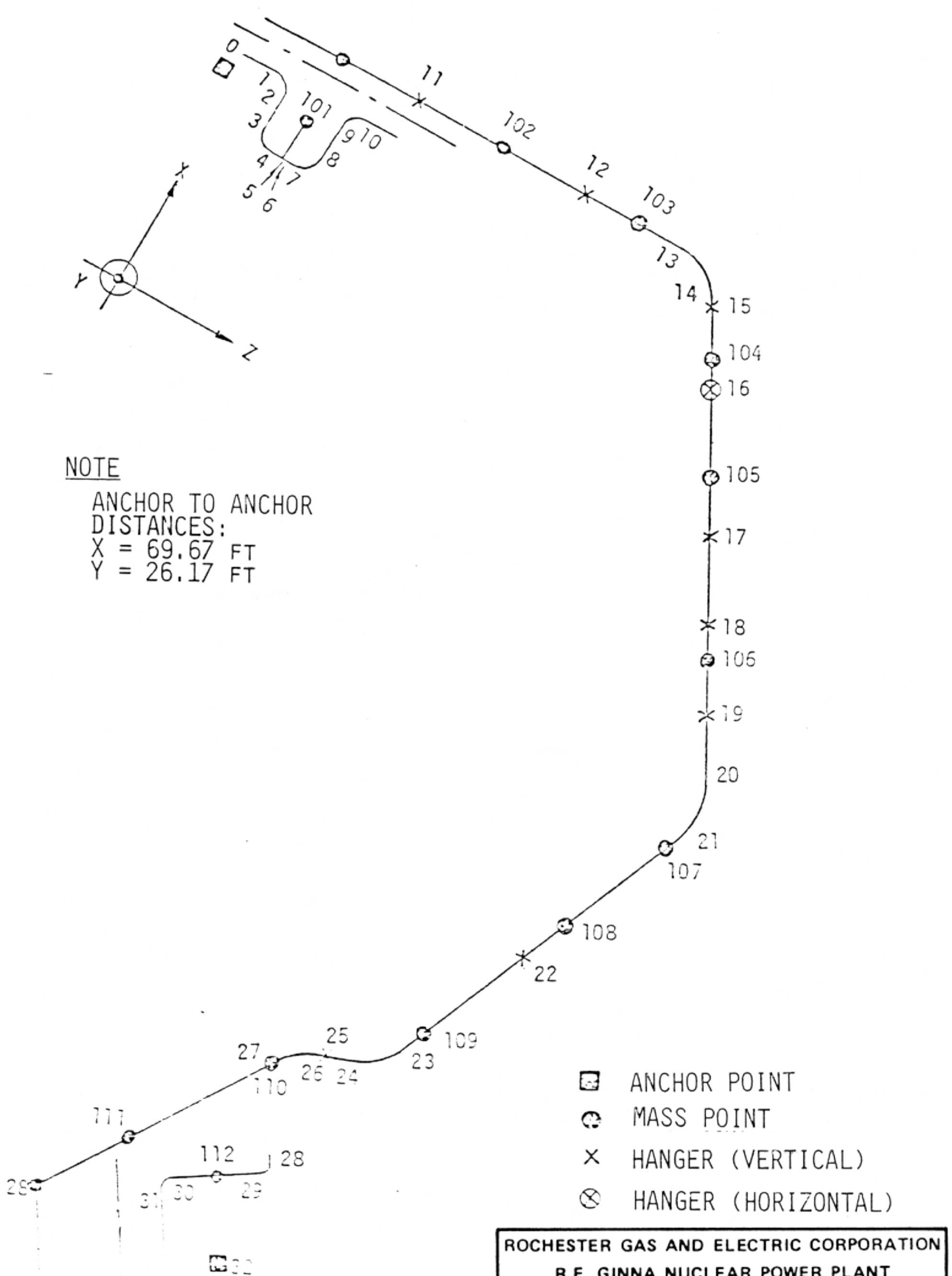

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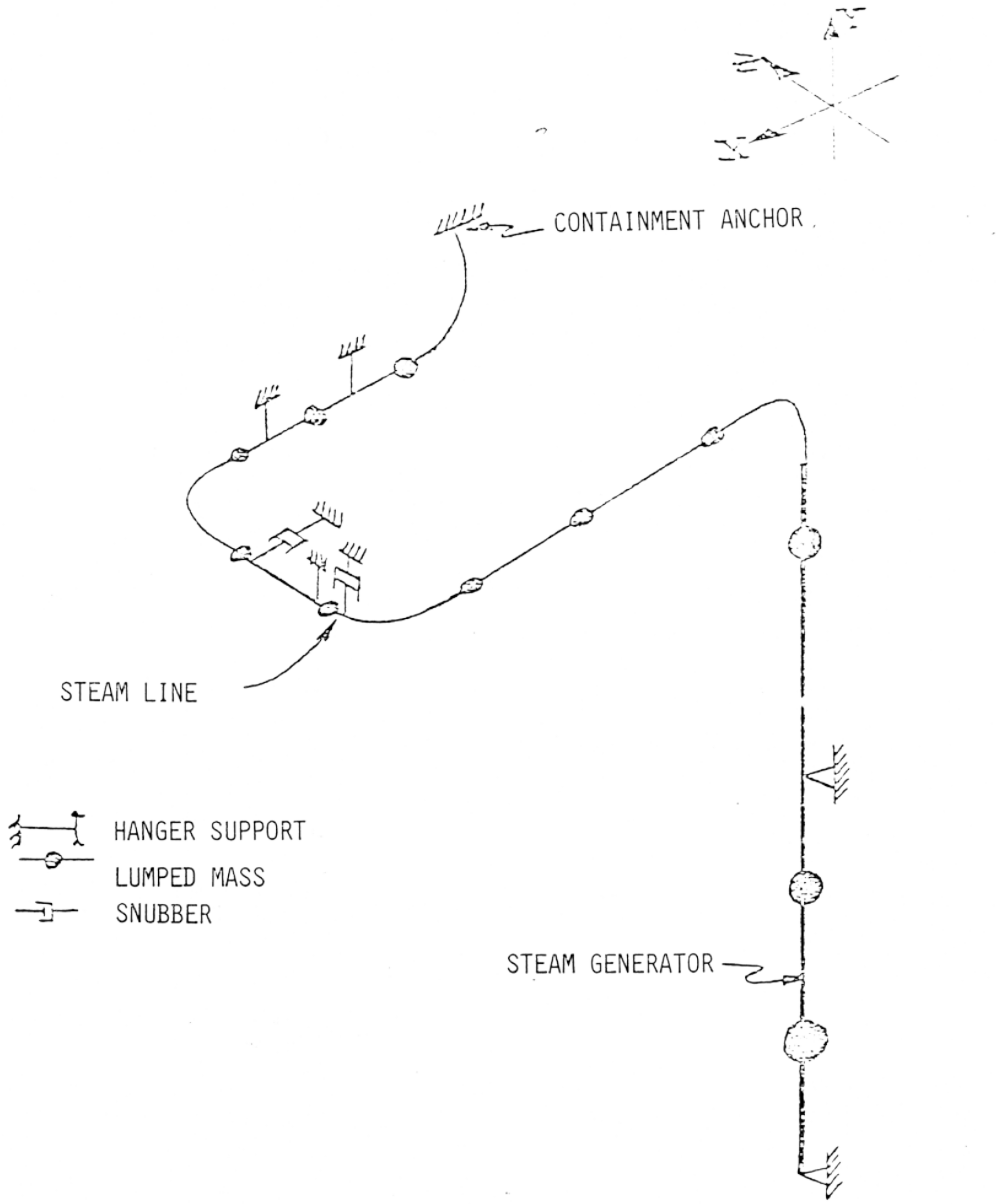
Figure 3.7-29

Residual Heat Removal Line Inside Containment

Figure 3.7-30 Lumped Mass Model - Steam Line B



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UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.7-30
Lumped Mass Model - Steam Line B

Figure 3.7-31 Structural Model, Pressurizer Safety and Relief Line

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Refer to Drawings
C381-0353,4, 5, 6

Sheet 2 of Figure 3.7-31

Figure Deleted

Refer to Drawings
C381-0353,4, 5, 6

Sheet 3 of Figure 3.7-31

Figure Deleted

Refer to Drawings
C381-0353,4, 5, 6

Sheet 4 of Figure 3.7-31

Figure Deleted

Refer to Drawings
C381-0353,4, 5, 6

Sheet 5 of Figure 3.7-31

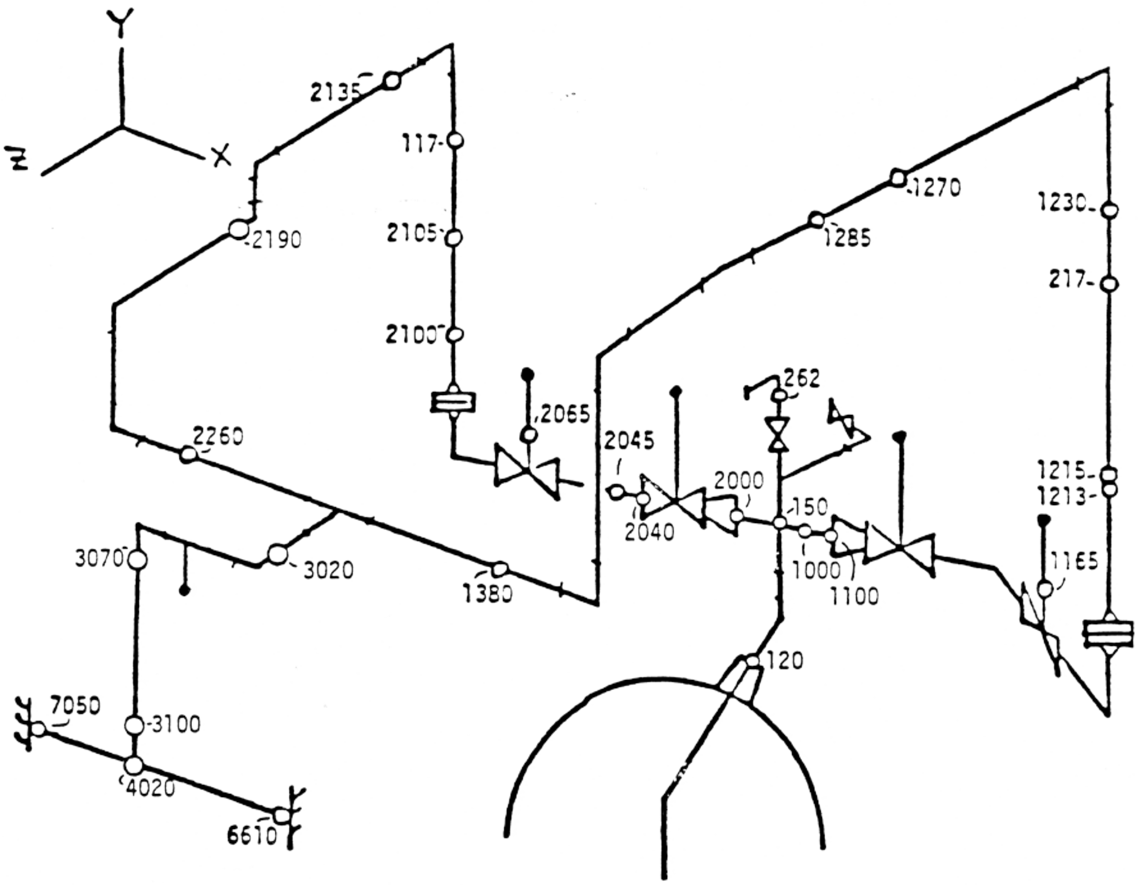

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Figure 3.7-31, Sheet 5

Structural Model, Pressurizer Safety and Relief Line

Figure 3.8-1 Containment Cross Section and Details

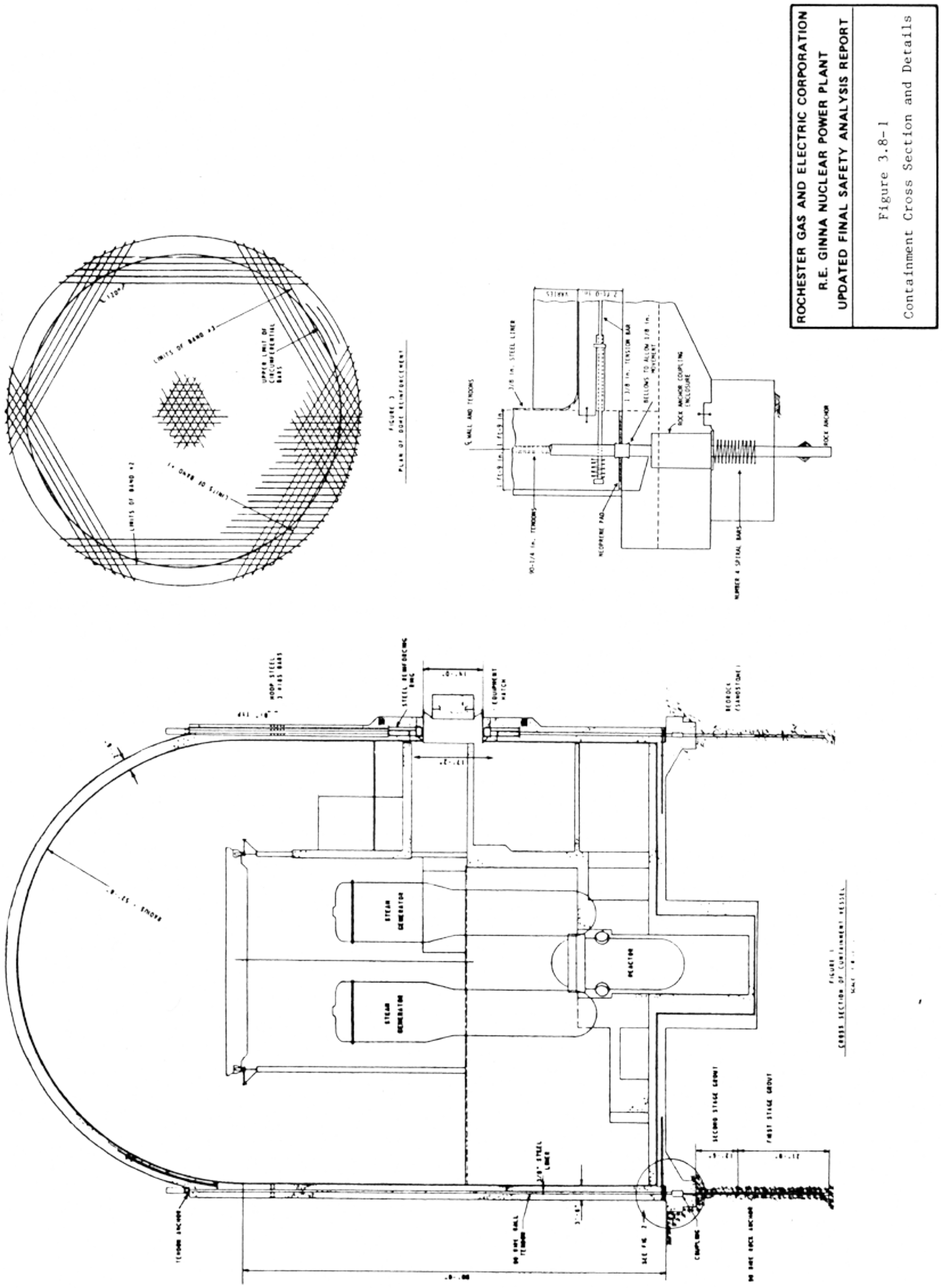

Figure 3.8-2 Containment Mat Foundation and Ring Girder

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Refer to Drawing
D421-0001 Rev. 006

Figure 3.8-3 Containment Mat Foundation, Reinforcement and Details

Figure Deleted

Refer to Drawing
D421-0002 Rev. 002

Figure 3.8-4 Containment Wall Reinforcement and Details

Figure Deleted

Refer to Drawing
D421-0007 Rev. 006

Figure 3.8-5 Containment Dome Reinforcement and Details

Figure Deleted

Refer to Drawing
D421-0021 Rev. 004

Figure 3.8-6 Containment Miscellaneous Embedded Back-Up Steel

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Refer to Drawing
D521-0041 Rev. 002

Figure 3.8-7 Tendon Vent Cans and Grease Fill Connections

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Refer to Drawing
D326-0023 Rev. 004

Figure 3.8-8 Temperature Gradients - Operating Conditions

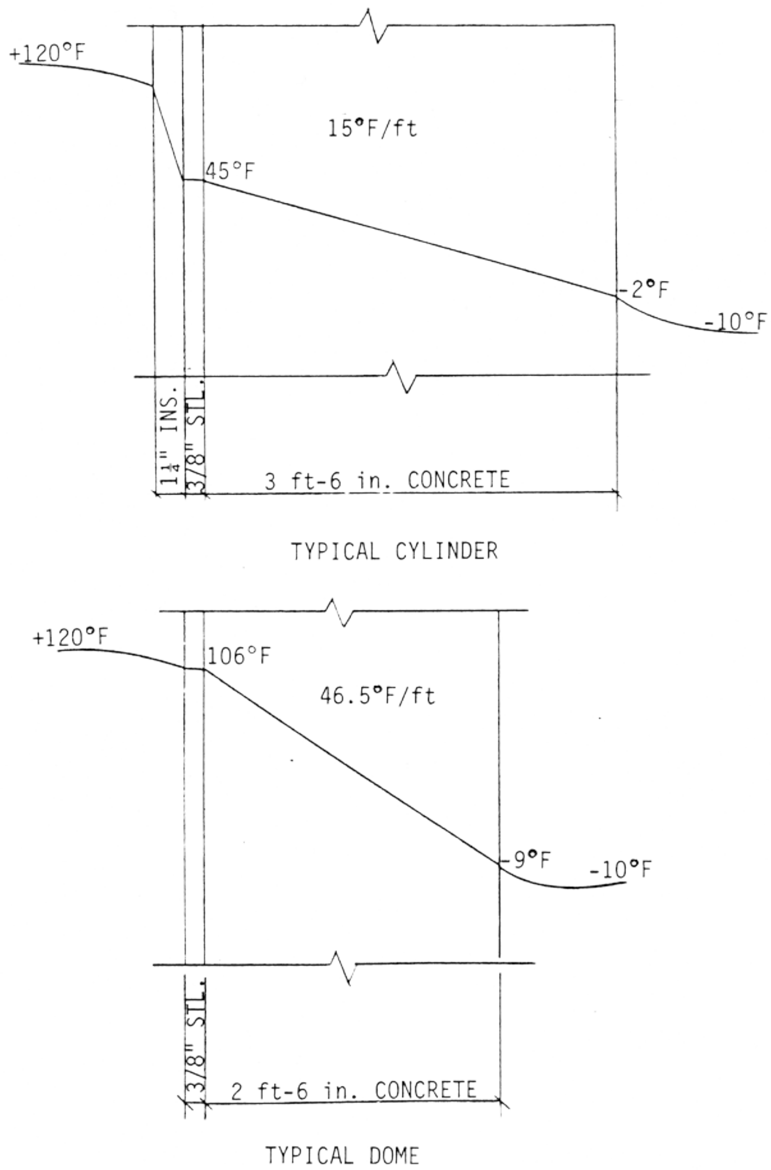

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Figure 3.8-8

Temperature Gradients - Operating Conditions

Figure 3.8-9 Earthquake Meridional Forces

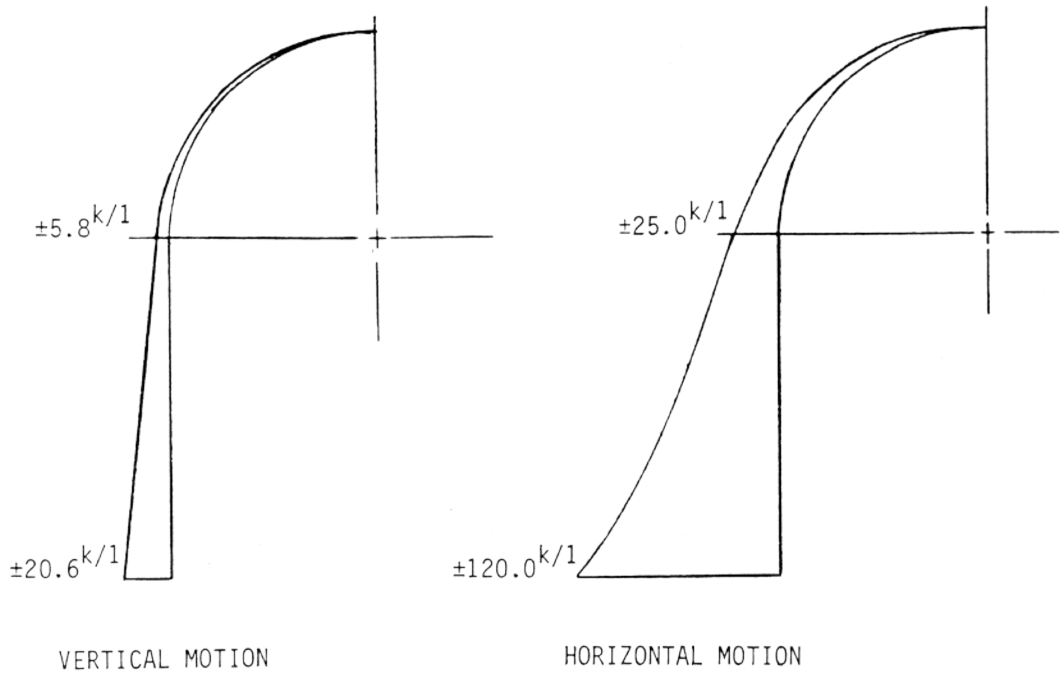

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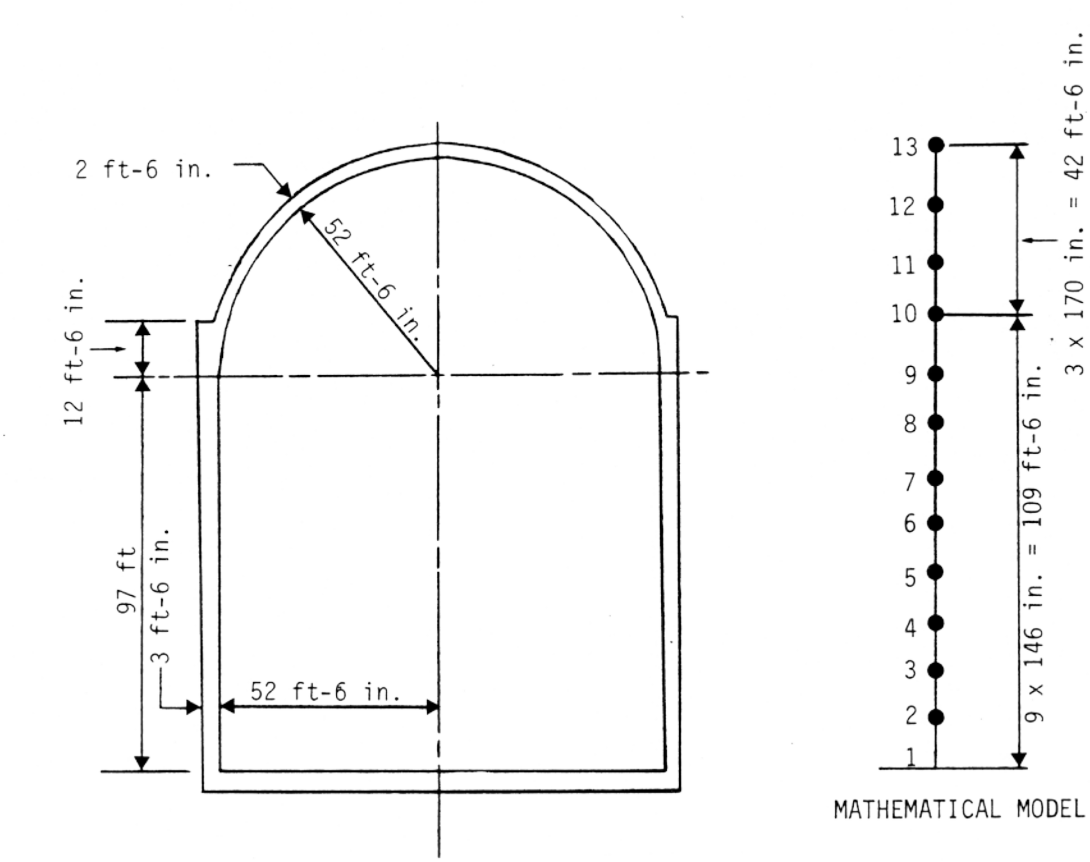
Figure 3.8-9
Earthquake Meridional Forces

Figure 3.8-10 Containment Dynamic Analysis Model



CONCRETE CYLINDER STRENGTH = 5000 psi

TAKE $E = W^{1.5}\ 33\ \sqrt{fi} = 145^{1.5} \times 33 \times \sqrt{5000}$

$= 4{,}070{,}000$ psi

POISSON'S RATIO = 0.15

DENSITY OF REINFORCED CONCRETE = 160 lb/ft^3

$= .092\ lb/in.^3$

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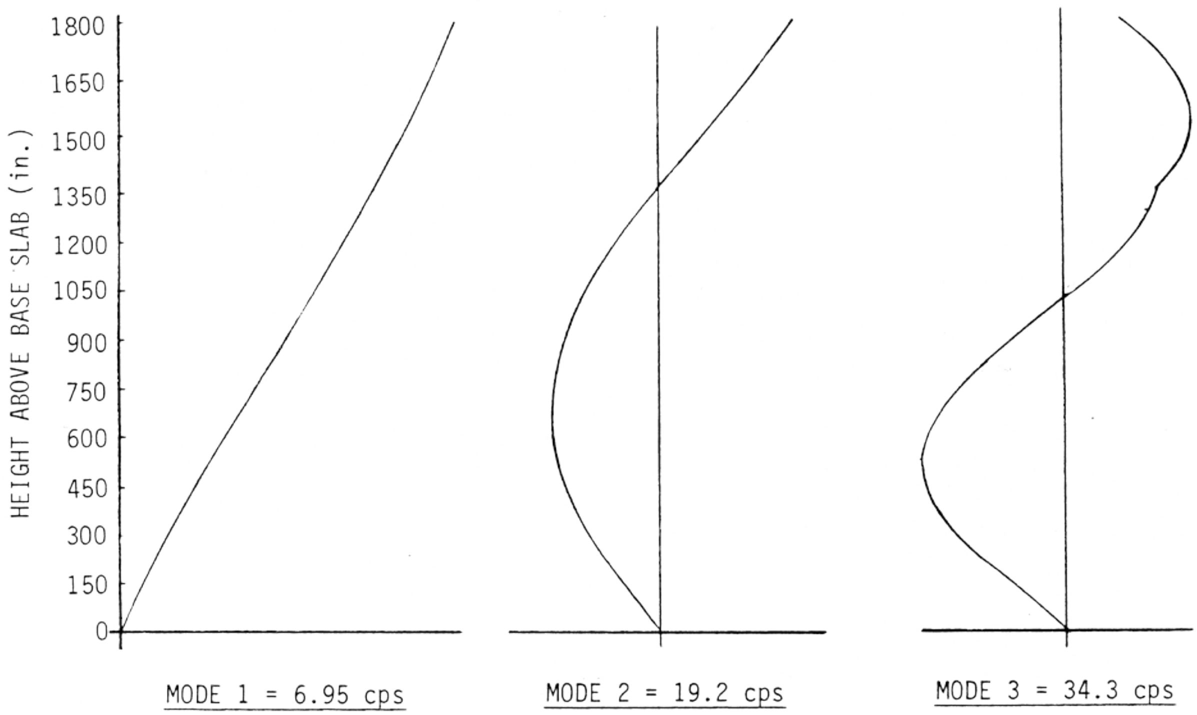
Figure 3.8-10
Containment Dynamic Analysis Model

Figure 3.8-11 Ginna Containment Mode Shapes



NOTE: $E = 4.07 \times 10^6$ psi
$\gamma = 0.15$
$e = 160$ lb/w ft

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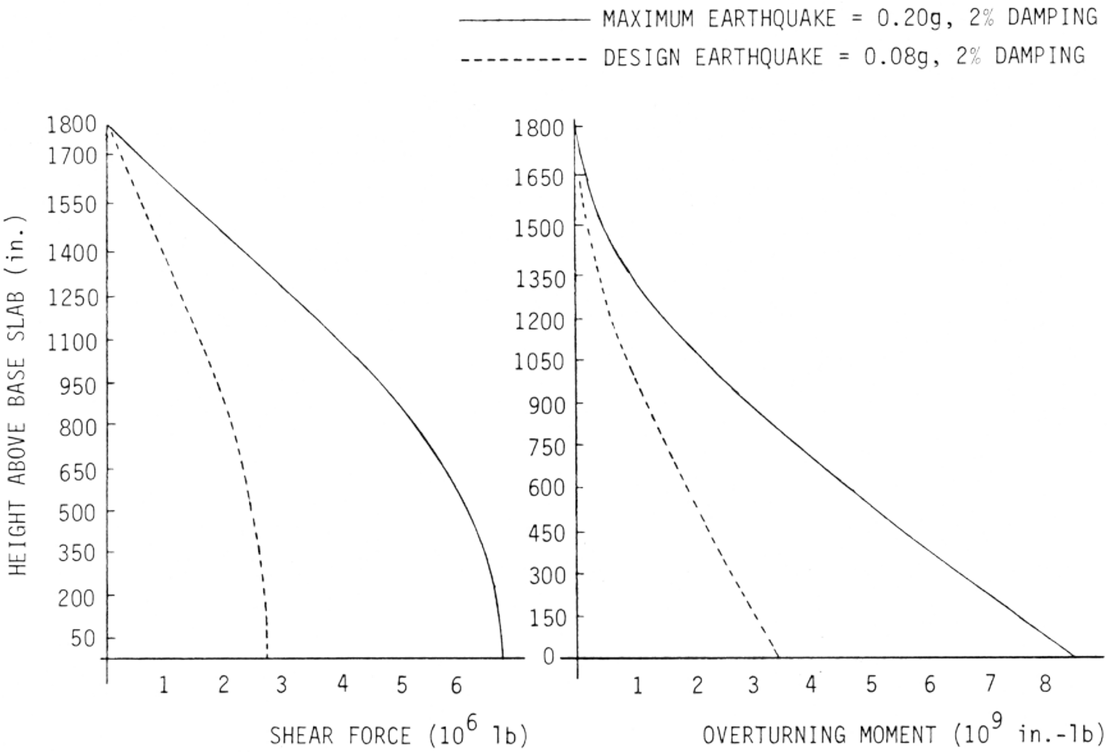
Figure 3.8-11
Ginna Containment Mode Shapes

Figure 3.8-12 Ginna Containment - Earthquake Response



NOTE: $E = 4.07 \times 10^6$ psi

$\gamma = 0.15$

e = 160 lb/w ft

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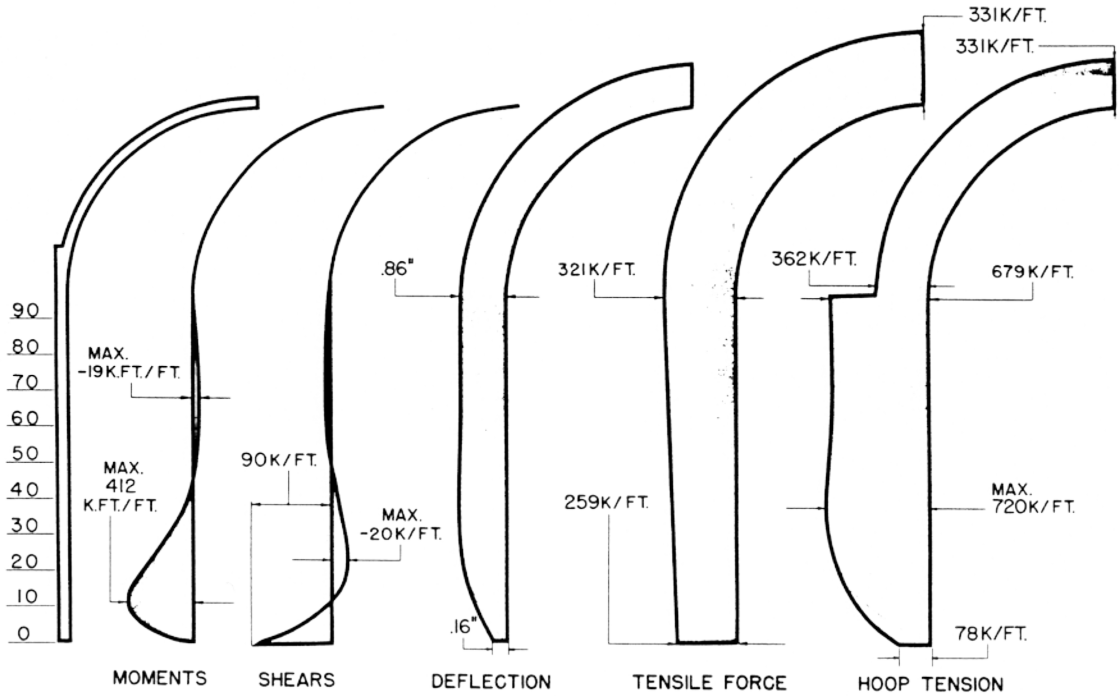
Figure 3.8-12

Ginna Containment - Earthquake Response

Figure 3.8-13 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination A



C = .95 D + 1.5 P + 1.0 T

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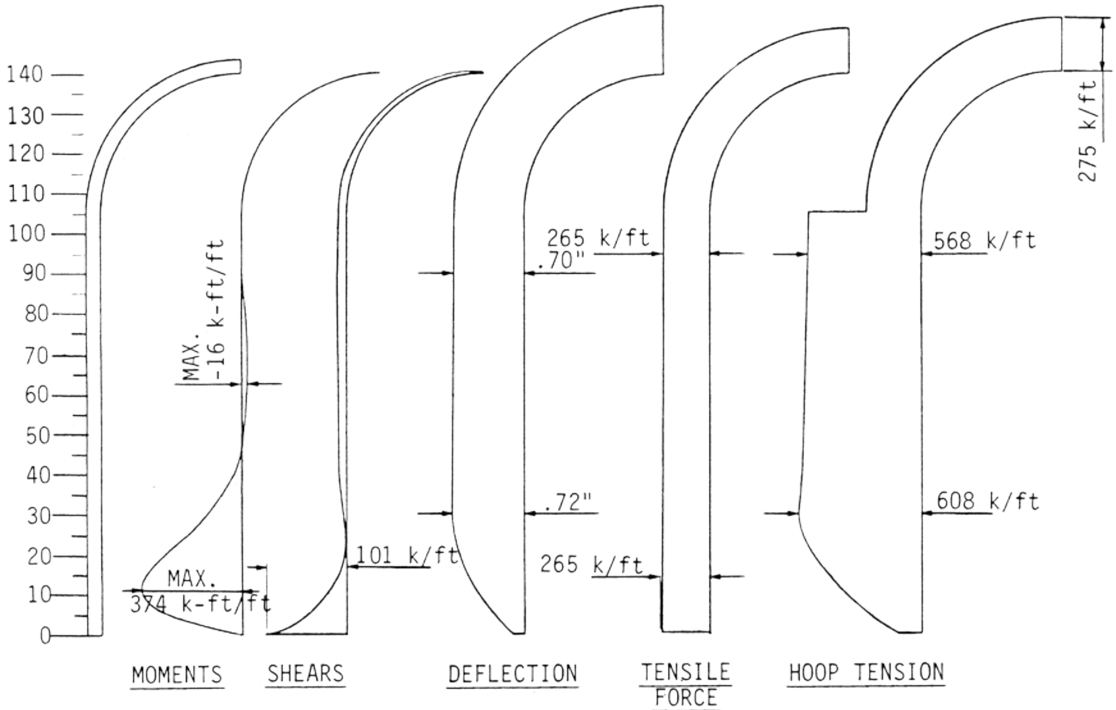
Figure 3.8-13 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination A

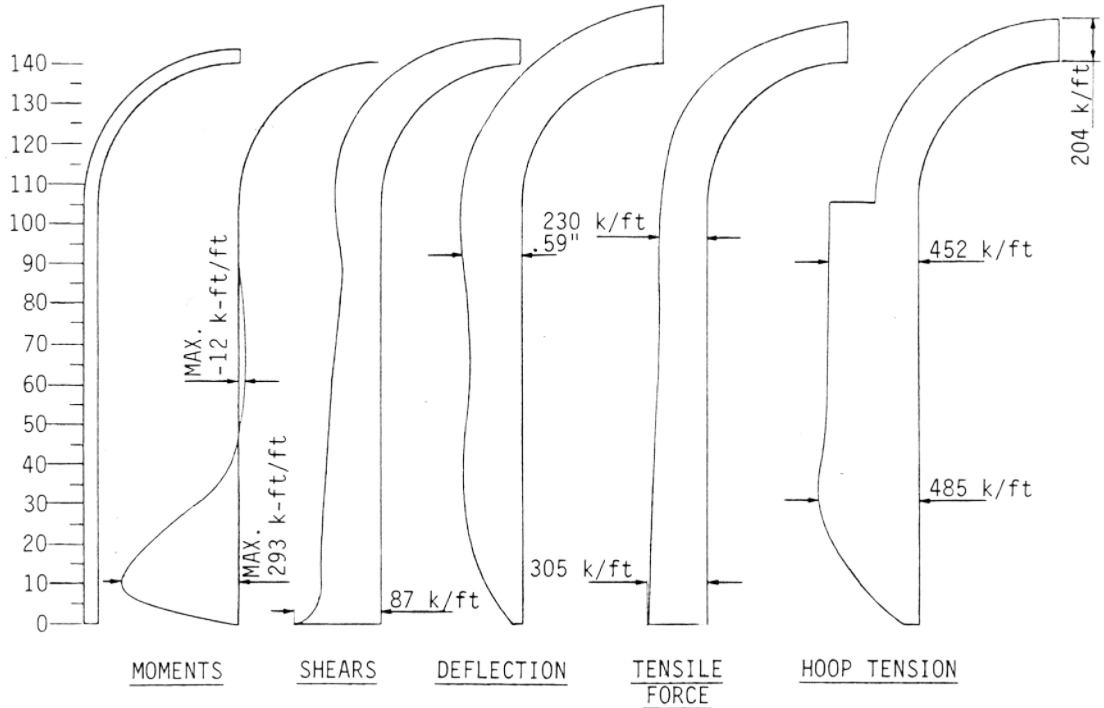
Figure 3.8-14 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination B



C = .95 D + 1.25 P + 1.0 T' + 1.25 E

ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 3.8-14 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination B

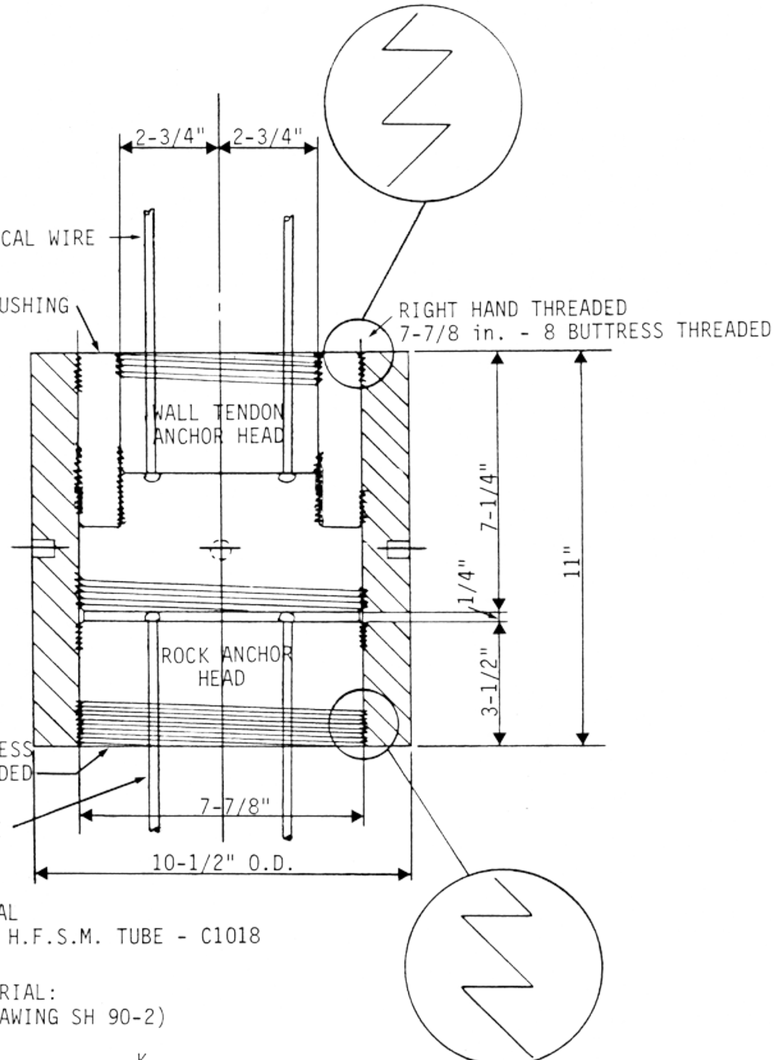
Figure 3.8-15 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination C



C = .95 D + 1.0 P + 1.0 T + 1.0 E'

ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 3.8-15 Moments, Shears, Deflection, Tensile Force, and Hoop Tension Diagrams Load Combination C

Figure 3.8-16   Tendon to Rock Coupling



COUPLING MATERIAL
10-1/2 in. O.D. H.F.S.M. TUBE - C1018

ANCHOR HEAD MATERIAL:
(REF. RYERSON DRAWING SH 90-2)

JACKING FORCE 847^K

EFFECTIVE PRESTRESS FORCE 635^K

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Figure 3.8-16 Tendon to Rock Coupling

Figure 3.8-17   Containment - Top Tendon Access

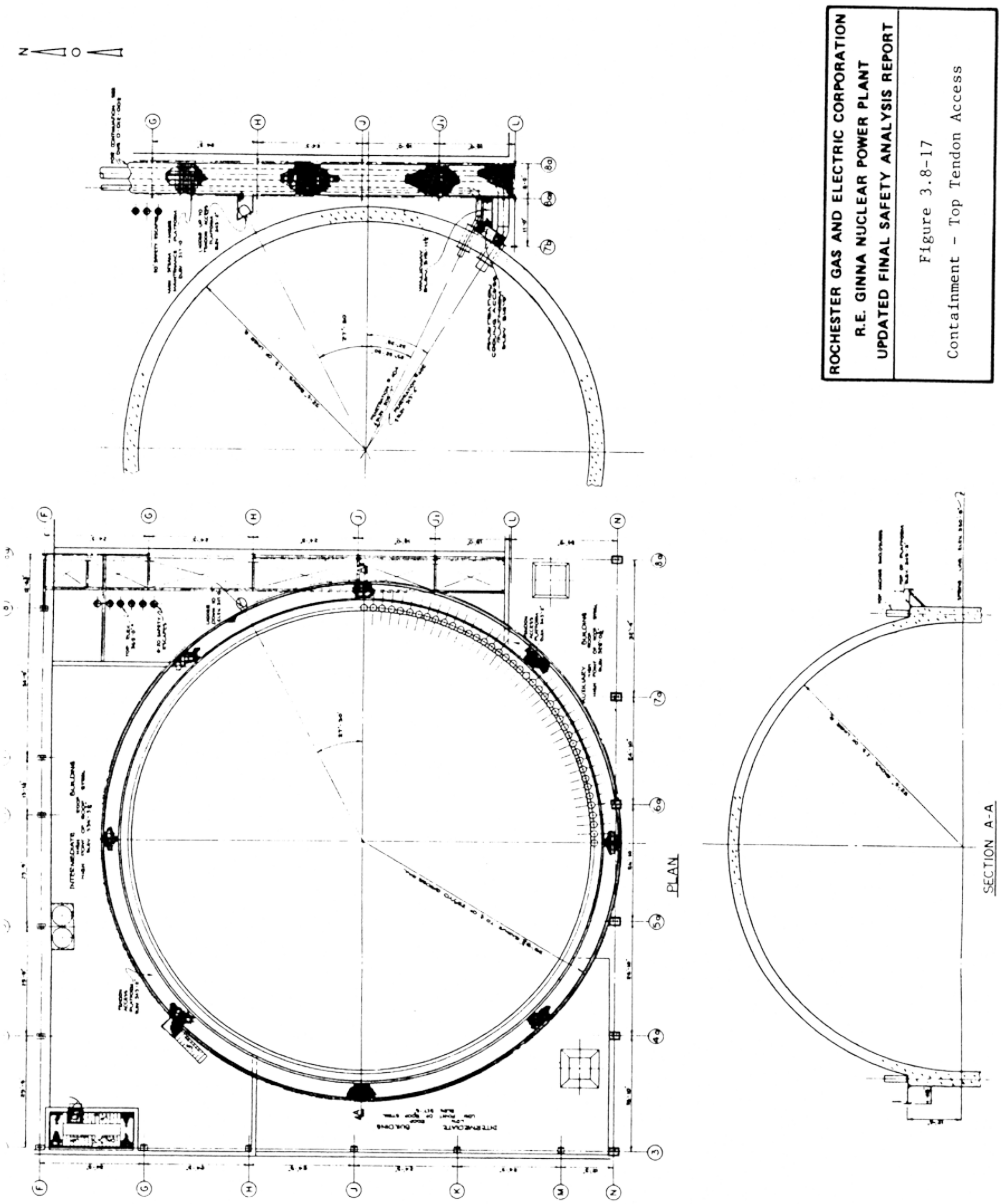
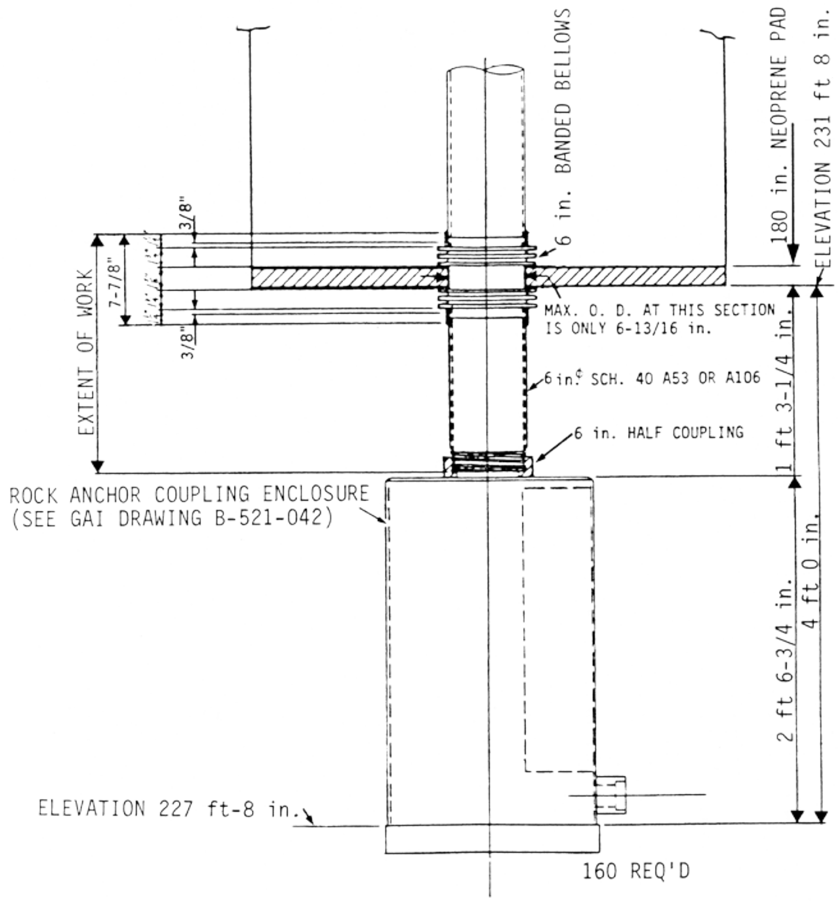


Figure 3.8-18 Containment Miscellaneous Steel Tendon Conduit - Hinge Detail



ENGINEERING DATA

1. MOVEMENTS: CASE (1) FROM UNDEFLECTED POSITION VERTICALLY DOWNWARD 0.14 INCHES.
   CASE (2) FROM ABOVE POSITION VERTICALLY UPWARD 0.10 INCHES AND SIMULTANEOUS LATERALLY 0.16 INCHES.
2. FATIGUE: TWO CYCLES PER YEAR.
3. WORKING PRESSURE: 60 psig.
   TEST PRESSURE: HYDROSTATIC AT 150% OF WORKING PRESSURE.
   PNEUMATIC AT 125% OF WORKING PRESSURE.
4. MAXIMUM WORKING TEMPERATURE: 160°F.
5. STANDARD SPECIFICATION: ASA B31.1 CODE FOR PRESSURE PIPING.
6. TEST TWO RANDOM ASSEMBLIES FOR SPECIFIED MOVEMENTS.

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Figure 3.8-18

Containment Miscellaneous Steel Tendon Conduit - Hinge Detail

Figure 3.8-19 Liner Knuckle Dimensions

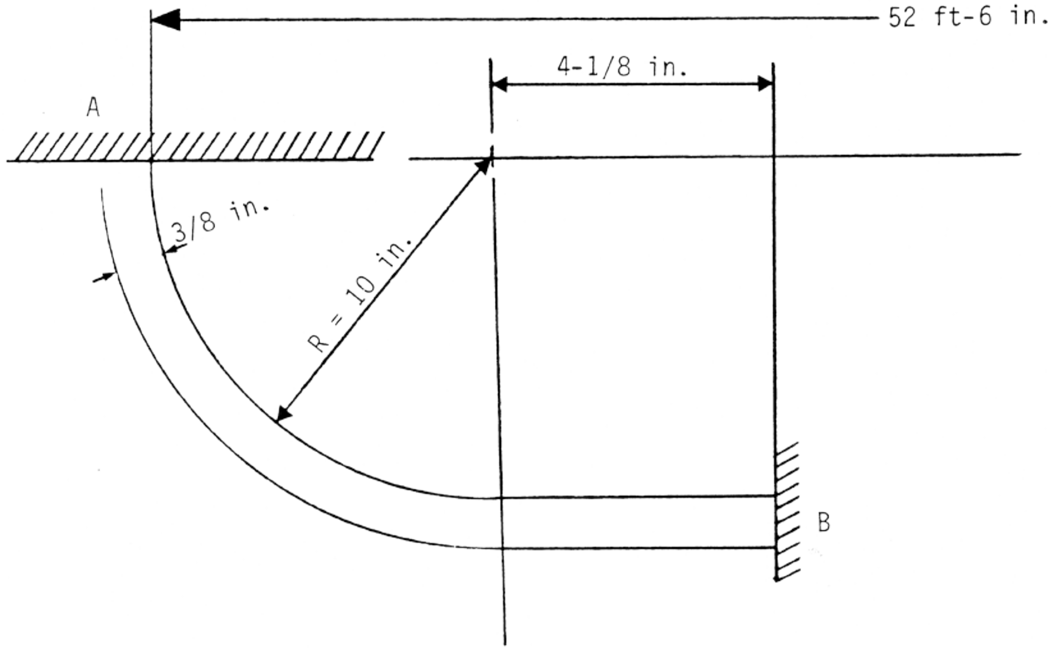

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Figure 3.8-19
Liner Knuckle Dimensions

Figure 3.8-20 Containment Base to Cylinder Model

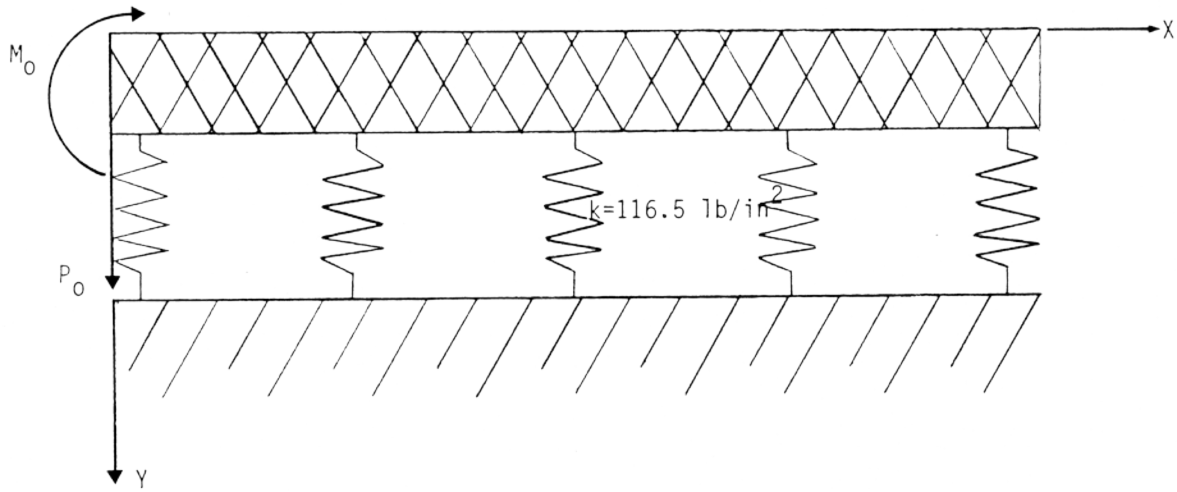

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Figure 3.8-20
Containment Base to Cylinder Model

Figure 3.8-21 Containment Dome to Cylinder Discontinuity Model

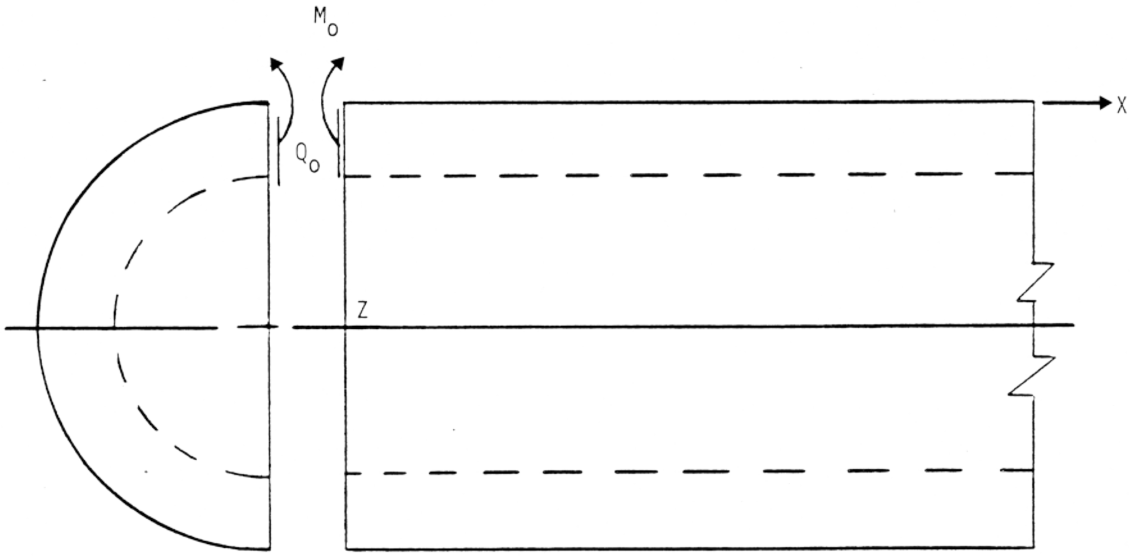

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Figure 3.8-21

Containment Dome to Cylinder Discontinuity Model

Figure 3.8-22 Cracked Wall Shear Modulus Analysis

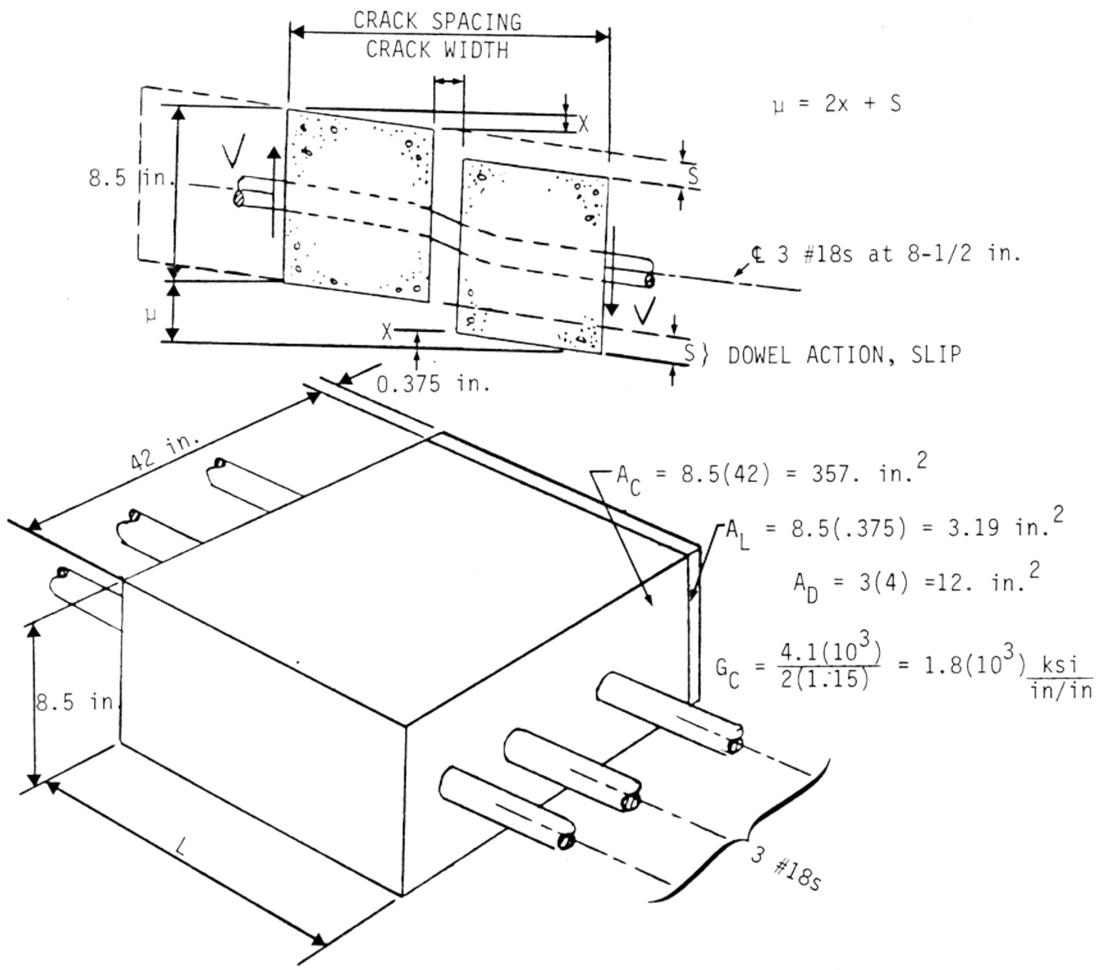

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Figure 3.8-22
Cracked Wall Shear Modulus Analysis

Figure 3.8-23 Liner Shear Stress Analysis

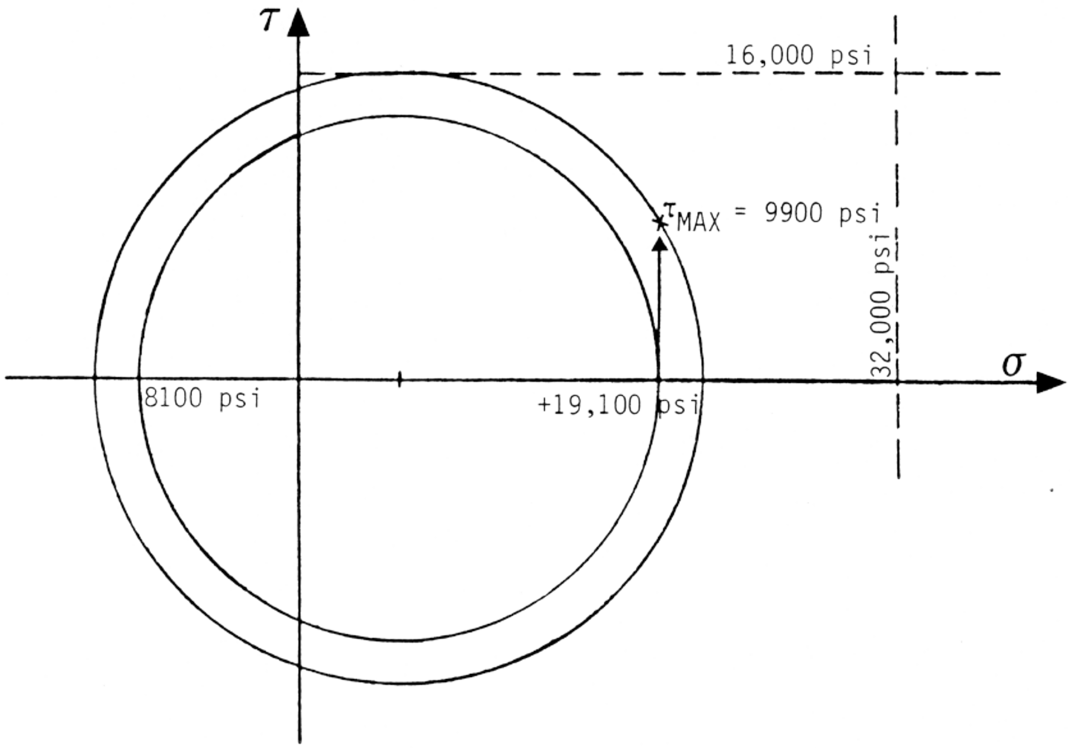

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Figure 3.8-23
Liner Shear Stress Analysis

Figure 3.8-24 Windgirder, Shear Channels, and Shear Studs

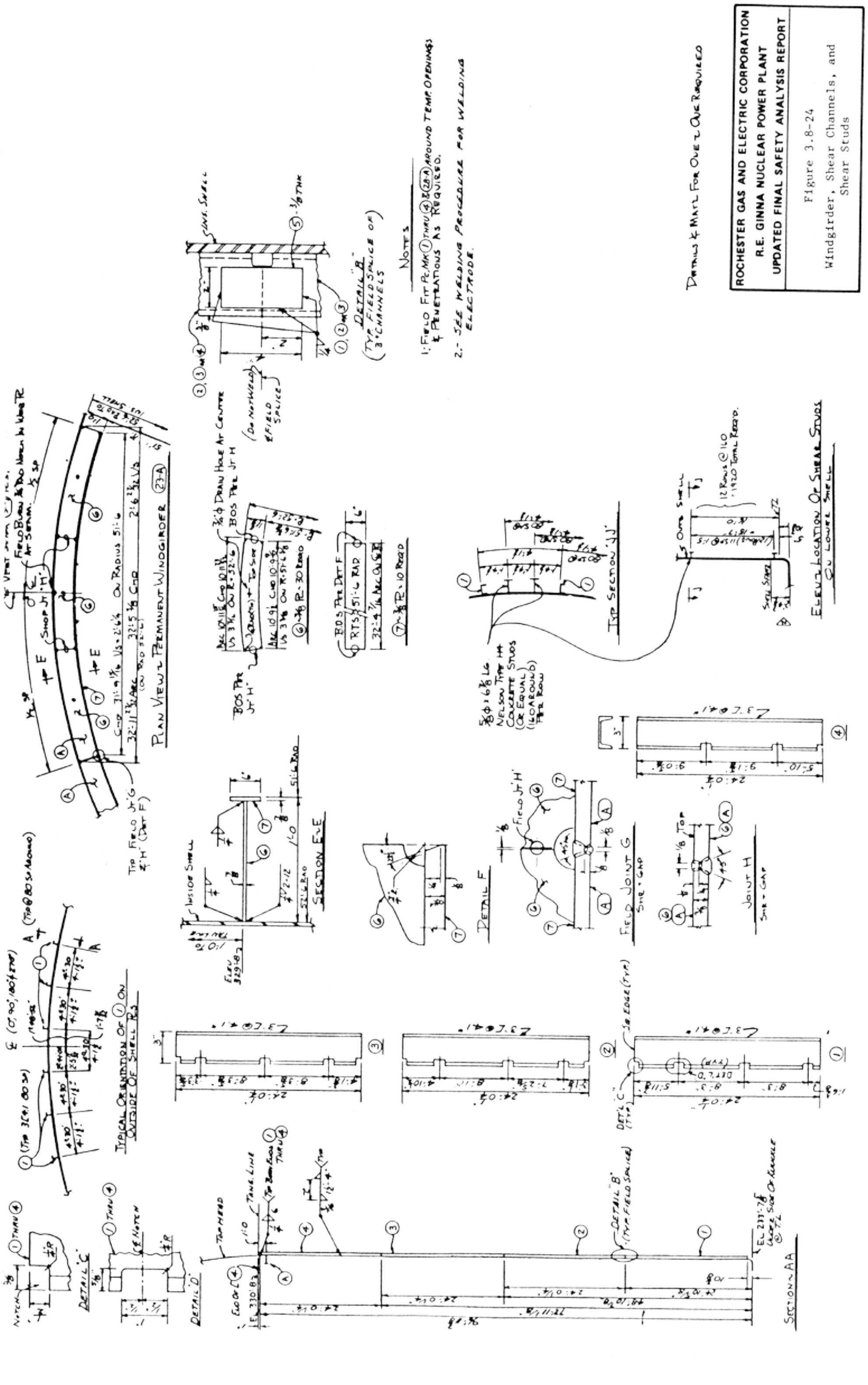

Figure 3.8-25 Cylinder Liner Plate Support Model

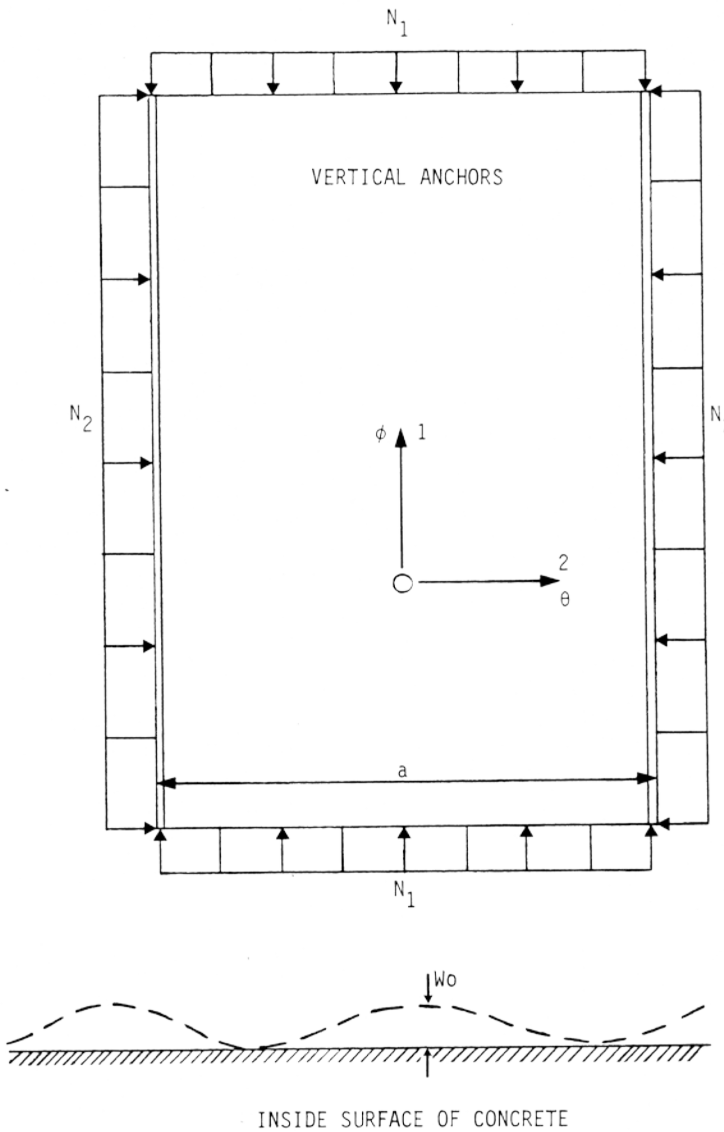

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Figure 3.8-25
Cylinder Liner Plate Support Model

Figure 3.8-26 Containment Penetration Details

Figure Deleted

Refer to Drawing
D521-0055 Rev. 007

Figure 3.8-27 Containment Penetration Details (Typical)

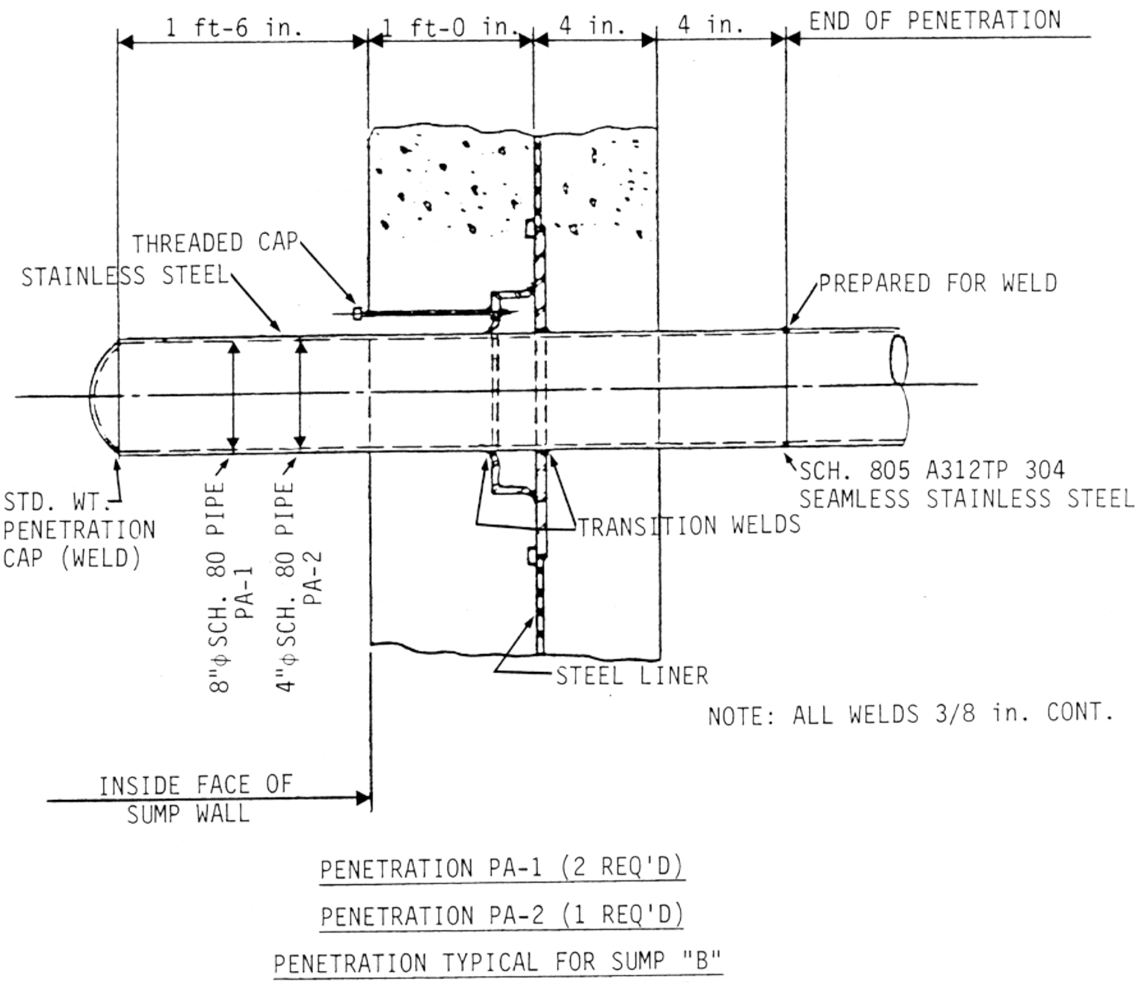

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Figure 3.8-27
Containment Penetration Details (Typical)

Figure 3.8-28 Composite Drawing Electrical Penetration

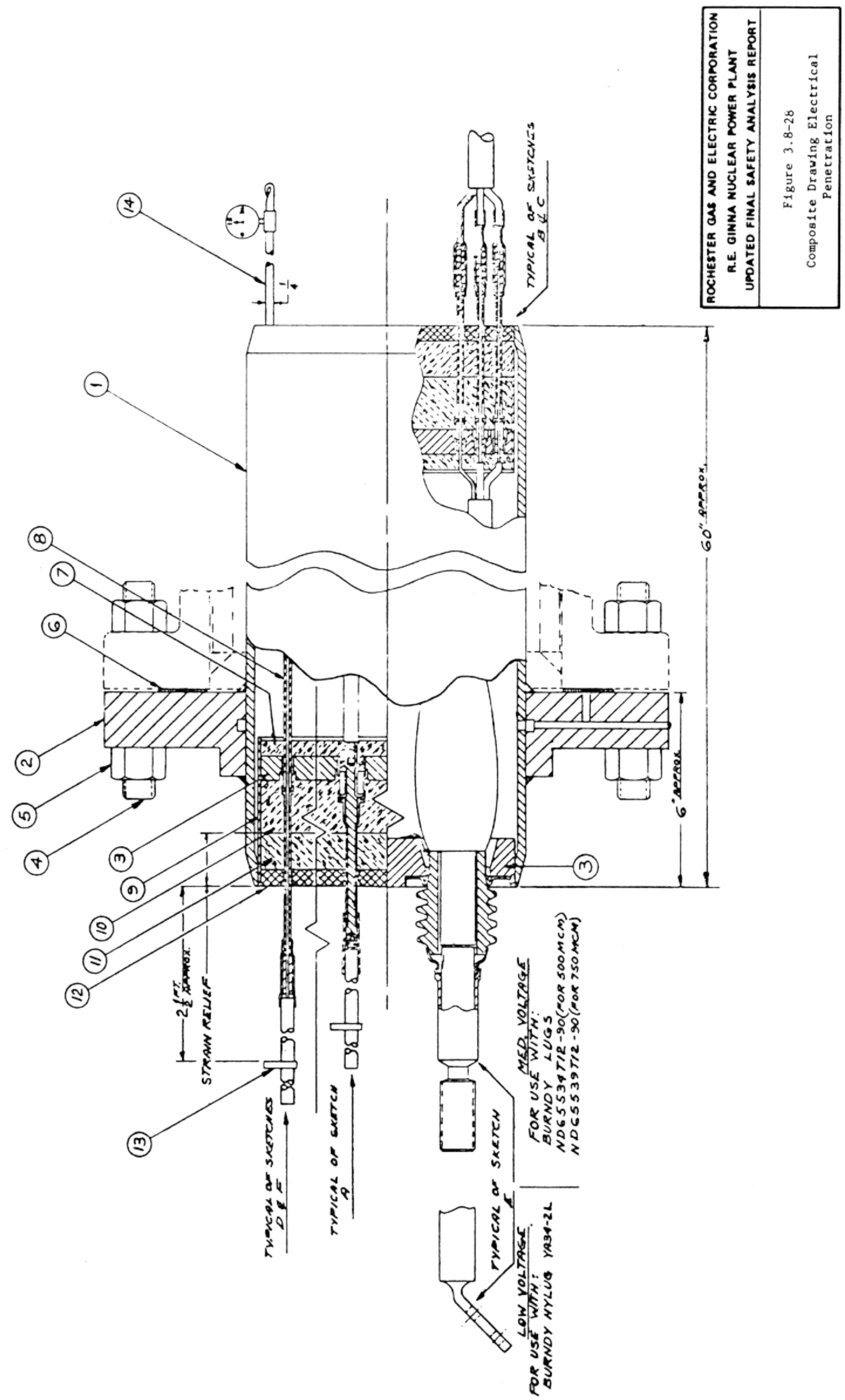

Figure 3.8-29 Containment Penetrations Section and Details

Figure Deleted

Refer to Drawing
D521-0057 Rev. 005

Figure 3.8-30 Containment Equipment Hatch

Figure Deleted

Refer to Drawing

Chicago Bridge and Iron
Drawing 170.95159

Figure 3.8-31 Containment Personnel Hatch

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Refer to Drawing

Chicago Bridge and Iron
Drawing 100.95159

Figure 3.8-32 Containment - Fuel Transfer Tube Penetration

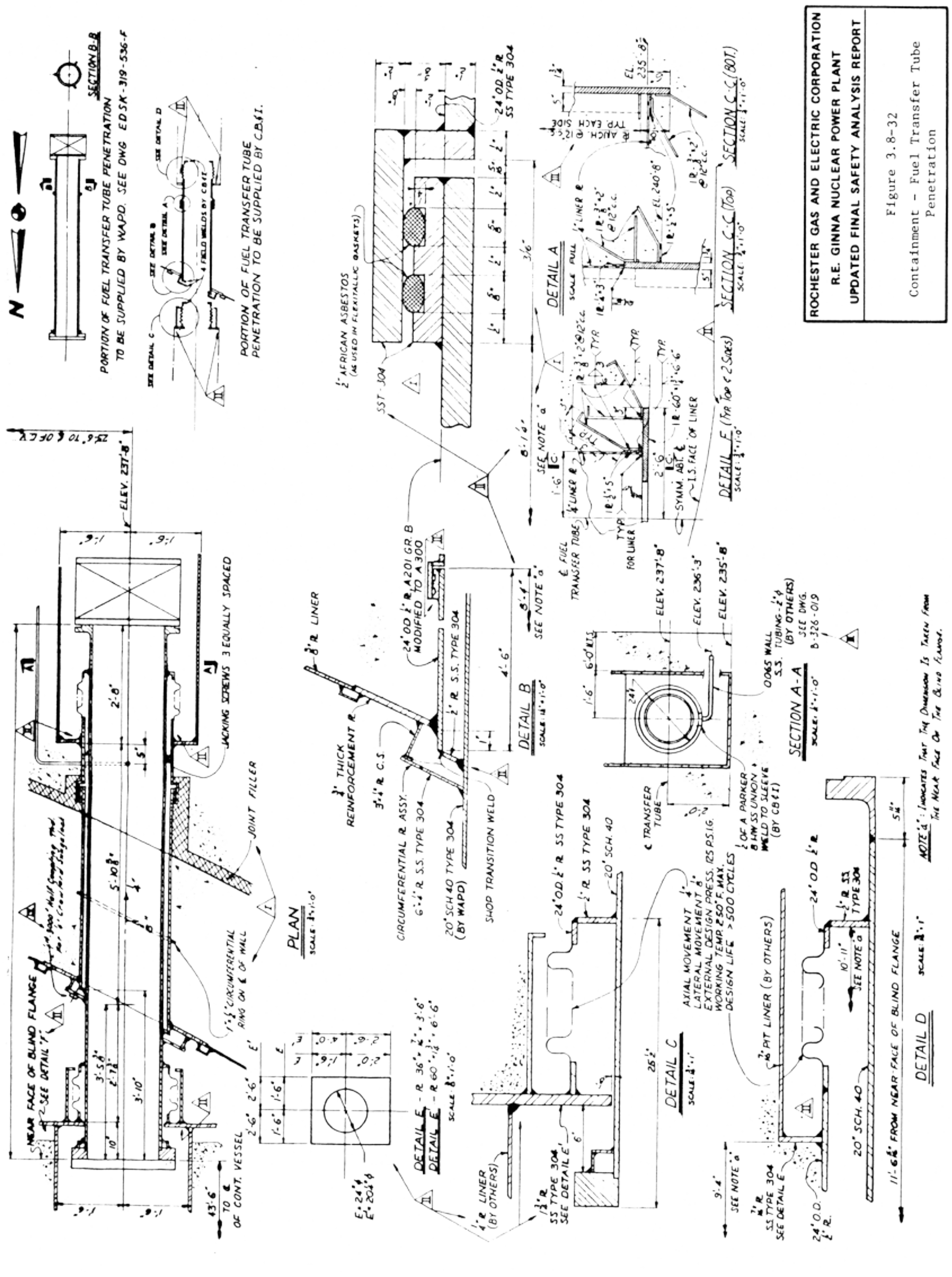

Figure 3.8-33 Containment Penetrations Arrangements and Location

Figure Deleted

Refer to Drawing
D521-0062 Rev. 002

Figure 3.8-34 Test Coupon - Containment Concrete Shell

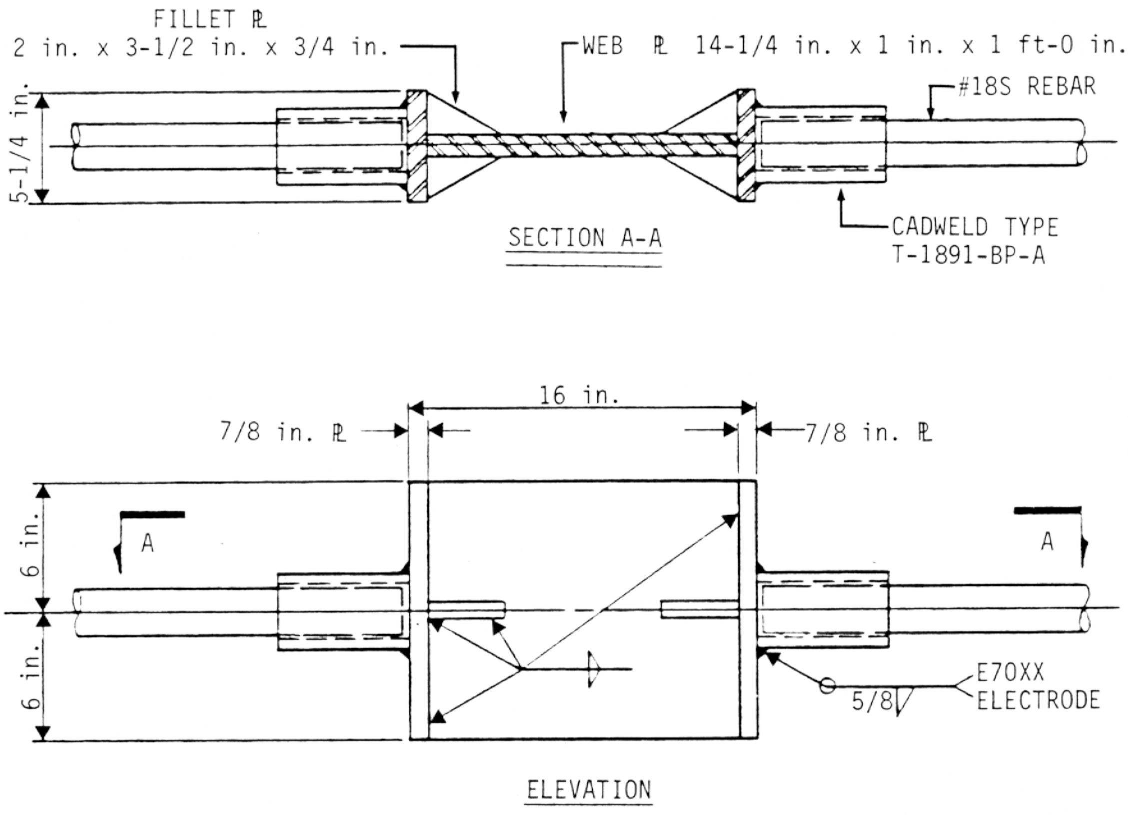

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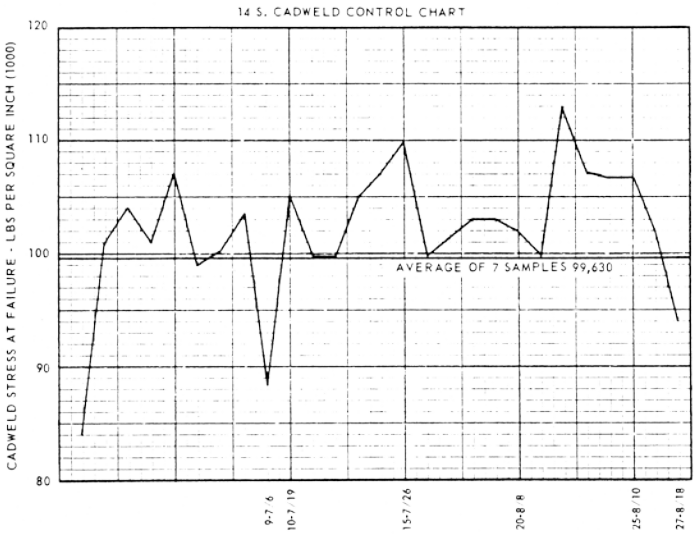
Figure 3.8-34

Test Coupon - Containment Concrete Shell

Figure 3.8-35 Cadweld Splice Test Results



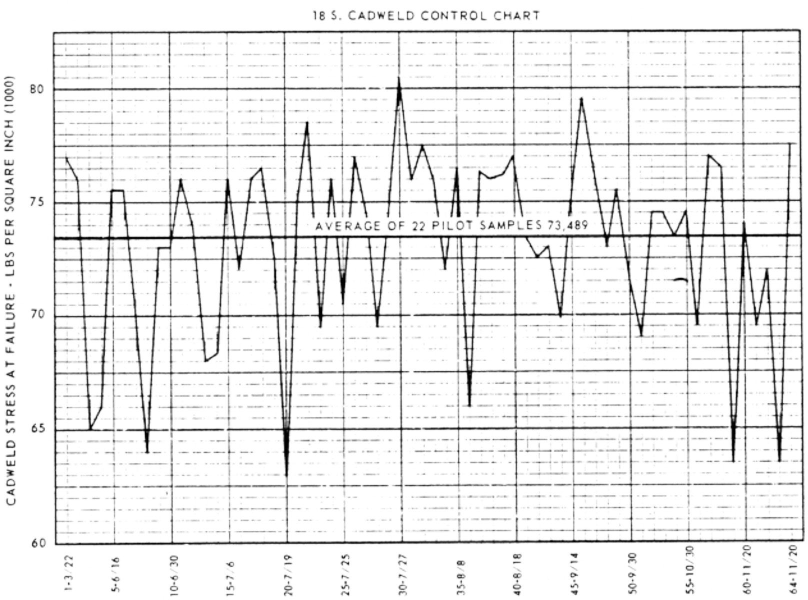


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Figure 3.8-35
Cadweld Splice Test Results

Figure 3.8-36 Quality Control Chart for 5000 PSI Concrete

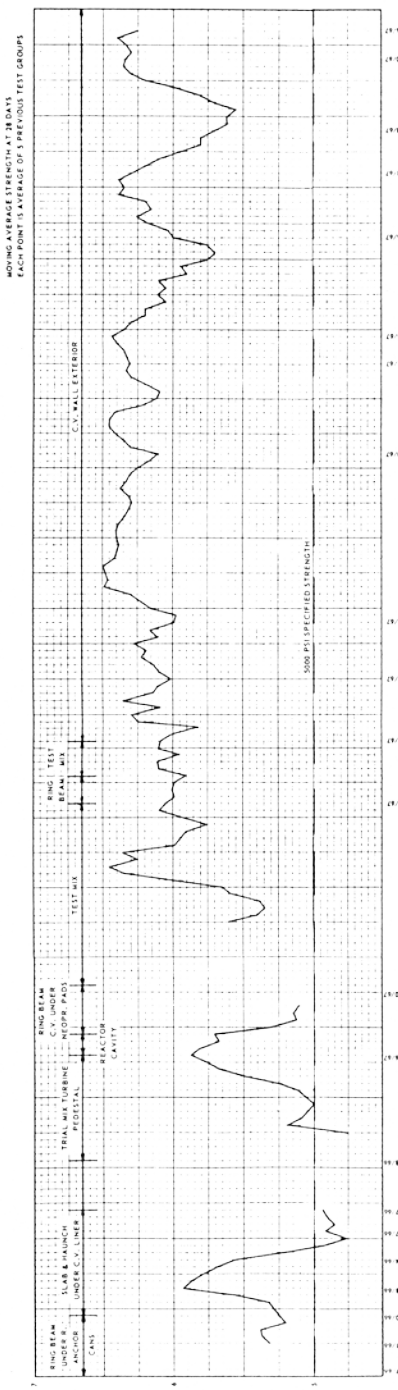

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Figure 3.8-36
Quality Control Chart for 5000 PSI Concrete

Figure 3.8-37 Neoprene Base Hinge Load Deformation Specimen 1

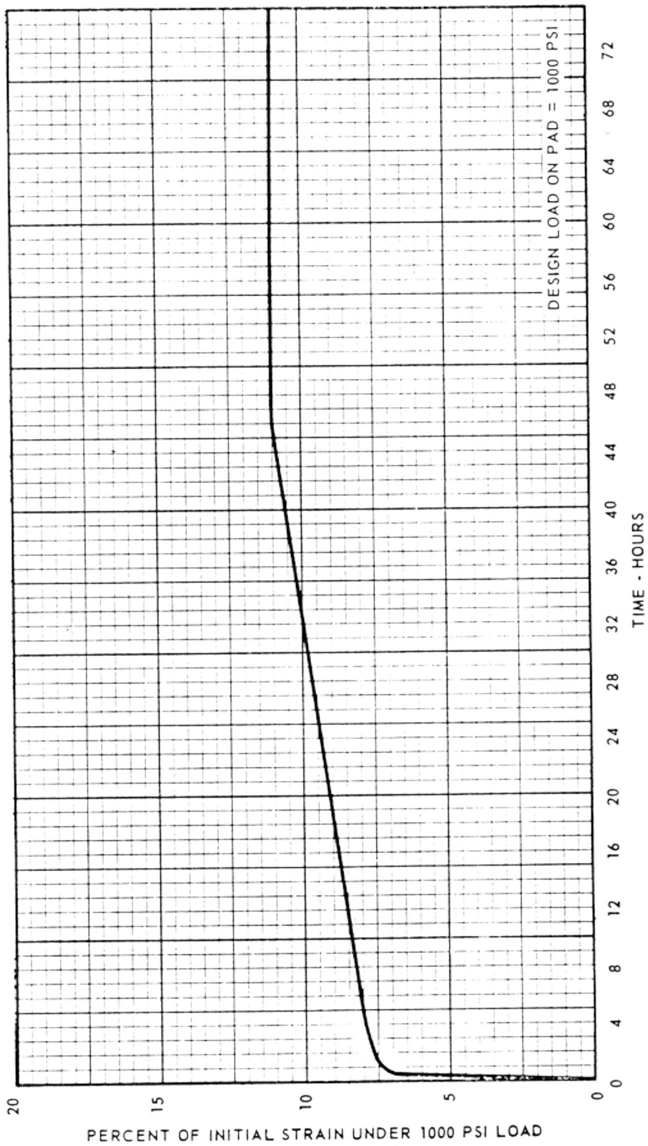

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Figure 3.8-37 Neoprene Base Hinge Load Deformation Specimen 1

Figure 3.8-38 Neoprene Base Hinge Load Deformation Specimen 2

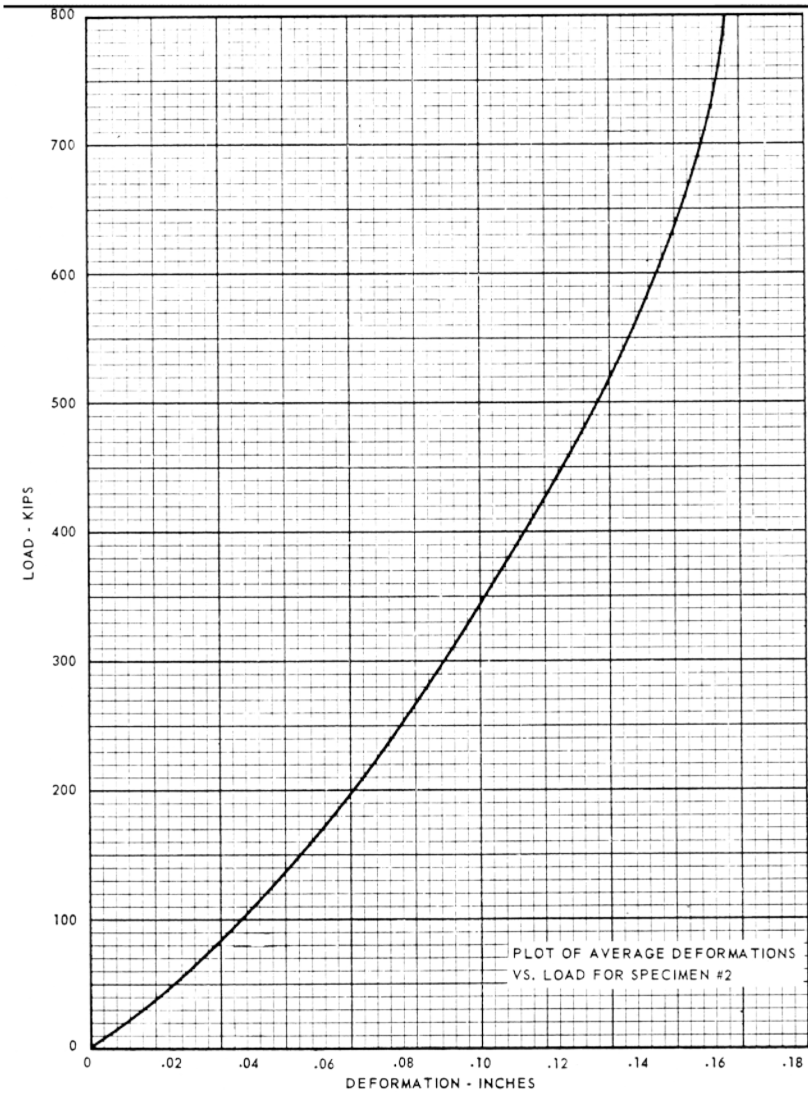

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Figure 3.8-38 Neoprene Base Hinge Load Deformation Specimen 2

Figure 3.8-39 Rock Anchor Test A-1

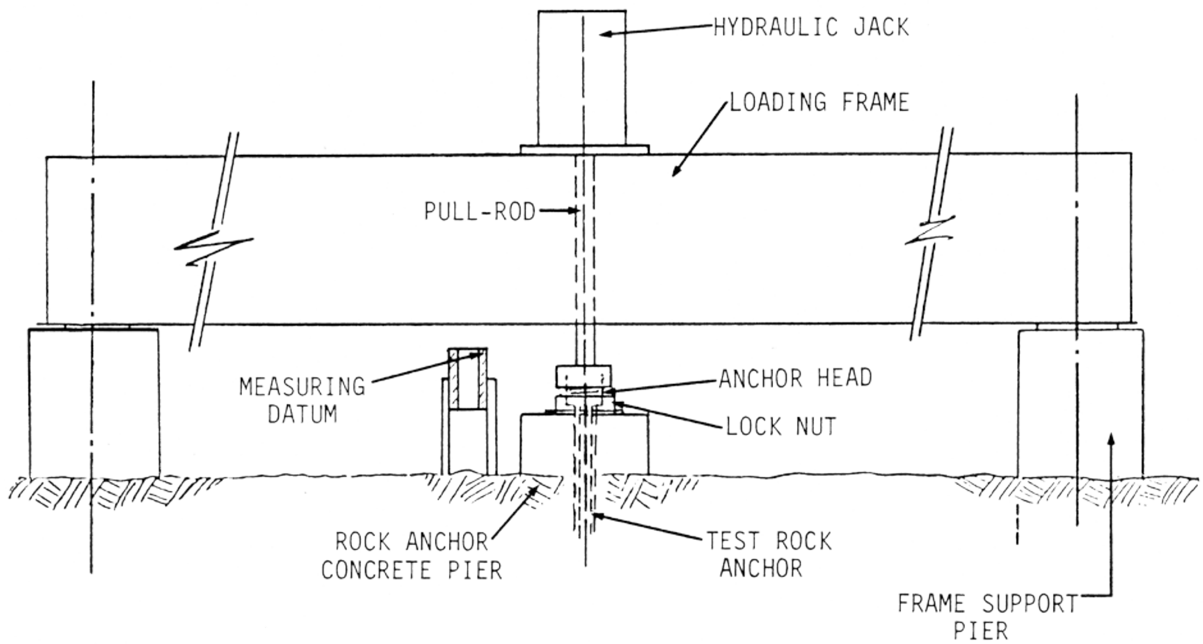

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Figure 3.8-39 Rock Anchor Test A-1

Figure 3.8-40   Containment - Rock Anchor A Test

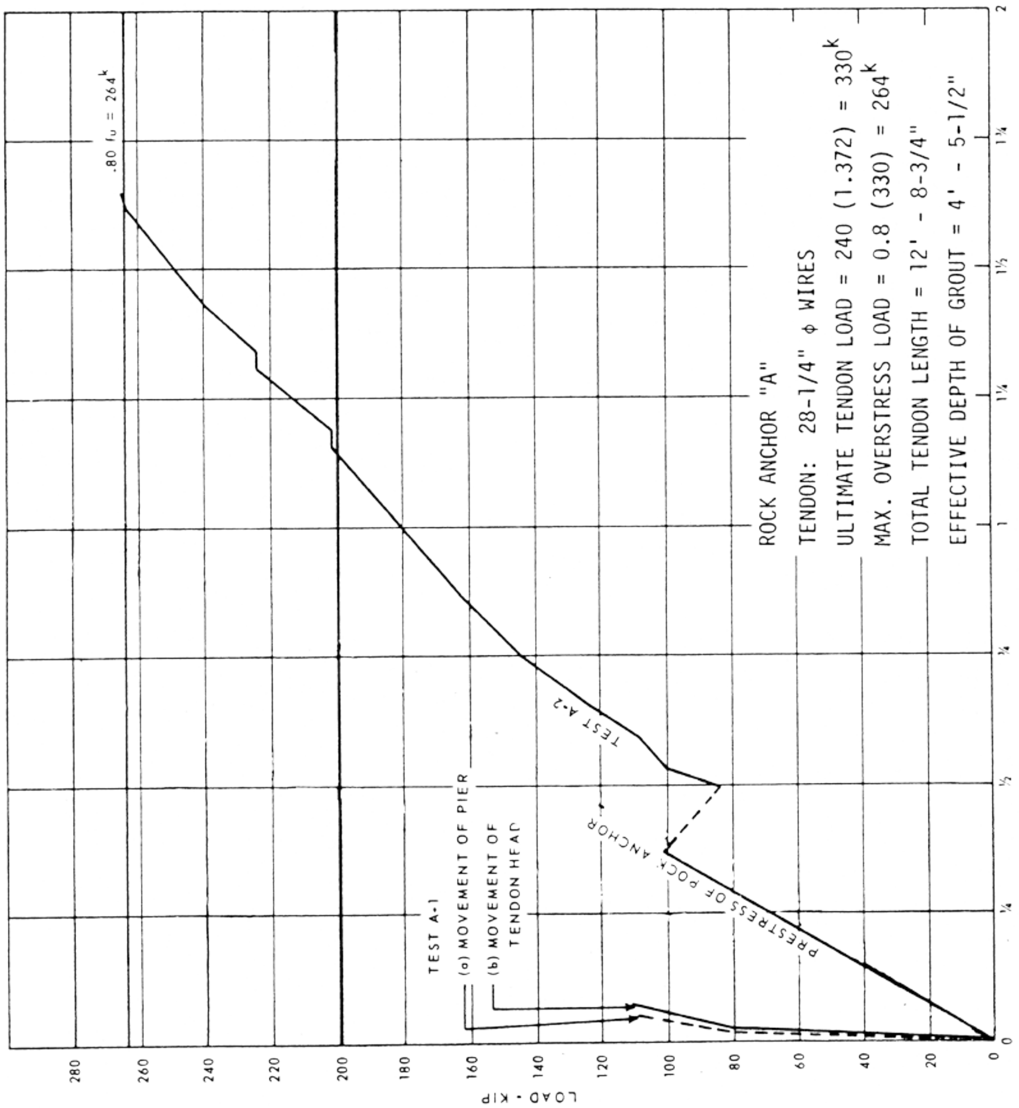

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Figure 3.8-40

Containment - Rock Anchor A Test

Figure 3.8-41 Containment - Rock Anchor B Test

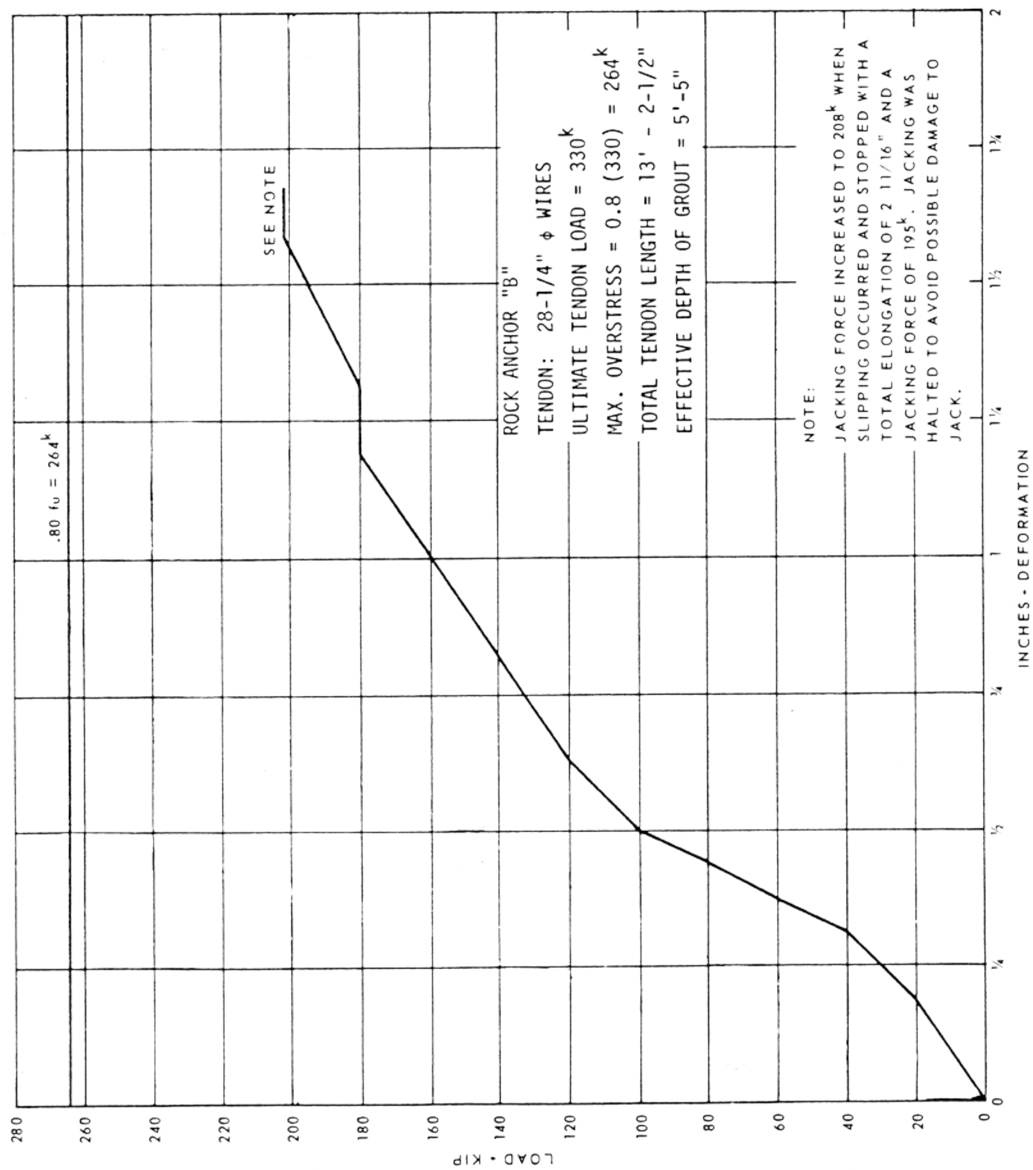

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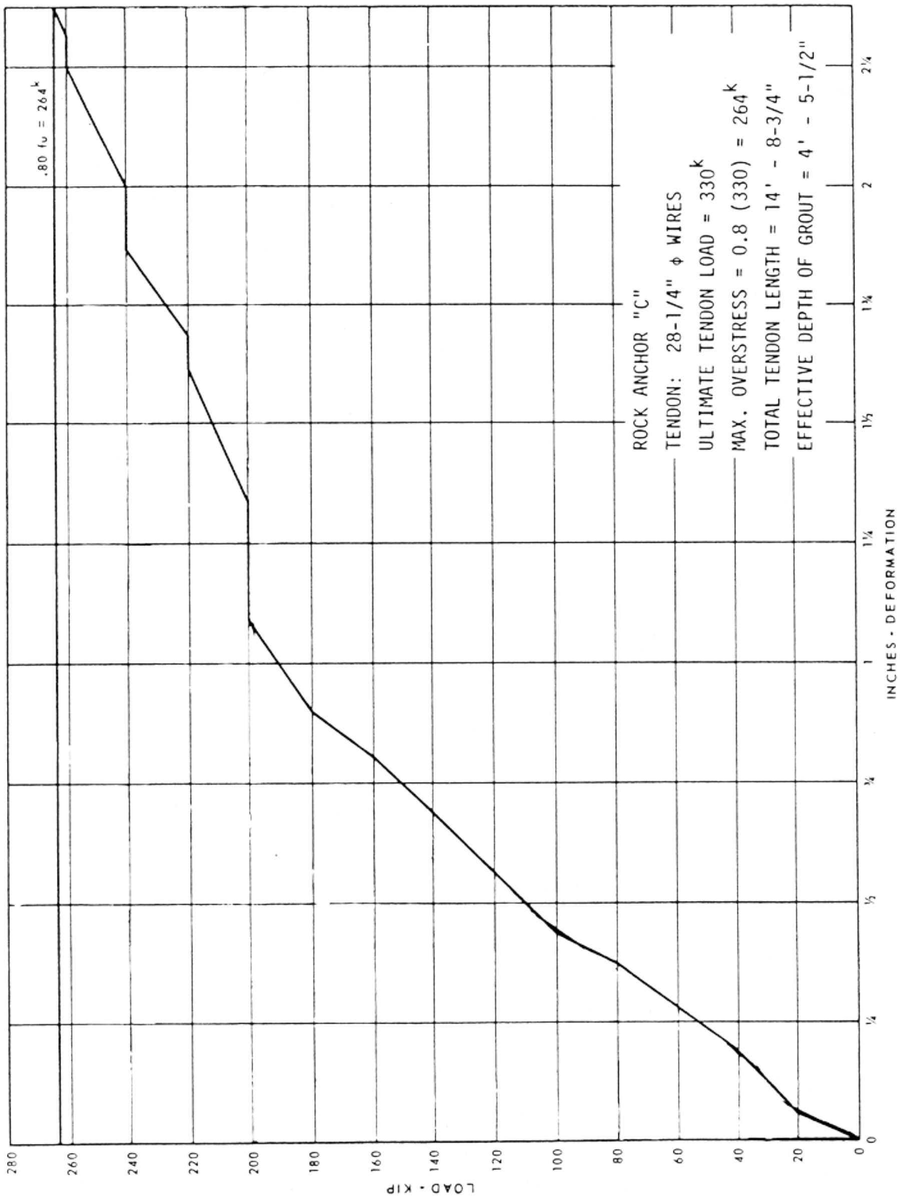
Figure 3.8-41
Containment - Rock Anchor B Test

Figure 3.8-42 Containment - Rock Anchor C Test



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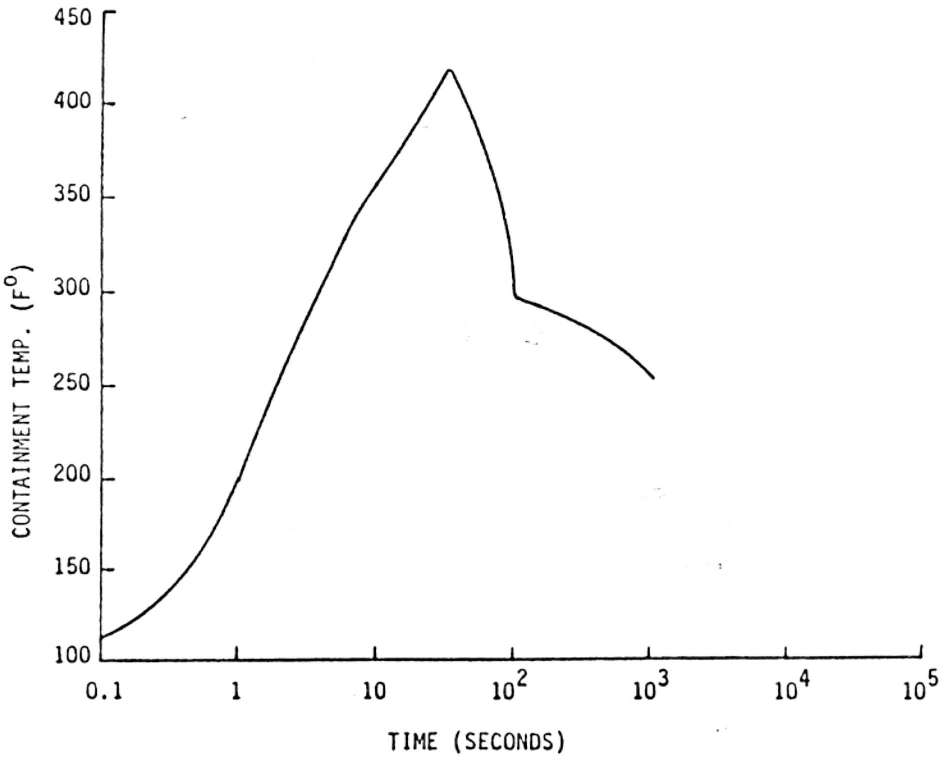
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-42
Containment - Rock Anchor C Test

Figure 3.8-43 Accident Temperature Transient Inside the Containment Used for Liner Analysis



ROCHESTER GAS AND ELECTRIC CORPORATION
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-43
Accident Temperature Transient Inside the Containment Used for Liner Analysis

Figure 3.8-44 Accident Pressure Transient Inside the Containment Used for Liner Analysis

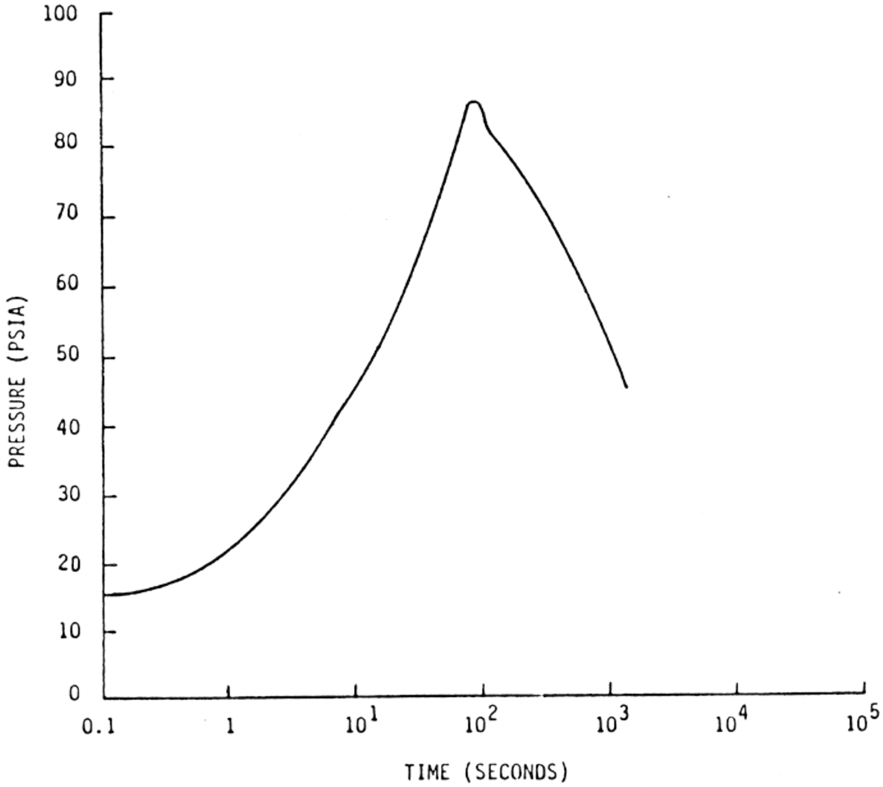


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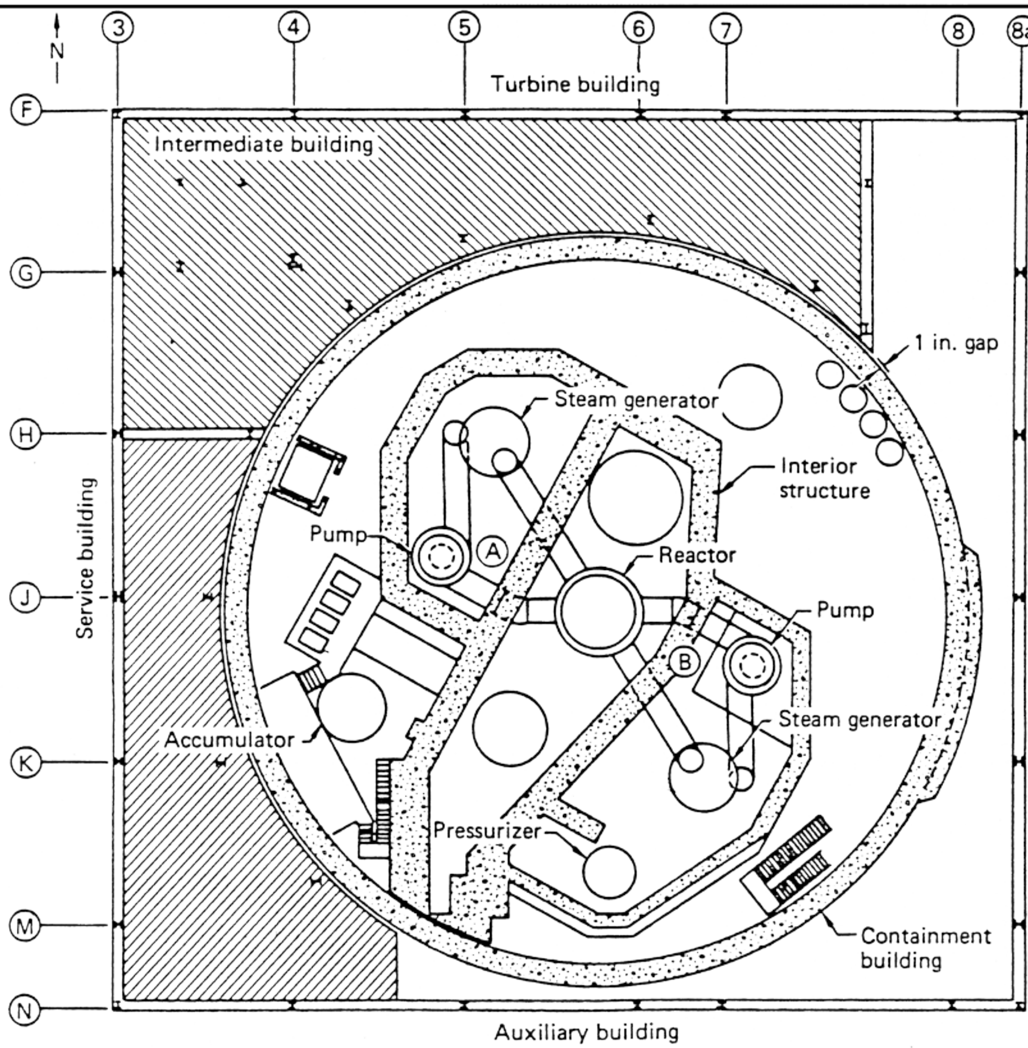
Figure 3.8-44 Accident Pressure Transient Inside the Containment Used for Liner Analysis

Figure 3.8-45 Plan View of the Facade Structure and Containment



NOTE:

DISCONTINUITY OF INTERMEDIATE BUILDING FLOORS AT COLUMN LINE H; 1-IN. GAP SURROUNDING THE CONTAINMENT BUILDING; ANOTHER GAP SEPARATING THE INTERNAL STRUCTURE FROM THE CONTAINMENT WALL; AND TWO CLOSED REACTOR COOLANT LOOPS (A AND B) CONNECTED IN PARALLEL TO THE REACTOR VESSEL.

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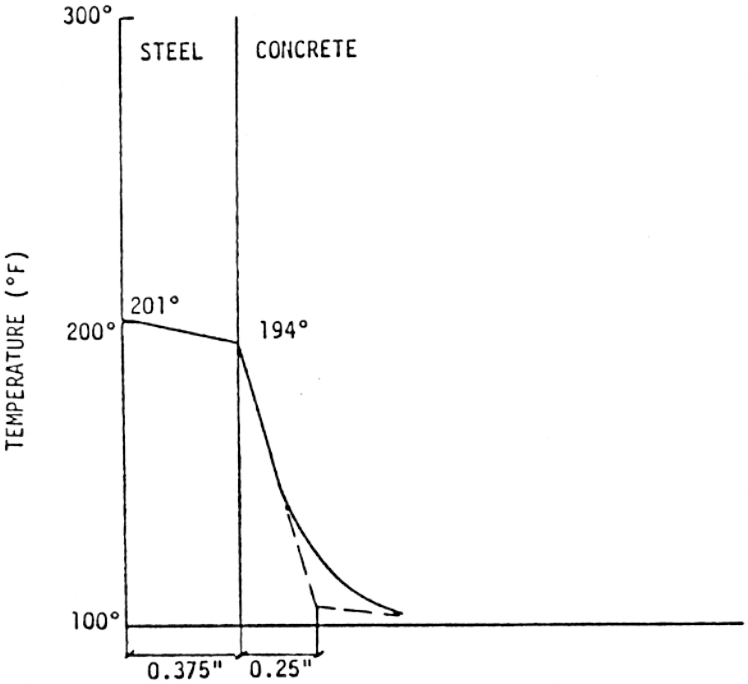
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-45
Plan View of the Facade Structure and Containment

Figure 3.8-46 Accident Temperature Gradient Through the Uninsulated Containment Shell After 94 Seconds



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UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-46

Accident Temperature Gradient Through the Uninsulated Containment Shell After 94 Seconds

Figure 3.8-47 Accident Temperature Gradient Through the Uninsulated Containment Shell After 380 Seconds

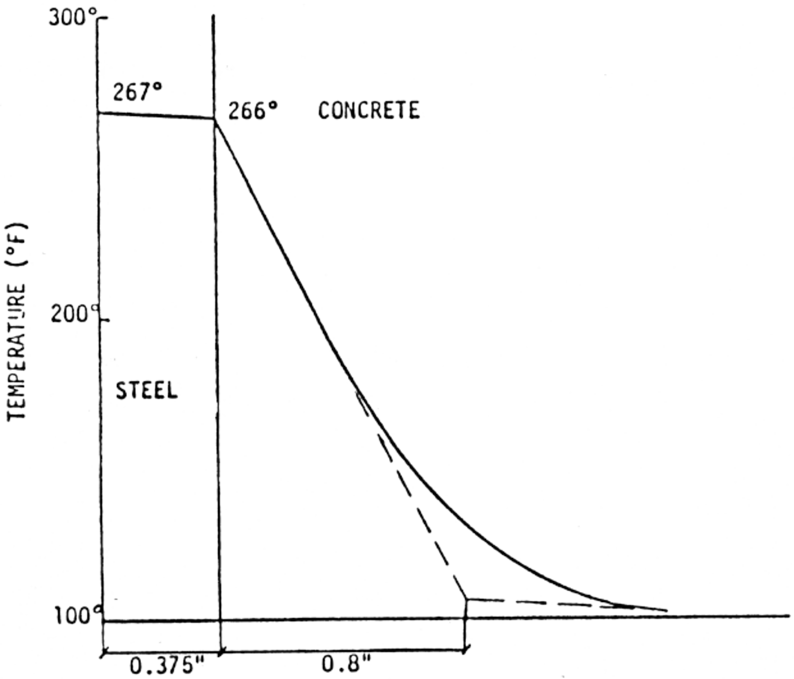

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Figure 3.8-47

Accident Temperature Gradient Through the Uninsulated Containment Shell After 380 Seconds

Figure 3.8-48 Ginna Containment Structure

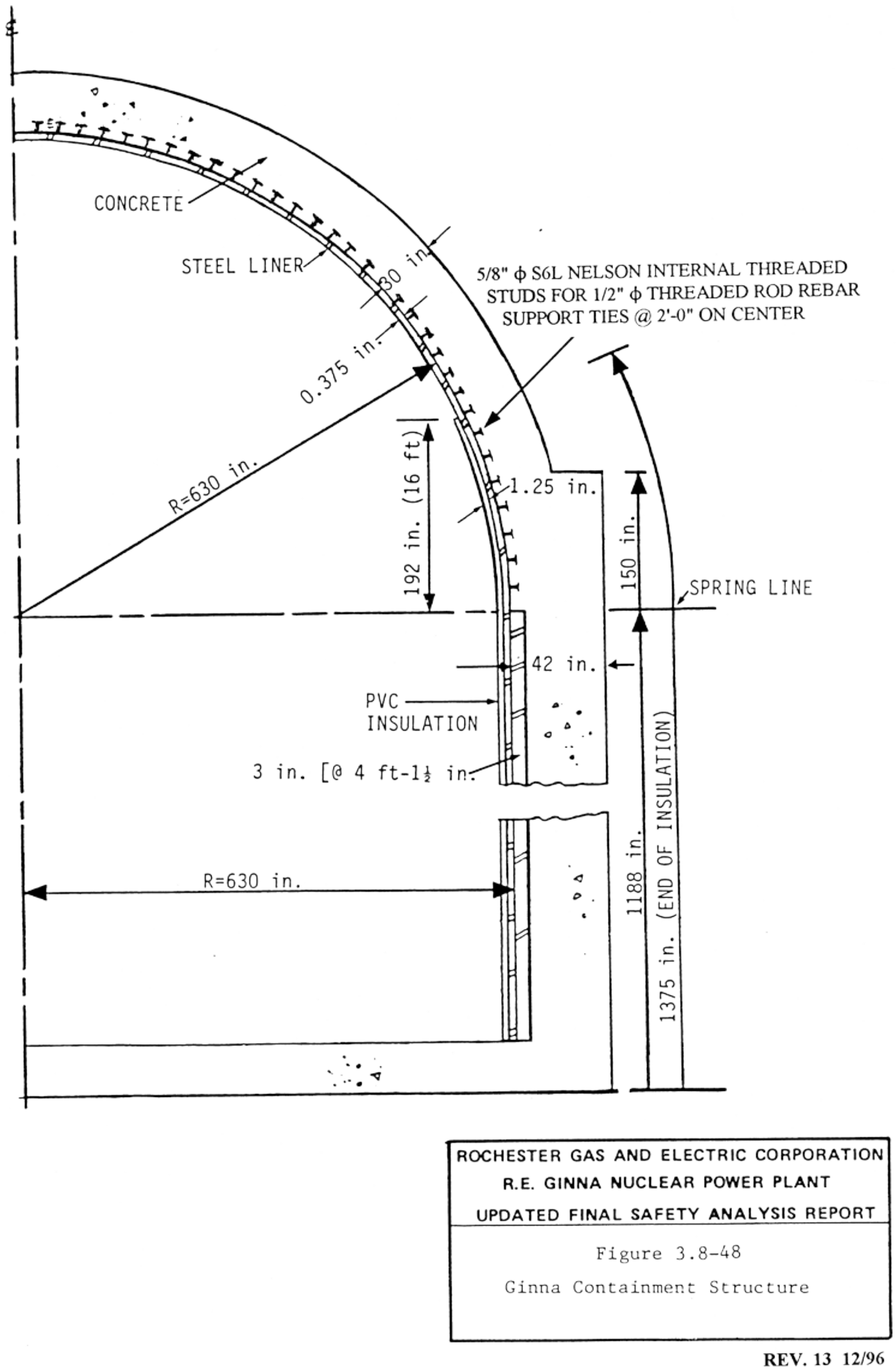

Figure 3.8-49 Liner Stud Interaction Models

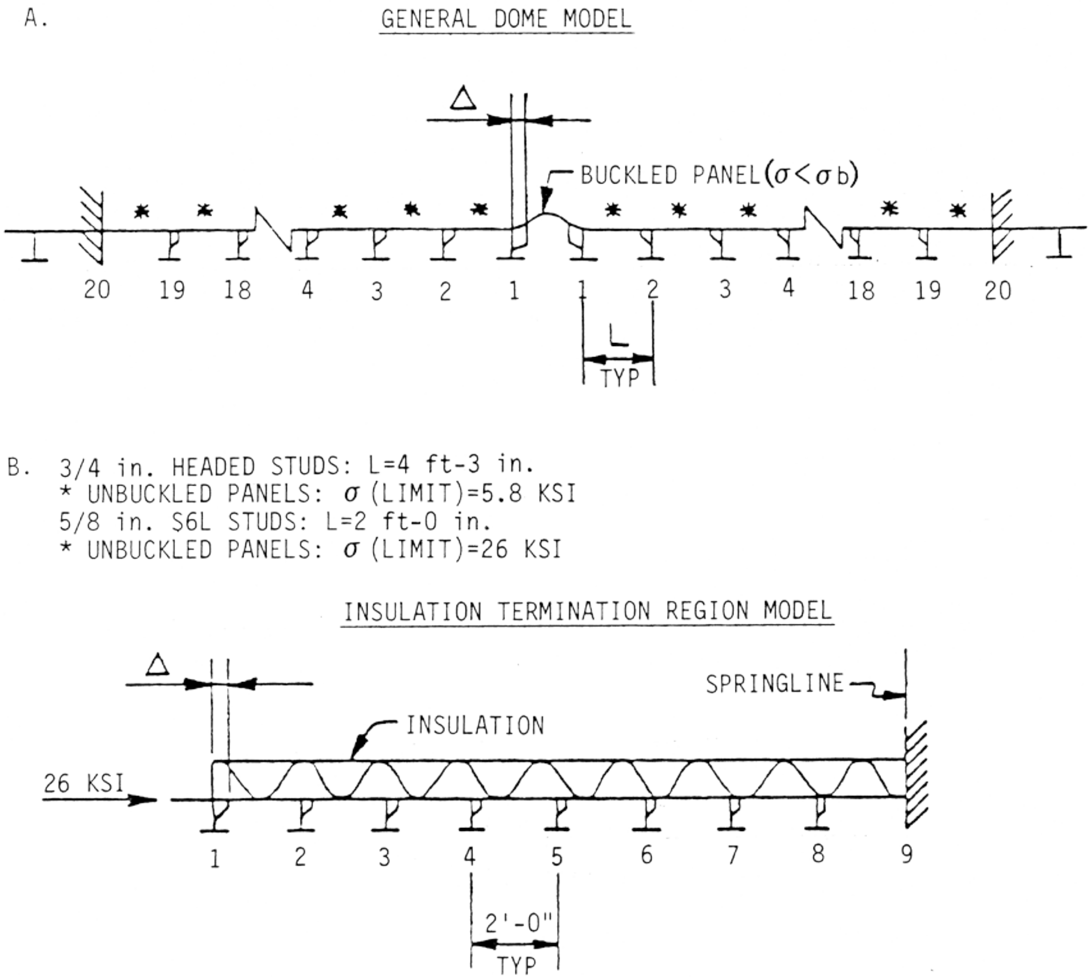

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Figure 3.8-49
Liner Stud Interaction Models

Figure 3.8-50 Accident Temperature Distribution in the Steel Liner

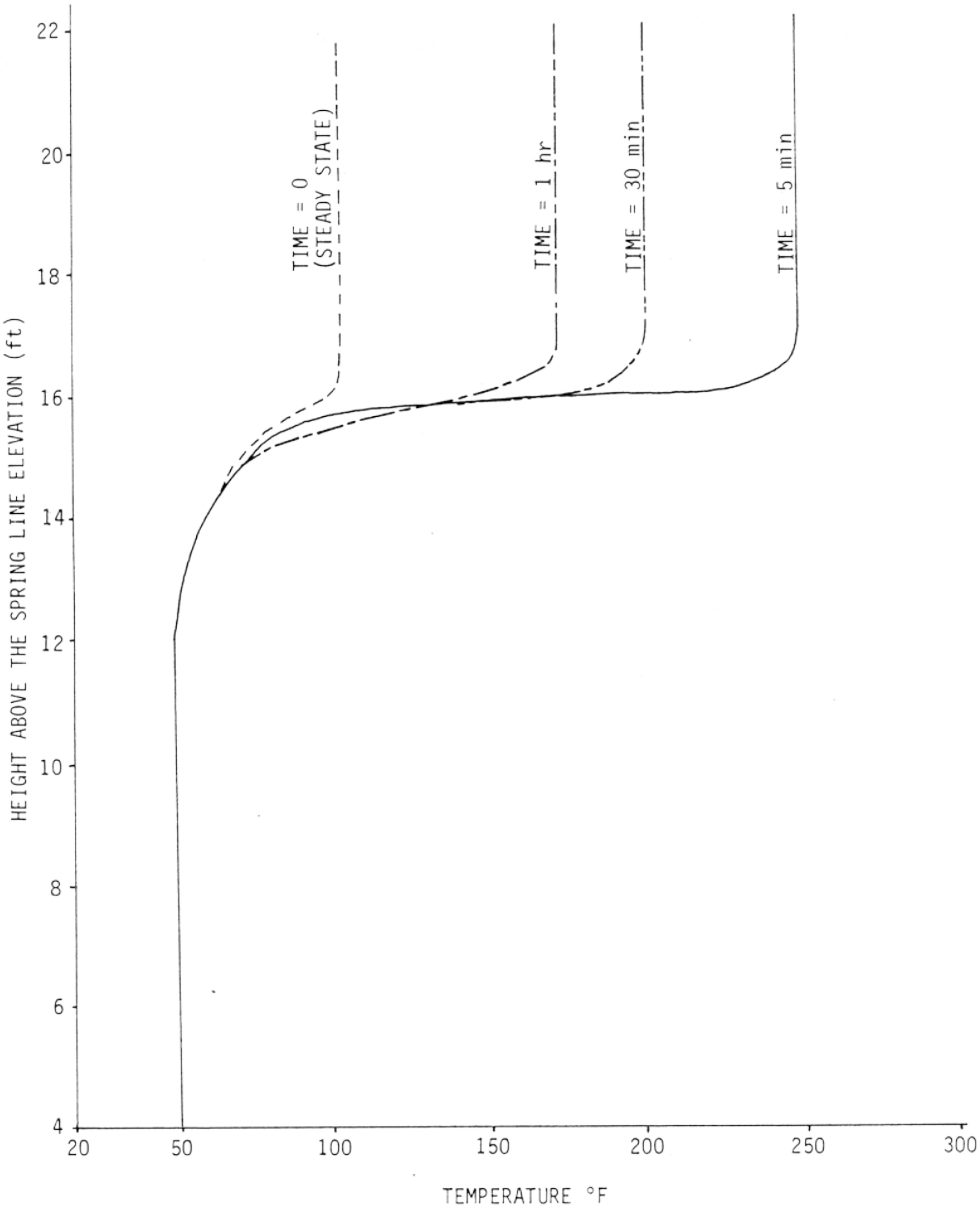

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Figure 3.8-50 Accident Temperature Distribution in the Steel Liner

Figure 3.8-51 Force Displacement Curve for 3/4 in. Headed Studs

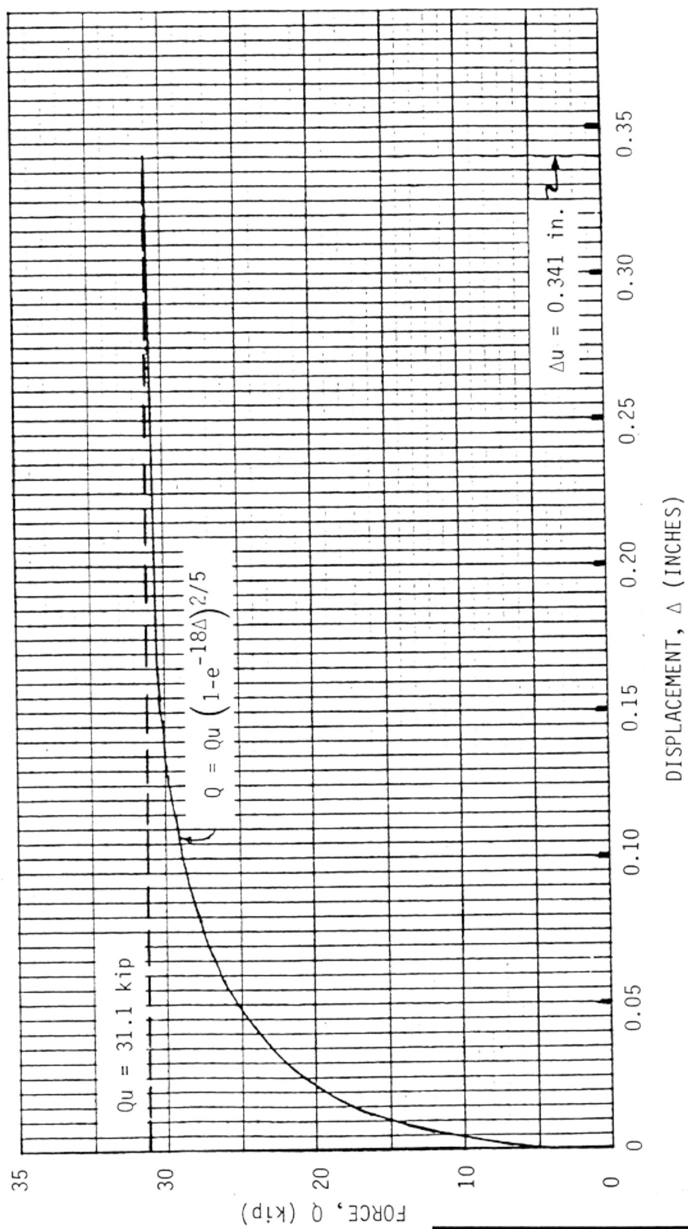

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Figure 3.8-51

Force Displacement Curve for 3/4 in. Headed Studs

Figure 3.8-52   Force Displacement Curve for 5/8 in. S6L Studs

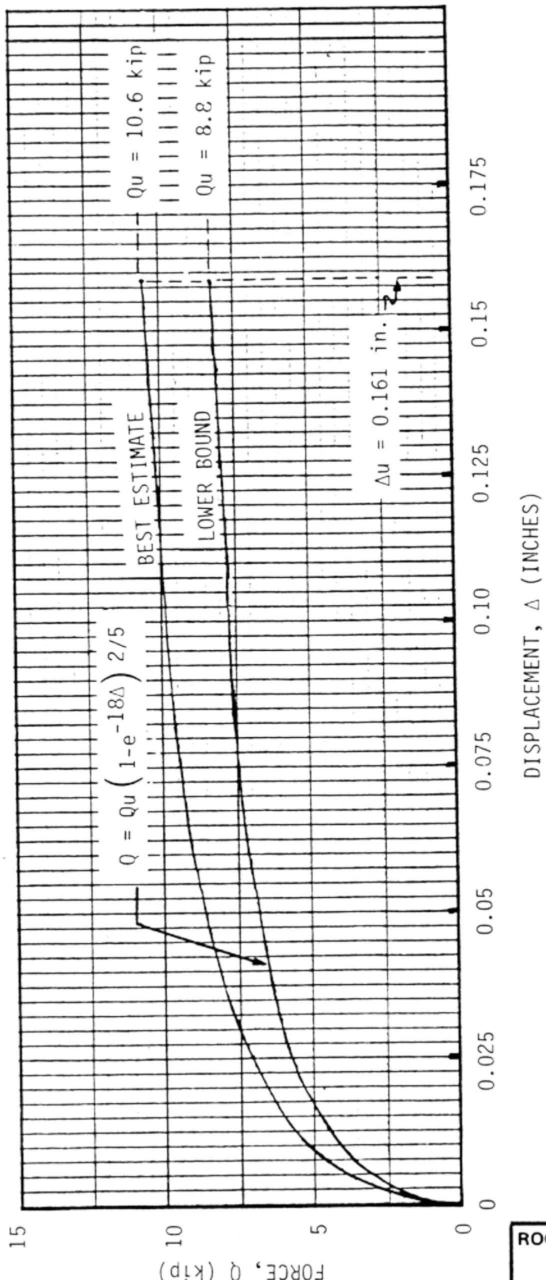

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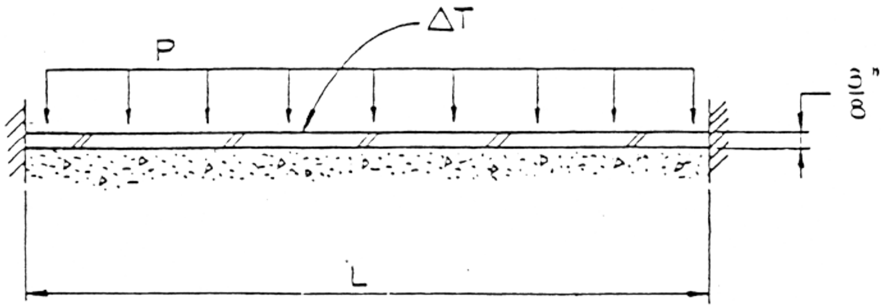
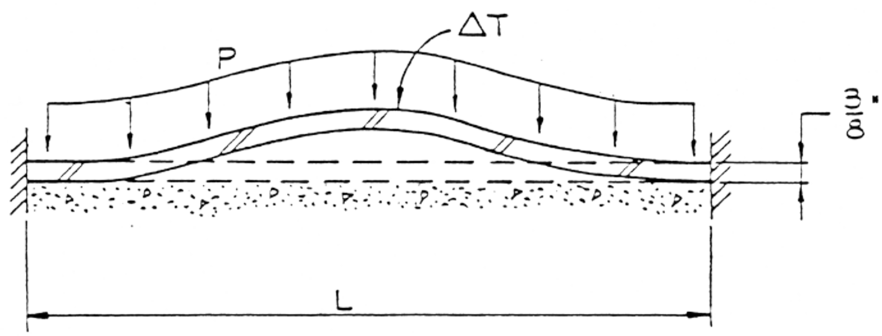
Figure 3.8-52
Force Displacement Curve for 5/8 in. S6L Studs

Figure 3.8-53   Strut Buckling Under P and Delta T



A. BEFORE BUCKLING



B. AFTER BUCKLING

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Figure 3.8-53
Strut Buckling Under P and Delta T

Figure 3.8-54   Pressure Effect on Liner Buckling Comparison With LOCA

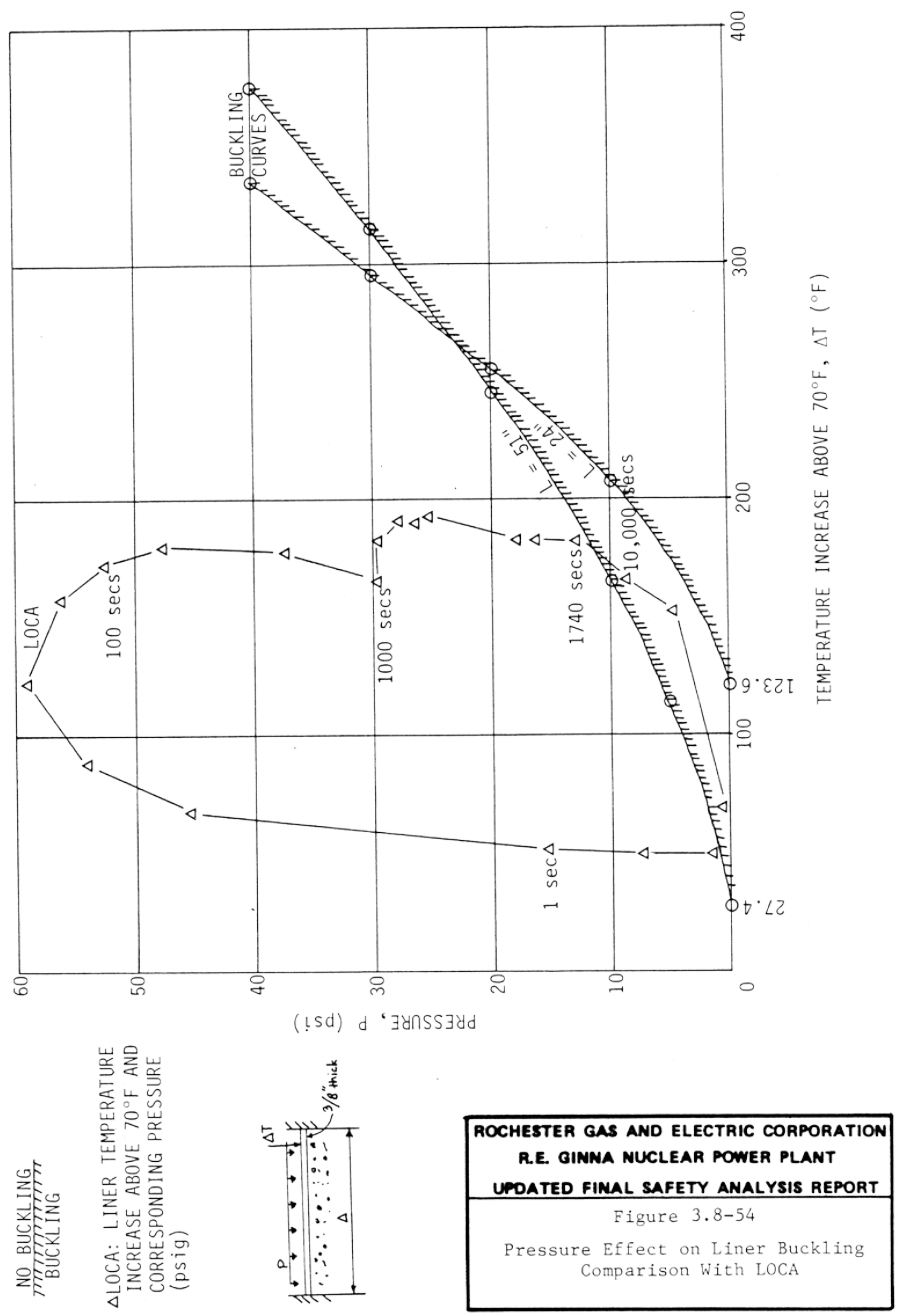

Figure 3.8-55 Reactor Containment Internal Structures

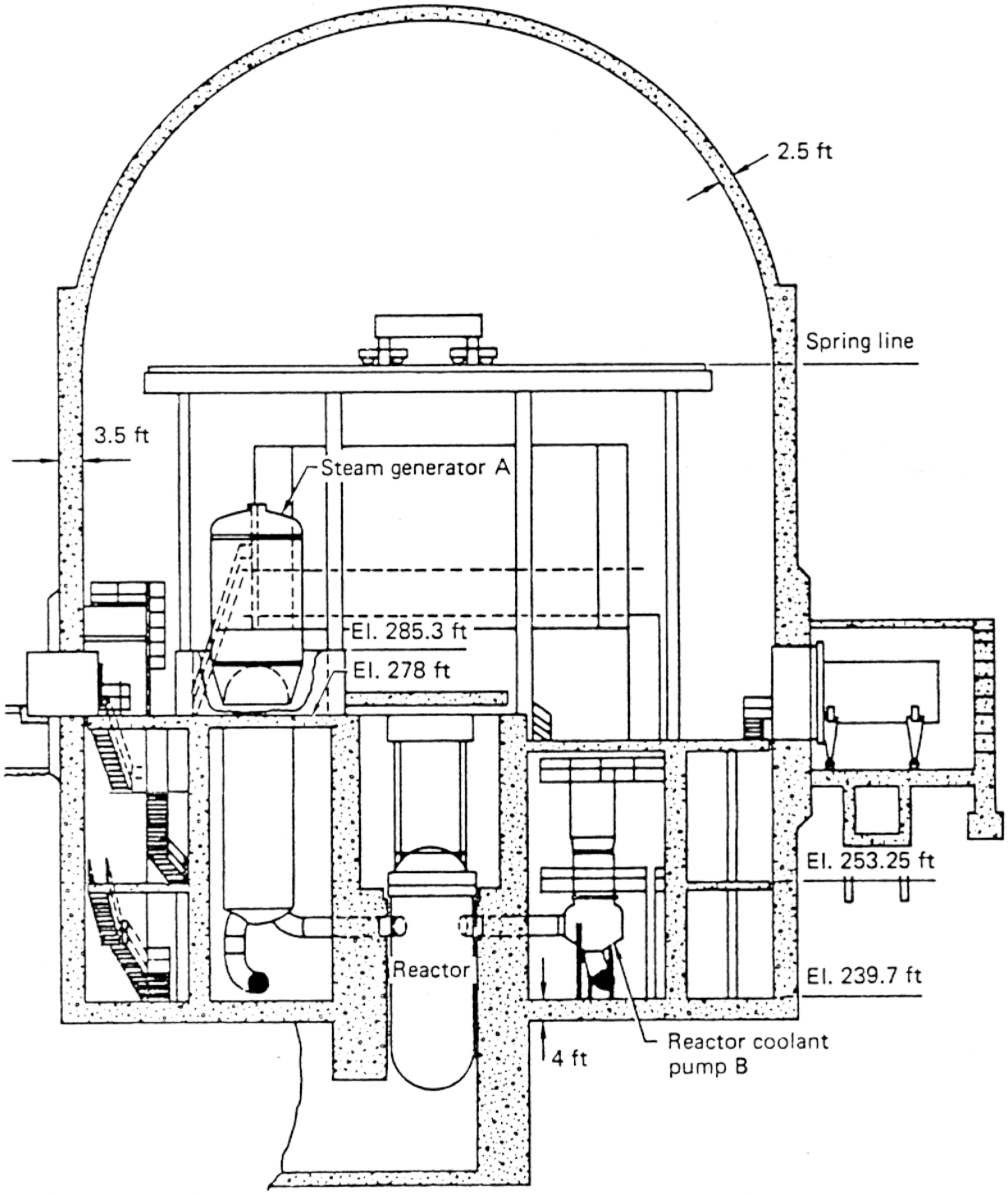

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Figure 3.8-55 Reactor Containment Internal Structures

Figure 3.8-56 Containment Interior Structures Model for STARDYNE

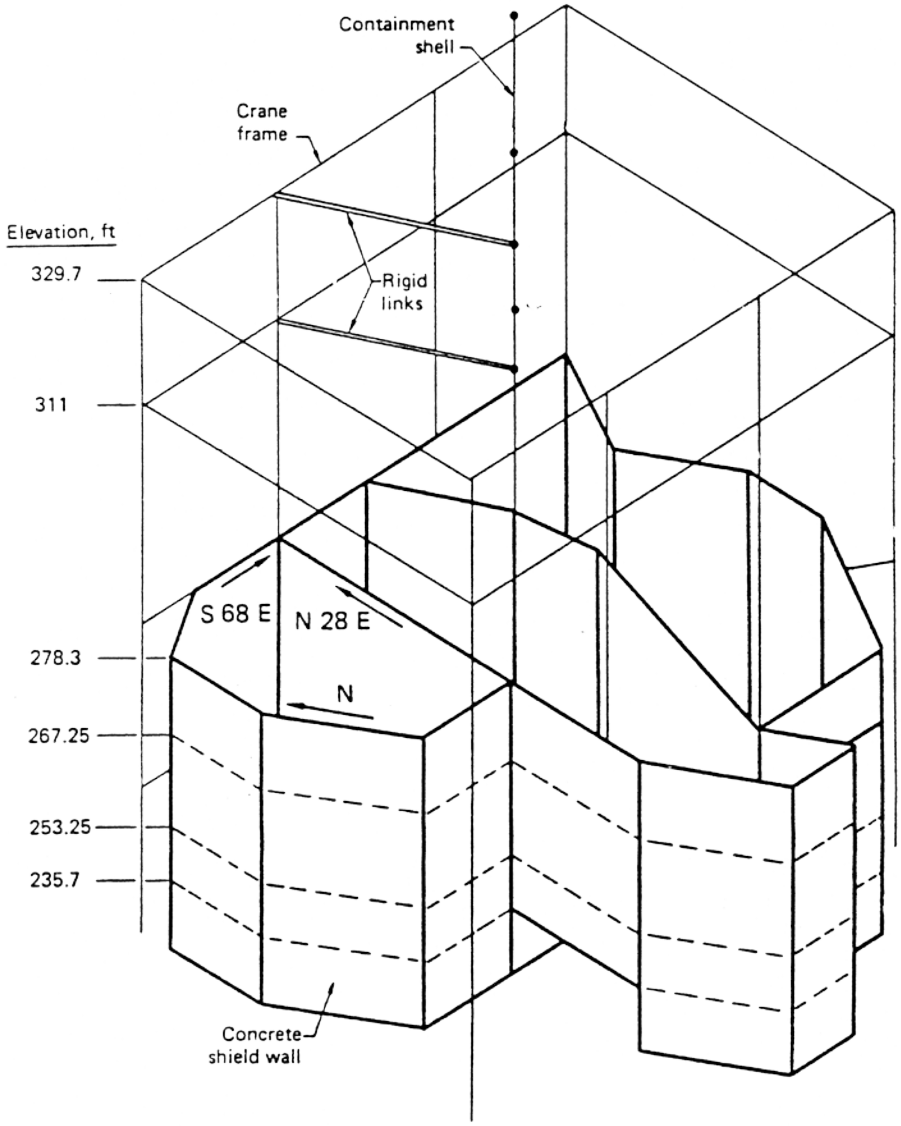

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Figure 3.8-56

Containment Interior Structures Model for STARDYNE

Figure 3.8-57 Schematic Plan View of Major Ginna Structures

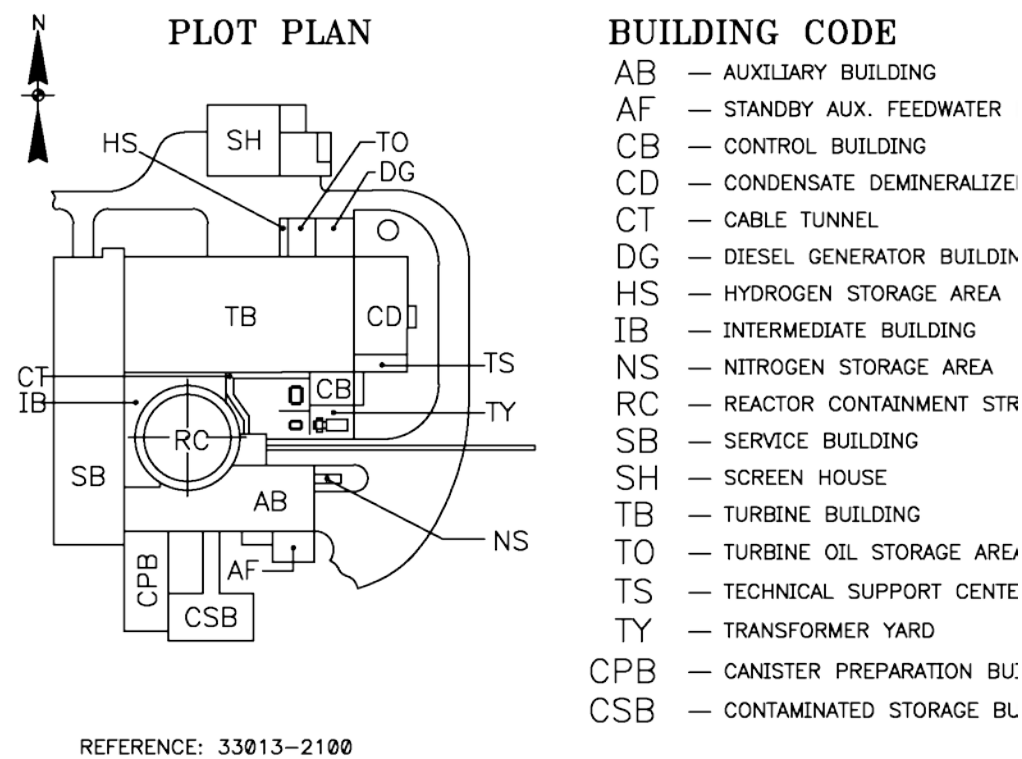

Figure 3.8-58 Three-Dimensional View of Interconnected Building Complex

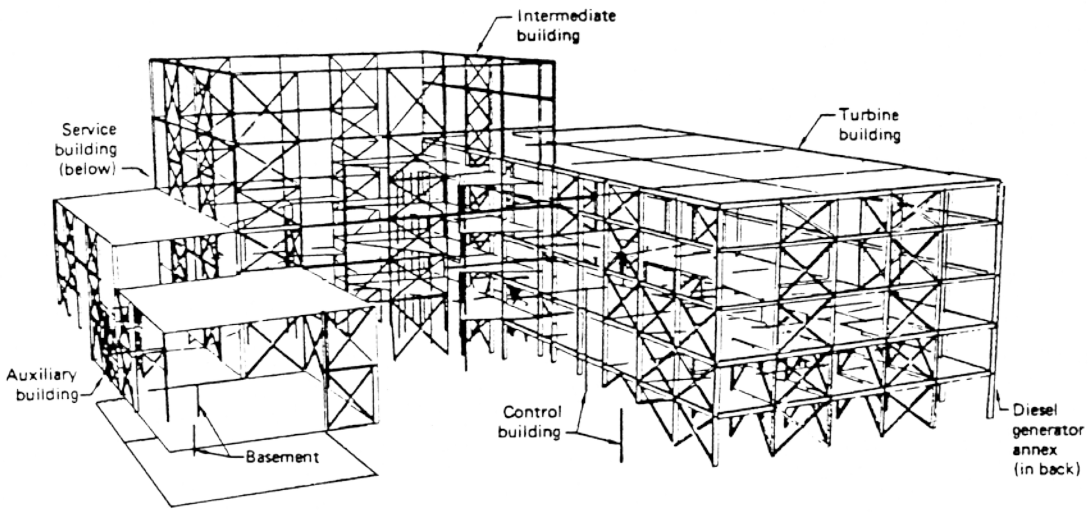

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Figure 3.8-58

Three-Dimensional View of Interconnected Building Complex

Figure 3.8-59 Flow Chart of the Analysis of the Interconnected Building Complex

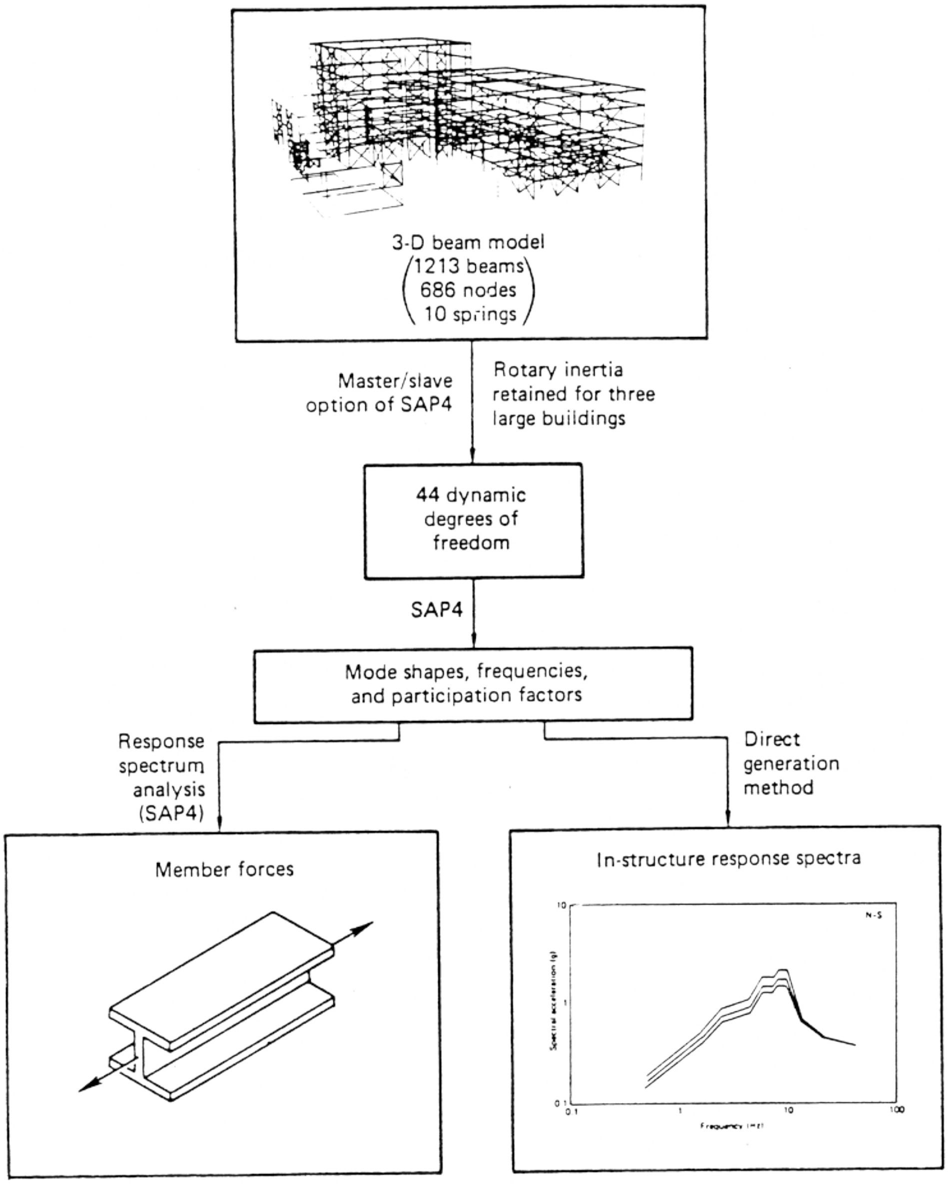

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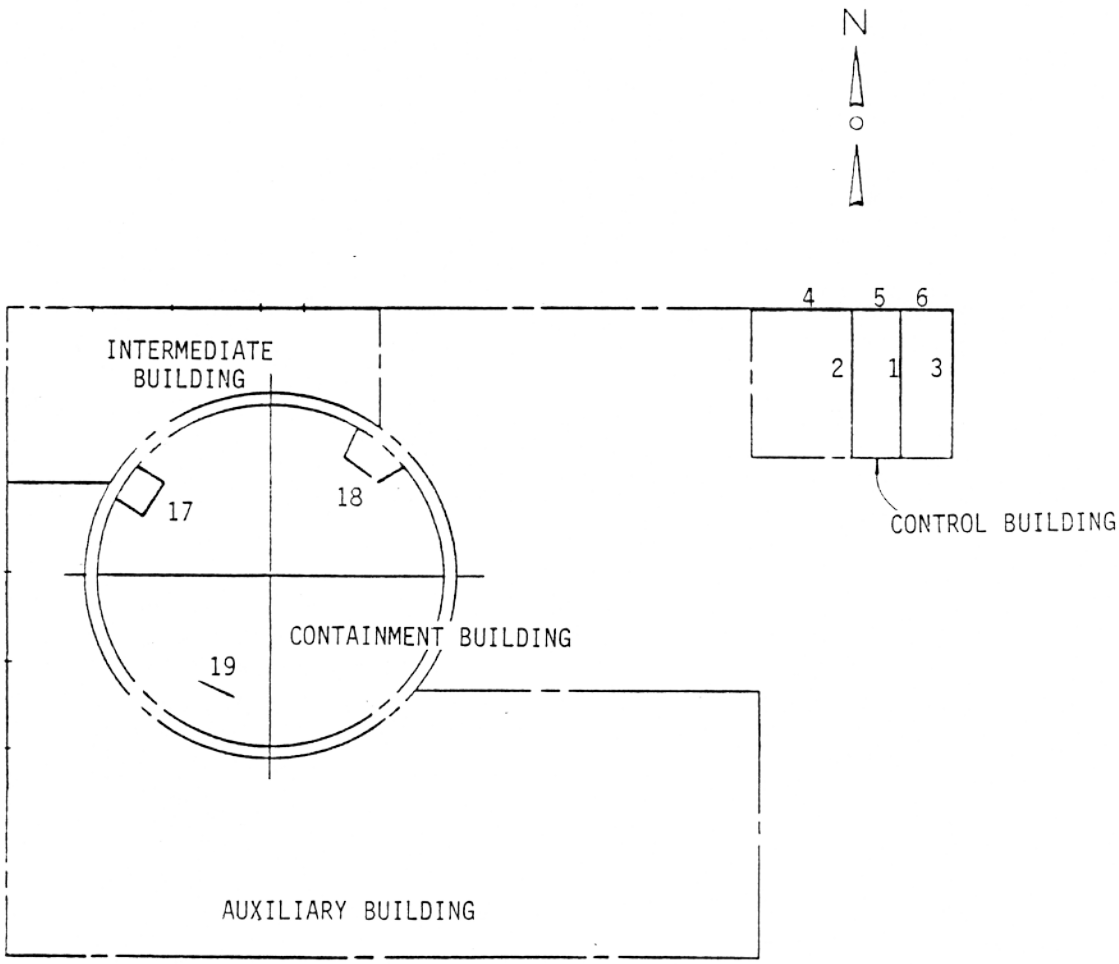
Figure 3.8-59

Flow Chart of the Analysis of the Interconnected Building Complex

Figure 3.8-60 Masonry Wall Reevaluation, Wall Location Plan, Lower Levels



WALL NO.	WALL ID NO.
1	971-1C
2	971-2C
3	971-3C
4	971-4C
5	971-5C
6	971-6C
17	971-1M
18	971-2M
19	971-3M

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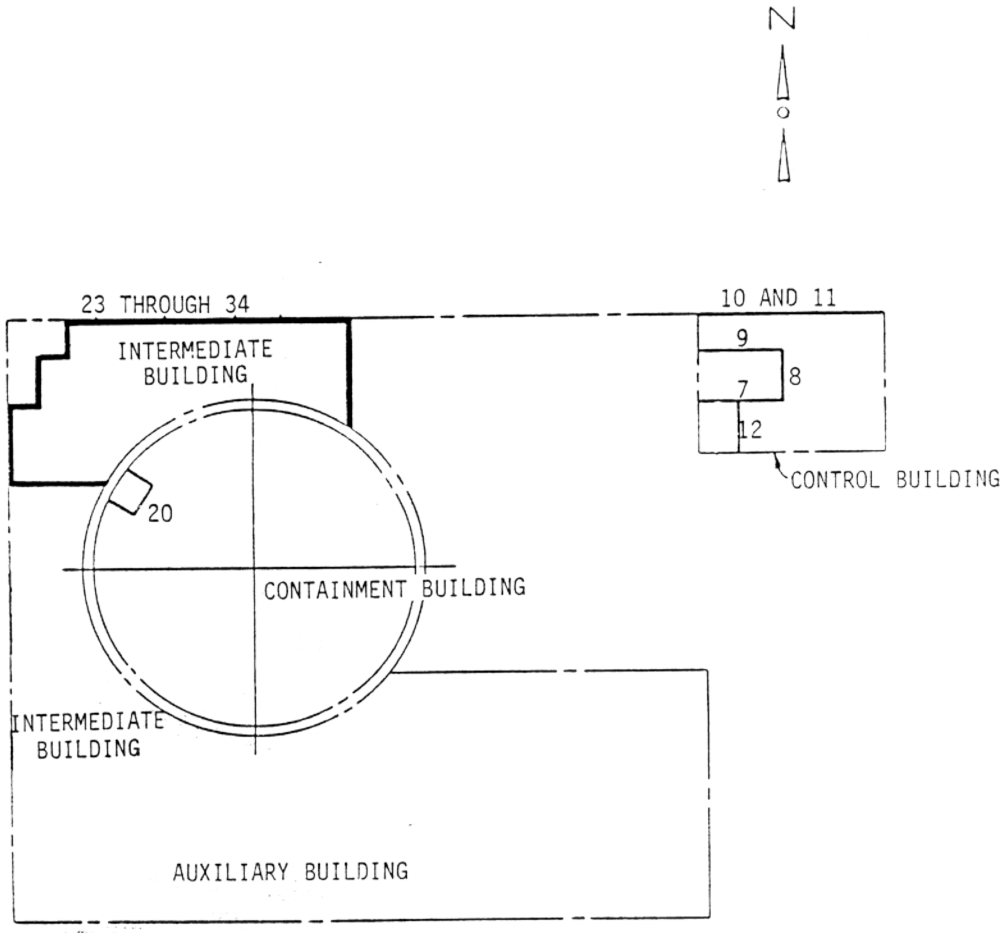
R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT

Figure 3.8-60

Masonry Wall Reevaluation, Wall Location Plan, Lower Levels

REV 2 12/86

Figure 3.8-61 Masonry Wall Reevaluation, Wall Location Plan, Intermediate Levels



WALL NO.	WALL ID NO.
7	972-1C
8	972-2C
9	972-3C
10	972-4C
11	972-5C
12	972-6C
20	972-1M
23-34	972-1I THROUGH 972-12I

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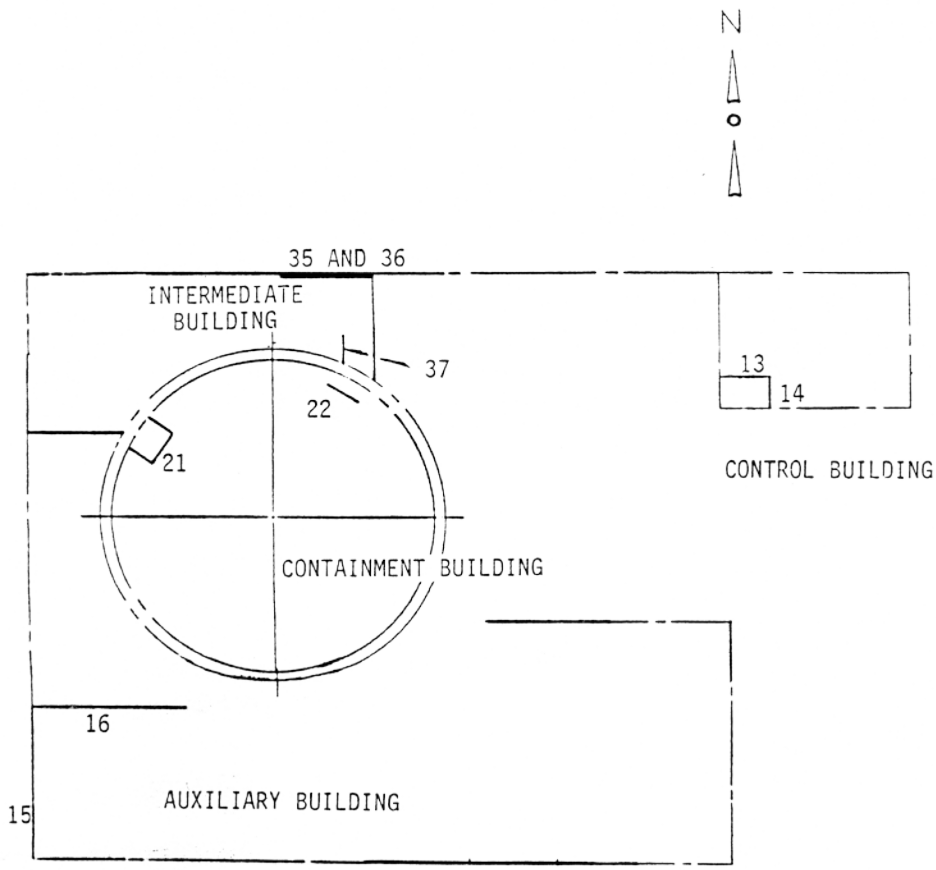
Figure 3.8-61

Masonry Wall Reevaluation, Wall Location Plan, Intermediate Levels

REV 2 12/86

Figure 3.8-62 Masonry Wall Reevaluation, Wall Location Plan, Operating Levels



WALL NO.	WALL ID NO.
13	973-3C
14	973-4C
15	973-16A
16	973-17A
21	973-1M
22	973-2M
35	973-1I
36	973-11I(P)
37	973-9I(P)

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Figure 3.8-62

Masonry Wall Reevaluation, Wall Location Plan, Operating Levels

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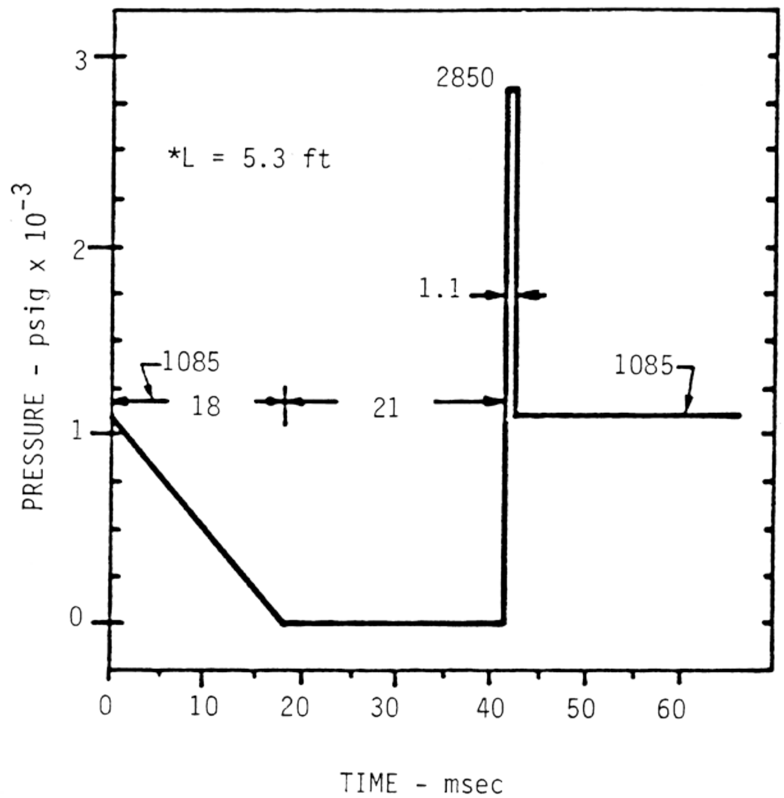
Figure 3.9-1 Steam-Generator Water Hammer Preliminary Forcing Function



(*L = LENGTH OF TEE + HORIZONTAL FEED LINE RUN)

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Figure 3.9-1 Steam-Generator Water Hammer Preliminary Forcing Function

Figure 3.9-2 Plastic Model of Reactor Coolant System - Plan View

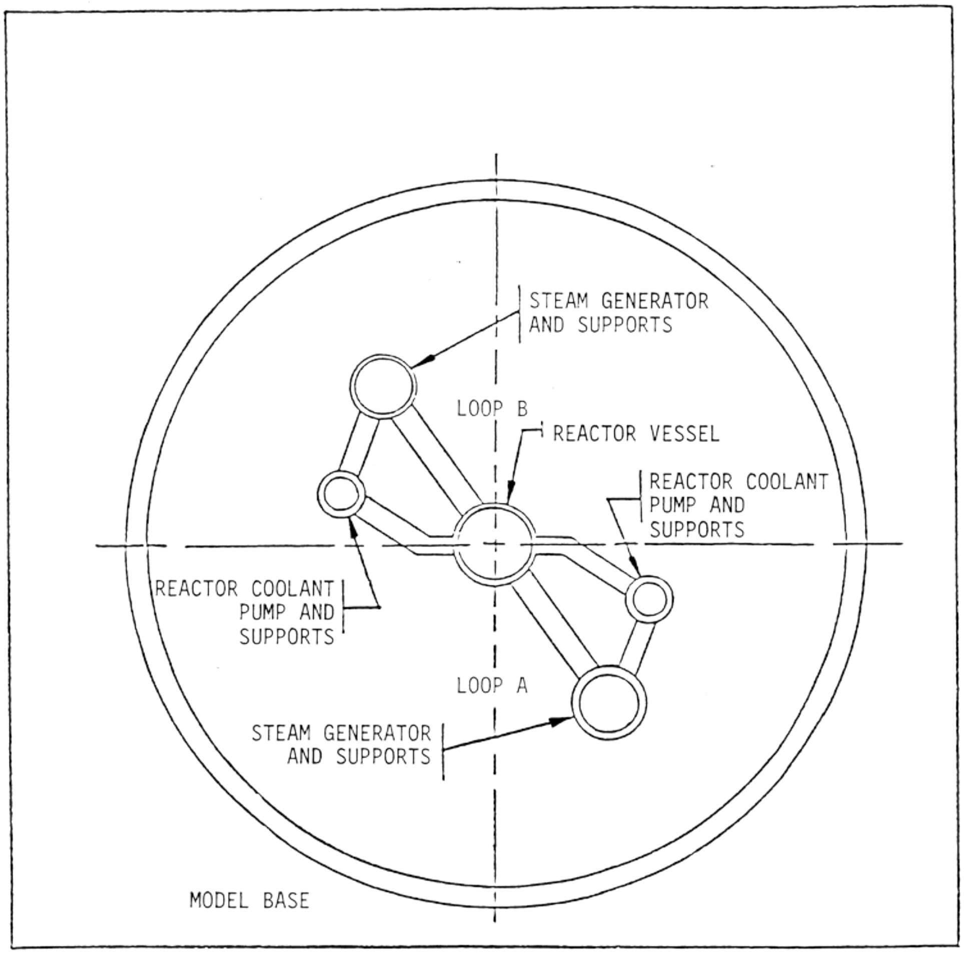

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Figure 3.9-2 Plastic Model of Reactor Coolant System - Plan View

Figure 3.9-3 Lumped Mass Dynamic Model of PCV 434

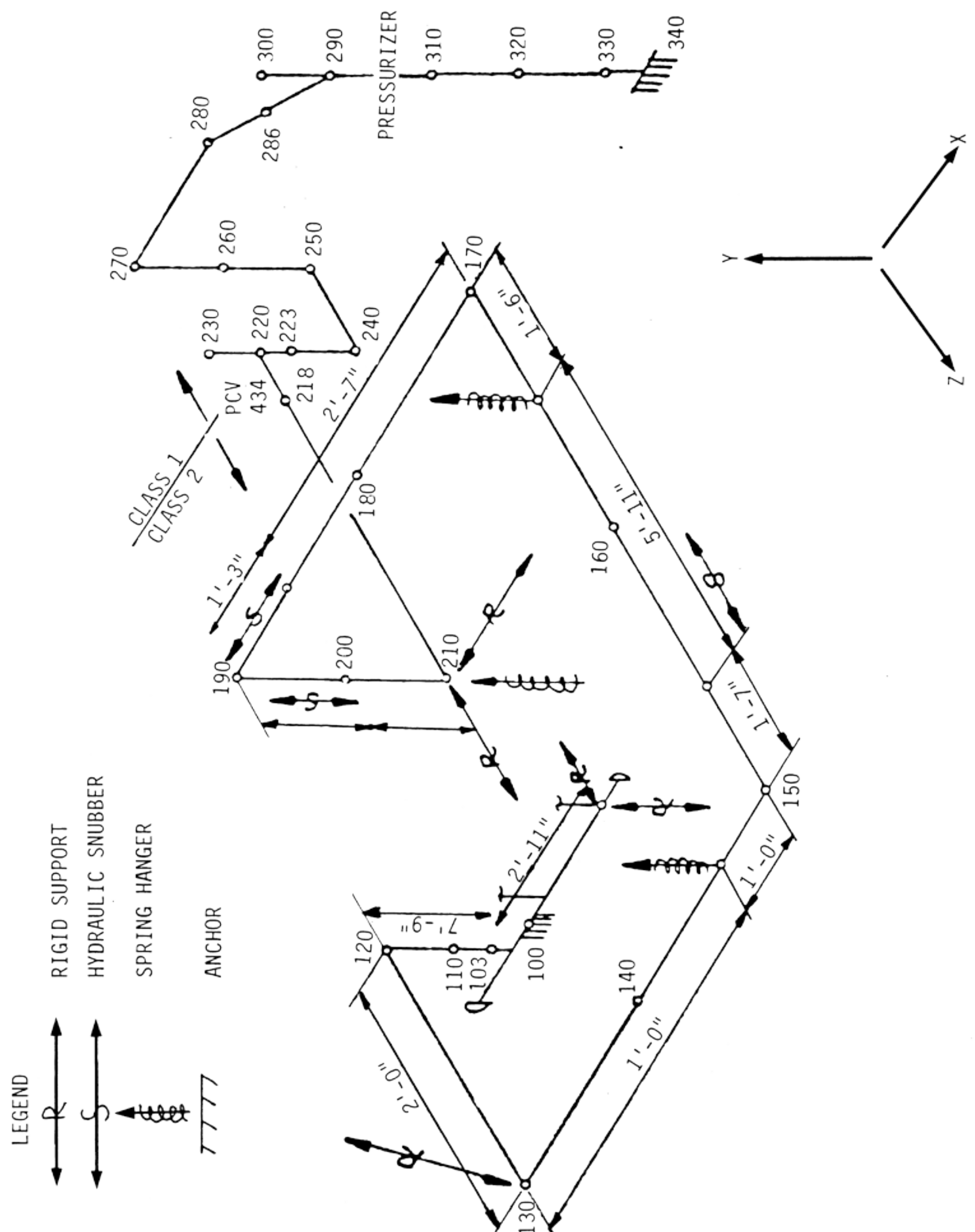

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Figure 3.9-3
Lumped Mass Dynamic Model of PCV 434

Figure 3.9-4 Lumped Mass Dynamic Model of PCV 435

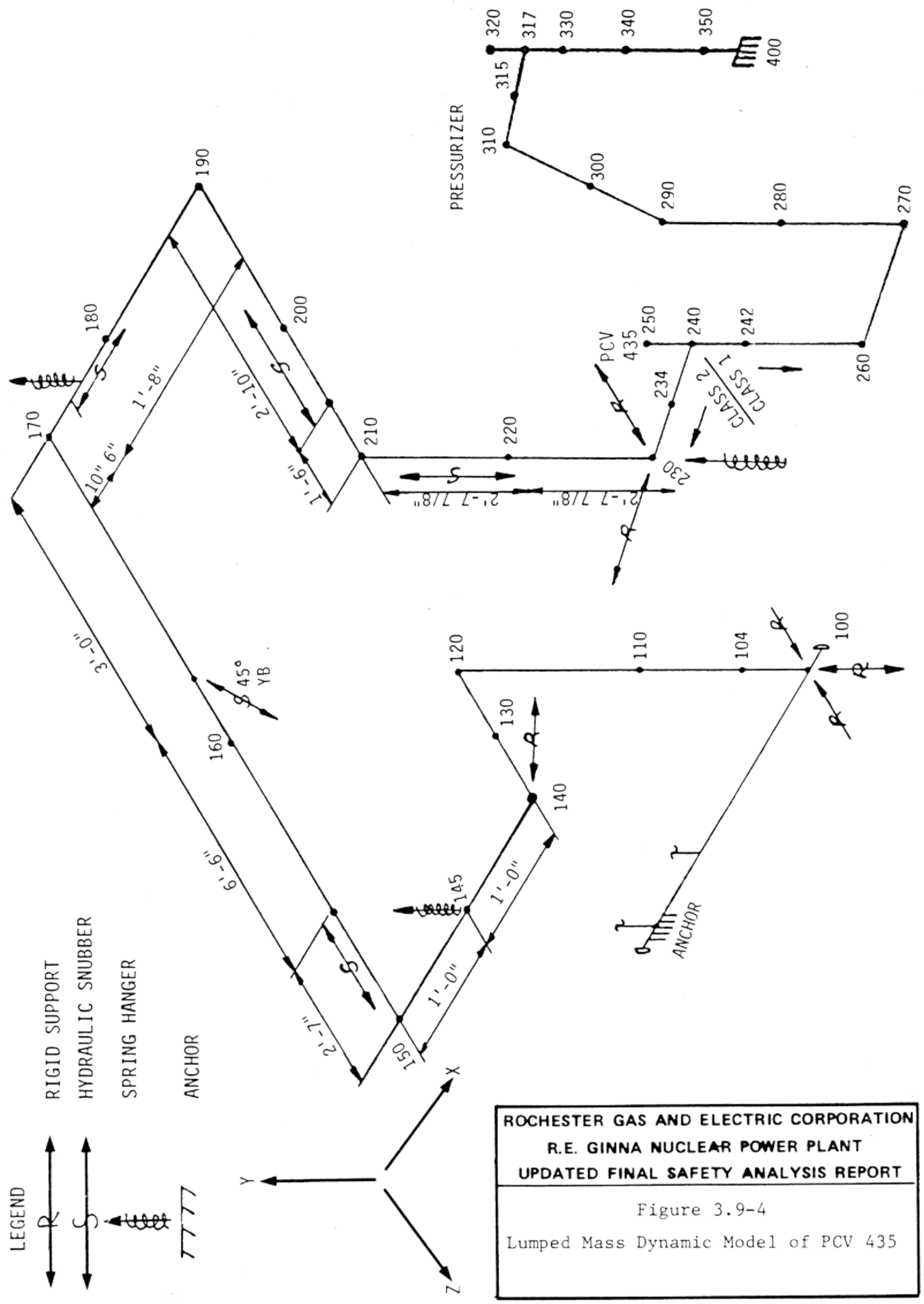

Figure 3.9-5 Comparison of WHAM Results With LOFT Semi-Scale Blowdown Experiments, Test No. 519

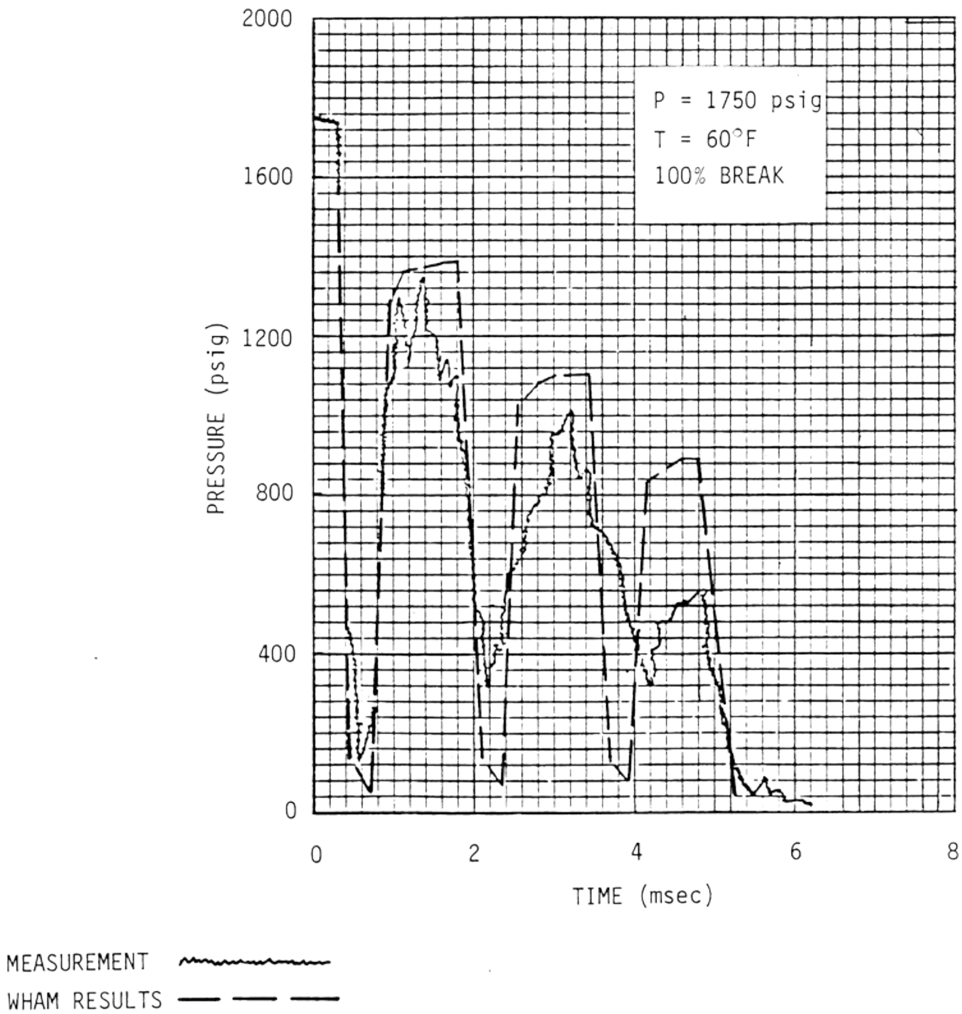

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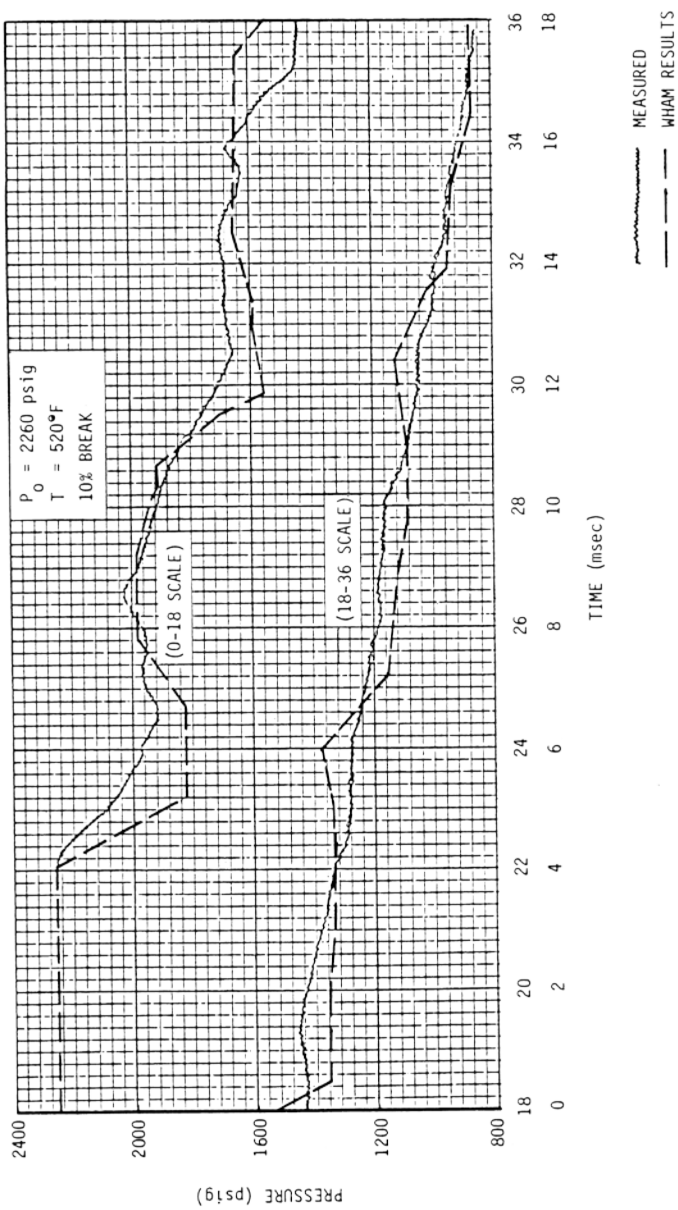
Figure 3.9-5 Comparison of WHAM Results With LOFT Semi-Scale Blowdown Experiments, Test No. 519

Figure 3.9-6 Comparison of WHAM Results With LOFT Semi-Scale Blowdown Experiments, Test No. 560



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R.E. GINNA NUCLEAR POWER PLANT
UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 3.9-6
Comparison of WHAM Results With LOFT Semi-Scale Blowdown Experiments, Test No. 560

Figure 3.9-6a Steam Generator Upper Support Systems

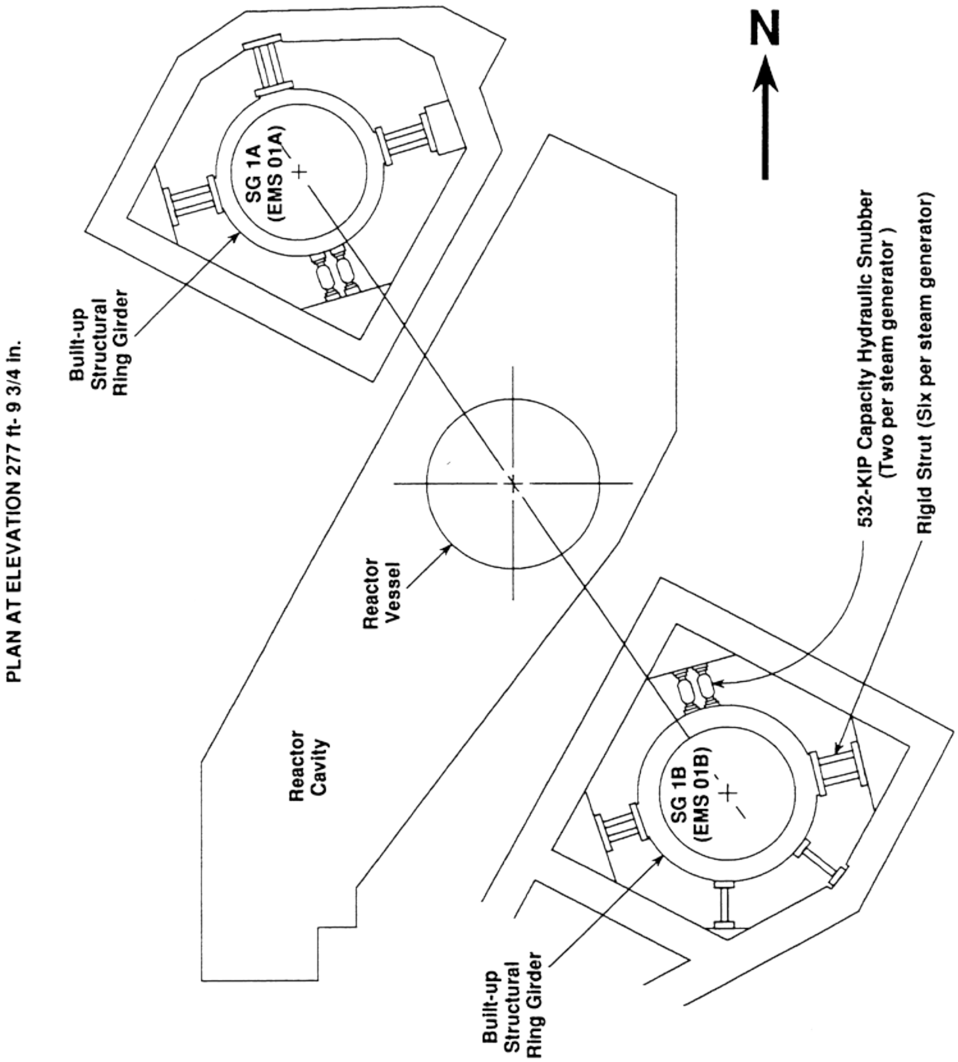

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Figure 3.9-6a Steam Generator Upper Support Systems

REV 10 12/93

Figure 3.9-7 Control Rod Drive Mechanism Assembly

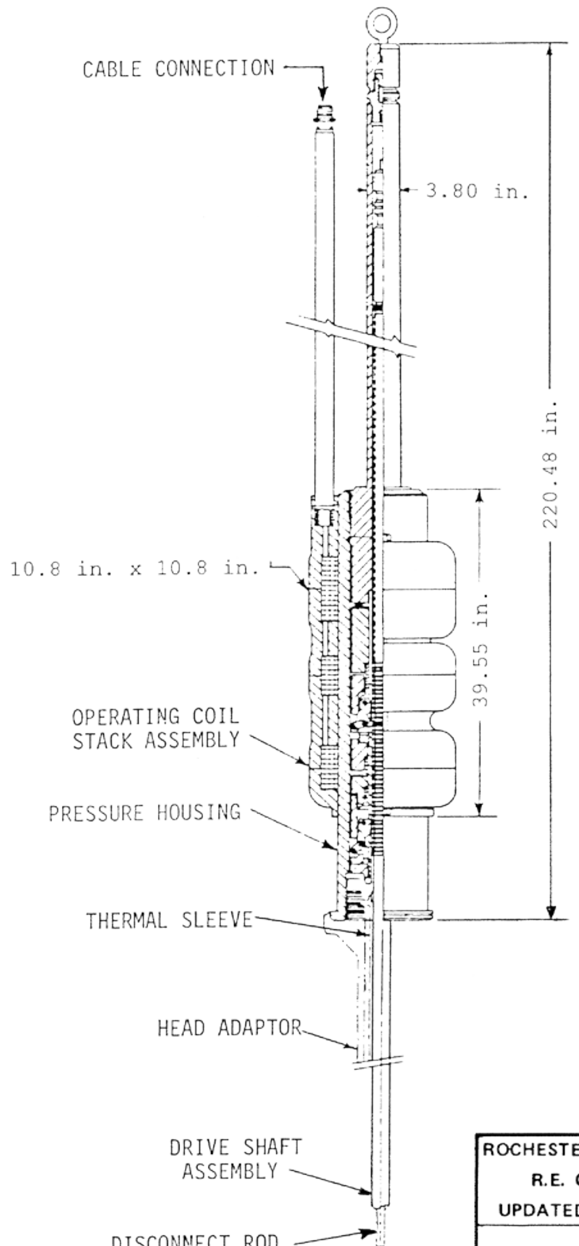

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Figure 3.9-7
Control Rod Drive Mechanism Assembly

REV. 15 10/99

Figure 3.9-8 Control Rod Drive Mechanism Schematic

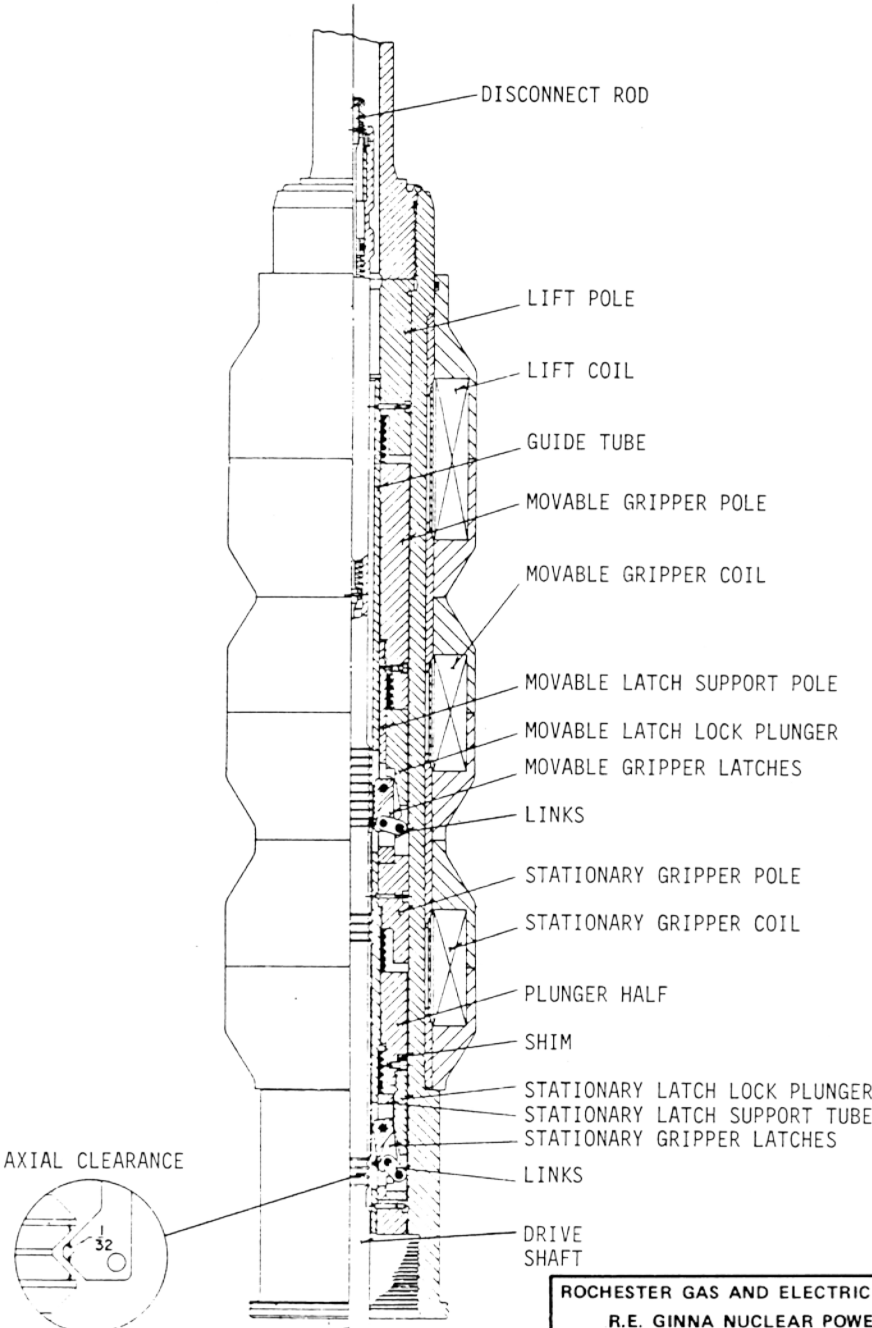

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Figure 3.9-8
Control Rod Drive Mechanism Schematic

Figure 3.9-9 Reactor Vessel Internals

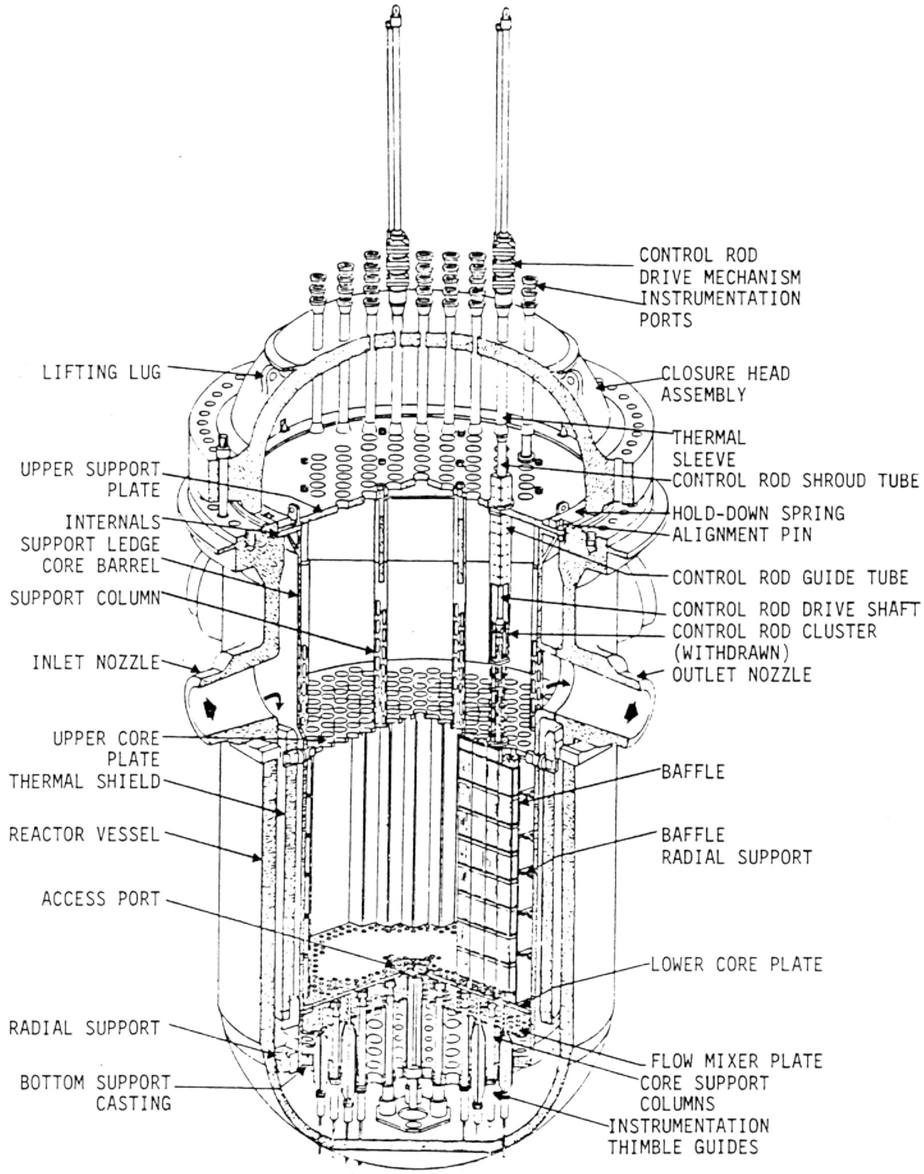

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Figure 3.9-9 Reactor Vessel Internals

Figure 3.9-10 Detailed View of Reactor Vessel Internals

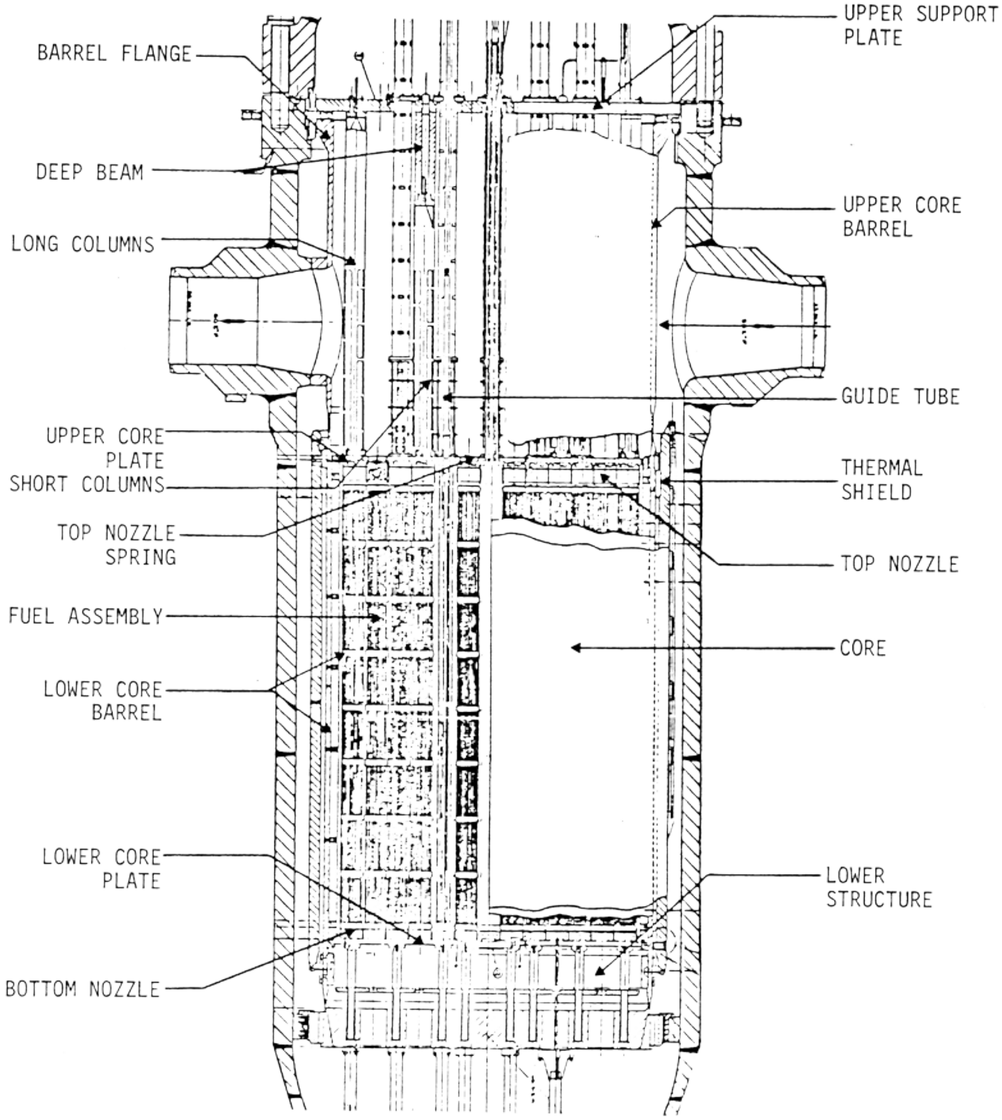

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Figure 3.9-10 Detailed View of Reactor Vessel Internals

Figure 3.10-1 Q-Deck Detail

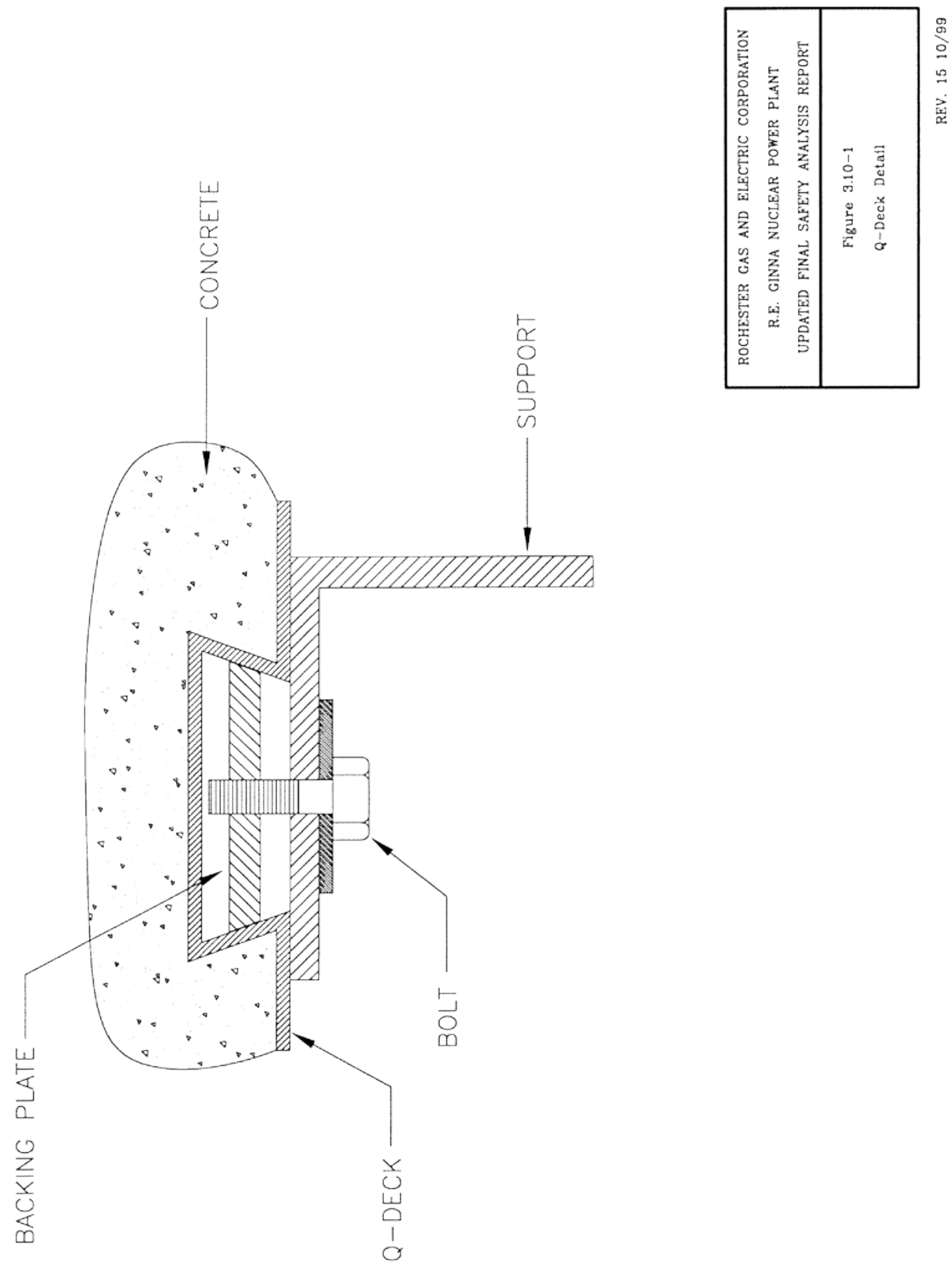

Figure 3.10-2 Unistrut Detail

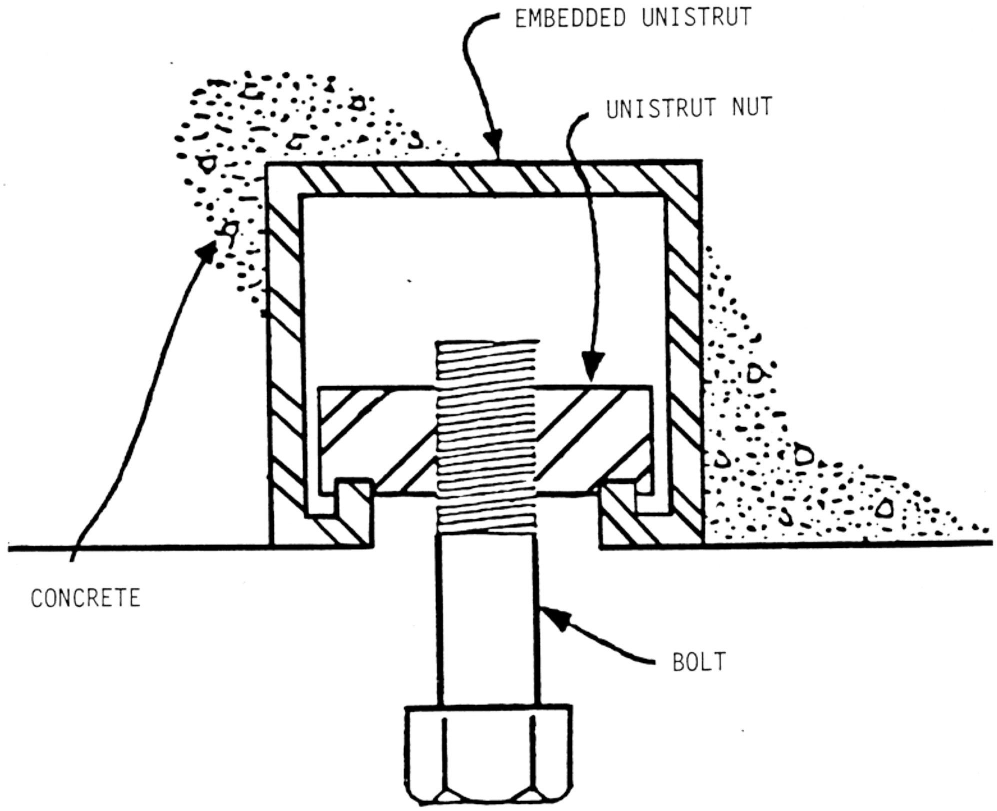

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Figure 3.10-2 Unistrut Detail

Figure 3.10-3 Threaded Insert Detail Poured in Place Anchor

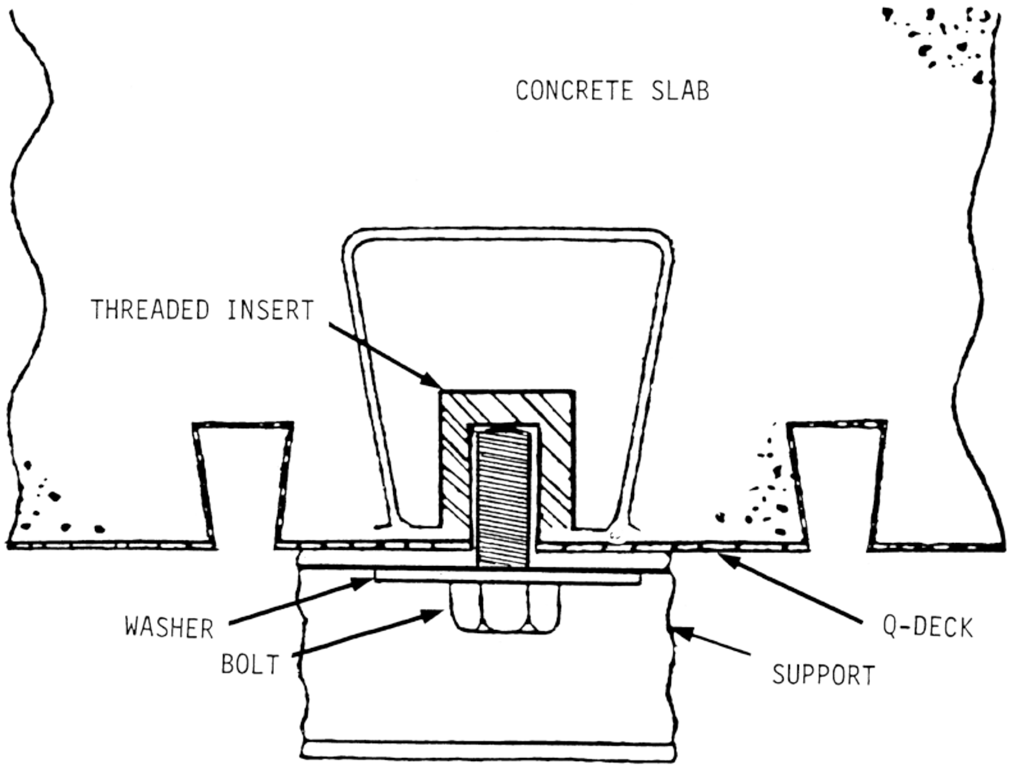

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Figure 3.10-3 Threaded Insert Detail Poured in Place Anchor

Figure 3.10-4 Tray Support Types for Friction Bolt Testing

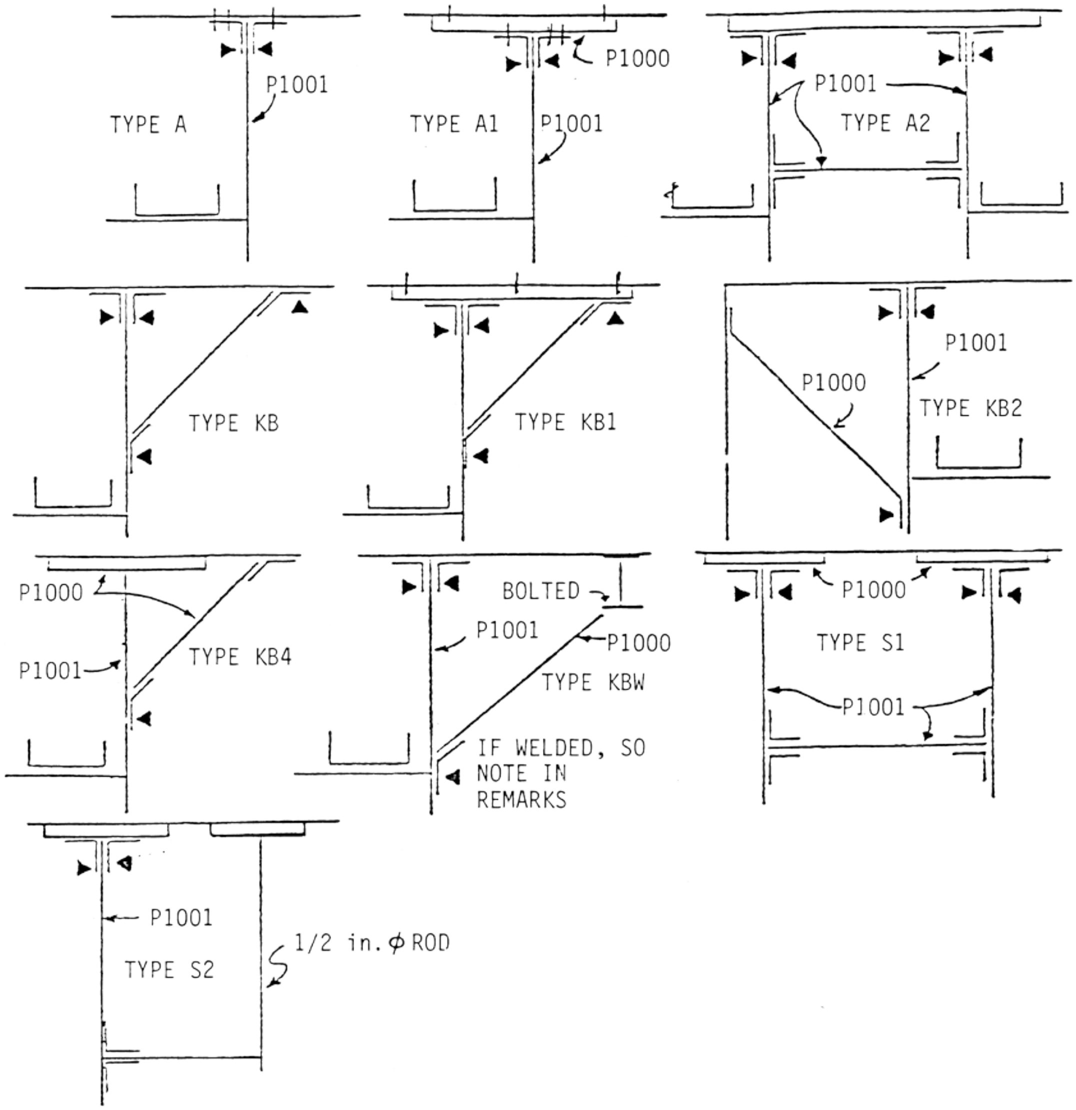

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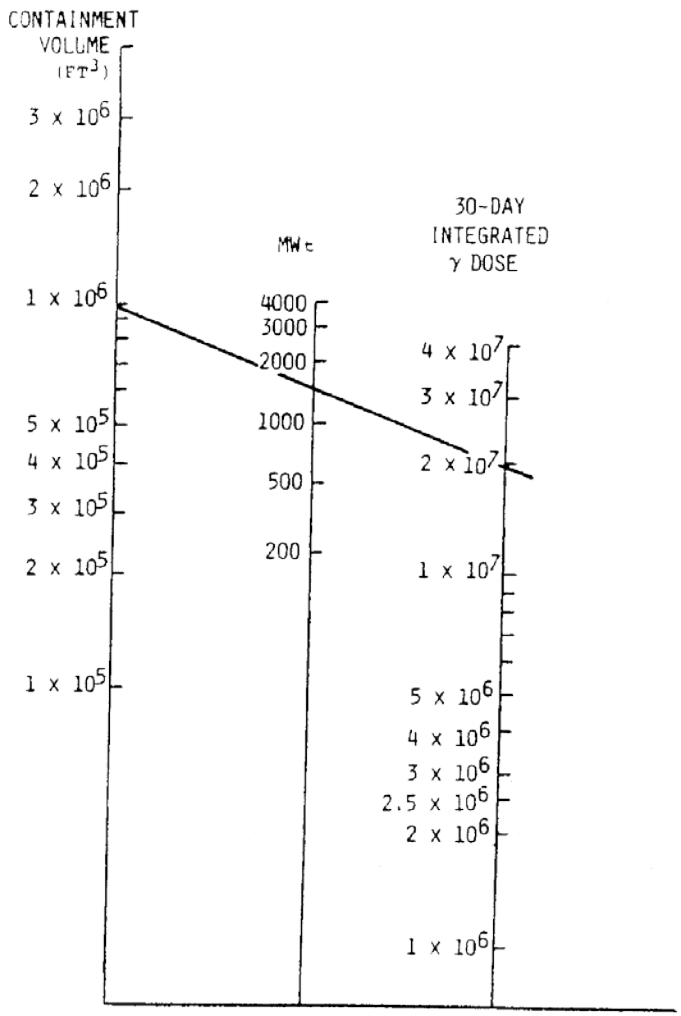
Figure 3.10-4
Tray Support Types for Friction Bolt Testing

Figure 3.11-1 Containment Volume and Reactor Power LOCA Dose Corrections



NOTES:

1. MAIN STEAM LINE BREAK ACCIDENT DOSES SHOULD BE READ AS A FACTOR OF 10 LESS
2. INITIAL EVALUATION BASED ON DIVISION OF OPERATING REACTORS (DOR) GUIDELINES AND A POWER LEVEL OF 1520 MWt (HISTORICAL)

4 REACTOR

4.1 SUMMARY DESCRIPTION

4.1.1 REACTOR CORE

The reactor core is a multi-region core containing 121 fuel assemblies. The fuel rods are cold worked ZIRLO™ tubes containing slightly enriched uranium dioxide fuel.

The fuel assembly is a canless type with the basic assembly consisting of the rod cluster control guide thimbles fastened to the grids and the top and bottom nozzles. The fuel rods are held by the grids and grid springs which provide lateral and axial support for the fuel rods.

Full-length rod cluster control assemblies (also commonly referred to as control rods) are inserted into the guide thimbles of the fuel assemblies. The absorber sections of the control rods are fabricated of silver-indium-cadmium alloy sealed in stainless steel tubes.

There are 29 full-length control rods. The control rod drive mechanisms are of the magnetic latch type. The latches are controlled by three magnetic coils. They are so designed that upon a loss of power to the coils, the rod cluster control assembly is released and falls by gravity to shut down the reactor.

4.1.2 WESTINGHOUSE OPTIMIZED FUEL ASSEMBLIES/422 VANTAGE + FUEL ASSEMBLIES

The transition to an all-Westinghouse Optimized Fuel Assembly (OFA) core began in cycle 14 (region 16) in the spring of 1984 and was completed at the start of cycle 21 (region 23) in the spring of 1991. Ginna operated with all-OFA cores through the end of cycle 32 (region 34) in the fall of 2006. OFA fuel assemblies consist of 179 0.400-inch diameter rodlets, of which typically 8 to 120 contain a burnable absorber consisting of a thin enriched zircdiboride coating on the surface of the fuel pellets. Solid natural uranium axial blankets were also introduced with OFA in cycle 14 (region 16).

The OFA top and bottom nozzles are fabricated from stainless steel. Both nozzles index the fuel assembly in the core and direct flow into and out of the assembly through perforated nozzle plates. The axial spacing between the top and bottom nozzle is established to accommodate the growth of the fuel rods due to irradiation effects on the zircaloy fuel tube. The optimized fuel assembly bottom nozzle can be removed to facilitate reconstitution. In cycle 20 (region 22) a removable top nozzle feature was also added to simplify reconstitution.

For cycle 21 (region 23) the debris filter bottom nozzle was introduced into the fuel assemblies to help reduce the possibility of fuel rod damage due to debris-induced fretting. The stainless steel debris filter bottom nozzle is similar to the conventional bottom nozzle design used previously. However, the debris filter bottom nozzle design incorporates a modified flow hole size and pattern. The relatively large flow holes in a conventional bottom nozzle are replaced with a new pattern of smaller flow holes in the debris filter bottom nozzle. The holes are sized to minimize passage of debris particles large enough to cause damage. The hole sizing was also designed to provide sufficient flow area, comparable pressure drop, and continued structural integrity of the nozzle. Significant testing to measure pressure drop and

demonstrate structural integrity has been performed to verify that the debris filter bottom nozzle is totally compatible with the previous design.

An additional level of debris damage mitigation was added to the bottom portion of each rodlet in cycle 25 (region 27), with the introduction of a pre-oxidized protective coating on the lowermost portion of the fuel rod cladding, further guards against debris-induced damage at the bottom grid location. The oxide coating is applied to the outside diameter surface of the bottom of the fuel rod cladding using an induction heating process which is indistinguishable from in-reactor oxidation. The end result of the oxide coating process is to accelerate the oxidation process that naturally occurs in-core. Analyses explicitly account for the thermal effects of the oxide coating and confirm that even with the initial coating, the limiting naturally occurring oxide at the higher temperature elevations bounds the maximum expected oxide thickness in the coated segment. Fuel rod performance and the core safety considerations are not adversely affected because the oxide coating is a naturally occurring phenomenon accounted for in the fuel performance and thermal-hydraulic models.

Also introduced in cycle 25 (region 27) were mid-enriched (2.6% U^{235}) axial blankets. A further enhancement to the axial blankets ocurred in cycle 27 (region 29) when annular axial blankets were introduced. Annular blankets allow additional volume to reduce rod internal pressure concerns.

In cycle 28 (Region 30), VANTAGE + fuel product features were introduced into the Ginna OFA fuel assemblies. These features included ZIRLO™ clad fuel rods, ZIRLO™ fabricated guide thimble tubes and instrumentation tubes. ZIRLO™ is a zirconium-based alloy that improves fuel assembly corrosion resistance and dimensional stability under irradiation. The chemical composition of the fuel rods and core components fabricated with ZIRLO™ alloy is similar to the previous components fabricated from Zircaloy-4 except for a slight reduction in the content of Tin (Sn), Iron (Fe) and the elimination of Chromium (Cr). The ZIRLO™ alloy also contains a nominal amount of Niobium (Nb). These composition changes, although small, are responsible for the improved corrosion resistance of ZIRLO™ compared to Zircaloy-4.

The Region 30 ZIRLO™ fuel rod is the same length as the Zircaloy-4 clad fuel rods used in the previous regions. Since the ZIRLO™ clad fuel rods grow less than their Zircaloy-4 equivalents, the design is capable of accommodating extended lead rod average burnups beyond 60,000 MWD/MTU. Also, since ZIRLO™ has improved corrosion resistance as compared to Zircaloy-4, the ZIRLO™ clad fuel rods will have more margin to the rod internal pressure limit than their Zircaloy-4 equivalents.

Holddown of the Ginna fuel assemblies is provided by four sets of double-leaf springs. The Inconel 718 spring design permits both a high spring rate and large travel, which is required to accommodate the difference in thermal expansion between the zircaloy/ZIRLO™ thimbles and the stainless steel reactor internals. This spring design also accommodates the growth of the zircaloy/ZIRLO™ thimbles during service and prevents fuel assembly liftoff during MODES 1 and 2.

The fuel rod fretting evaluation performed on a Westinghouse 14 x 14 seven-grid OFA design has shown that even with no grid spring force acting on the fuel rod by the five zircaloy grids at end-of-life, the clad wear criterion is met. Since the Ginna OFA design contains nine grids, including seven zircaloy or ZIRLO™ zircaloy grids, considerable additional wear margin exists for the fuel design to that for the seven-grid design.

The rod bow behavior of the optimized fuel assembly is expected to be better than that of the seven-grid Westinghouse fuel assembly. The optimized fuel assembly will have reduced grid spring forces due to the shorter span lengths and a higher fuel tube thickness-to-diameter ratio than the seven-grid fuel assembly. The zircaloy grid spring forces are lower during service than those typically used on Inconel grids. Therefore, lower friction forces are generated by the differential thermal expansion and irradiation growth of the fuel rods. This results in lower loads applied to the skeleton components than are present in the seven-grid Westinghouse assemblies. The skeleton components are conservatively designed to accept these loads with an adequate safety margin. The same conclusions apply to the OFA fuel with VANTAGE + features since the differential thermal expansion frictional forces are the same and the irradiation growth frictional forces are substantially less than those in the optimized fuel assembly.

A fuel transition to a Ginna specific version of the 422VANTAGE + (422V+) fuel assembly will occur in cycle 33 (region 35) to provide increased margins and uranium loading for the implementation of Extended Power Uprate (EPU) to 1775 MWt. This fuel assembly has a larger 0.422-inch diameter rod similar to the original Westinghouse fuel assemblies used at Ginna in cycles 1 through 7. The 422V+ assembly also has a reduced height standard top nozzle, allowing longer fuel rods and an increase in the fuel stack height from 141.4 to 143.25 inches.

All fuel resident in the Ginna core is now OFA with VANTAGE+ features (OFA/VANTAGE+) or 422V+.

The control rods used in the reactor core are compatible with the OFA and 422V+ fuel assemblies. Secondary sources were removed during the cycle 20/21 refueling since neutrons created by spontaneous fission in the burnt fuel provide sufficient source range detector response. Cycle 21 was operated with one thimble plug removed and a program for partial or full core thimble plug assembly removal was implemented starting in cycle 22. The resulting increase in core bypass flow is accounted for in the Chapter 15 safety analysis.

4.1.3 RECONSTITUTED FUEL ASSEMBLIES

Ginna is licensed to use reconstituted fuel assemblies in reload cores. Each 14 x 14 fuel assembly includes 179 fuel rod locations, 16 guide tubes, and one instrument thimble. Fuel rods that are known to be defective can be replaced with filler rods that are either zircaloy, ZIRLO™ or stainless steel. This reconstitution process permits the continued use of these reconstituted fuel assemblies without increasing coolant activity (*Reference 2*).

4.1.4 STARTUP REPORT

A summary report of plant startup and power escalation testing shall be submitted following: (1) amendment to the operating license involving a planned increase in power level, (2) installation of fuel that has a different design or has been manufactured by a different fuel supplier, or (3) modifications that may have significantly altered the nuclear, thermal, or hydraulic performance of the plant. The report shall address each of the tests performed and shall in general include a description of the measured values of the operating conditions or characteristics obtained during the test program and a comparison of these values with design predictions and specifications. Any corrective actions that were required to obtain satisfactory operation shall also be described. Any additional specific details required in license conditions based on other commitments shall be included in this report.

Startup reports shall be submitted within (1) 90 days following completion of the startup test program, or (2) 90 days following resumption of commercial power operation, whichever is earliest. If the Startup Report does not cover both events (i.e., completion of startup test program, and resumption of commercial power operation), supplementary reports shall be submitted at least every three months until both events have been completed.

REFERENCES FOR SECTION 4.1

1. Letter from J. E. Maier, RG&E, to H. R. Denton, NRC, Subject: Application for Amendment to OL, Westinghouse 14 x 14 Optimized Fuel for Cycle 14, dated December 20, 1983.

2. Westinghouse Electric Corporation, Westinghouse Fuel Assembly Reconstitution Evaluation Methodology, WCAP-13061-NP-A, July 1993.

3. S. L. Davidson, VANTAGE + Fuel Assembly Reference Core Report, WCAP-12610-P-A, April 1995, Westinghouse Proprietary.

4.2 FUEL SYSTEM DESIGN

4.2.1 DESIGN BASES

This section describes the fuel system design bases from the standpoint of performance objectives, principal design criteria, and safety limits.

4.2.1.1 Performance Objectives

The fuel rod cladding is designed to maintain its integrity for the anticipated core life. The effects of gas release, fuel dimensional changes, and corrosion-induced or irradiation-induced changes in the mechanical properties of cladding are considered in the design of fuel assemblies.

The control rods, being long and slender, are relatively free to conform to any small misalignments. Tests show that the rods are very easily inserted and not subject to binding even under conditions of severe misalignments. In order to address issues of control rod binding as described in USNRC Information Notice 96-01, fuel assemblies are evaluated on a cycle basis for susceptibility to Incomplete Rod Insertion (IRI). Where warranted, detailed analysis is performed during the cycle design phase to ensure that IRI is precluded. The control rods provide sufficient control rod worth to shut the reactor down with sufficient shutdown margin in the hot condition at any time during the cycle life with the most reactive control rod stuck in the fully withdrawn position. Redundant equipment is provided to add soluble poison to the reactor coolant to ensure a similar shutdown capability when the reactor coolant is cooled to ambient temperatures.

Measurements from critical experiments or operating reactors, or both, are used to validate the methods employed in the design. During design, nuclear parameters are calculated for every phase of operation of each core cycle and, where applicable, are compared with design limits to show that an adequate margin of safety exists.

In the thermal hydraulic design of the core, the maximum fuel and clad temperatures during normal reactor operation and at power densities up to the design limit are conservatively evaluated and found to be consistent with safe operating limitations.

4.2.1.2 Principal Design Criteria

The design criteria cited in Sections 4.2.1.2.1 through 4.2.1.2.8 were used during the design and initial licensing of Ginna Station. They represent the Atomic Industrial Forum version of proposed criteria issued by the AEC for comment on July 10, 1967. Therefore, they are identified as AIF-GDCs. Conformance with the applicable General Design Criteria of 10 CFR 50, Appendix A, identified in Section 4.2.1.2.9, is discussed in Section 3.1.2.

4.2.1.2.1 Reactor Core Design

CRITERION: The reactor core with its related controls and protection systems shall be designed to function throughout its design lifetime without exceeding acceptable fuel damage limits which have been stipulated and justified. The core and related auxiliary system designs shall provide this integrity under all expected

conditions of MODES 1 and 2 with appropriate margins for uncertainties and for specified transient situations which can be anticipated (AIF-GDC 6).

The reactor core, with its related control and protection system, is designed to function throughout its design lifetime without exceeding acceptable fuel damage limits. The core design, together with reliable process and decay heat removal systems, provides for this capability under all expected conditions of MODES 1 and 2 with appropriate margins for uncertainties and anticipated transient situations, including the effects of the loss of reactor coolant flow (Section 15.3), loss of electrical load (Section 15.2.2), loss of normal feedwater (Section 15.2.6), and loss of all offsite power (Section 15.2.5).

The reactor control protection system is designed to actuate a reactor trip for any anticipated combination of plant conditions, when necessary, to ensure departure from nucleate boiling (DNB) will not occur on the limiting fuel rods during MODES 1 and 2, operational transients, or transient conditions of moderate frequency, with a 95% probability and at a 95% confidence level. The integrity of the fuel cladding is ensured by preventing excessive fuel swelling, excessive cladding overheating, and excessive cladding stress and strain. This is achieved by designing the fuel rods so that the following conservative limits are not exceeded during MODES 1 and 2 or any anticipated transient condition:

A. DNB on limiting fuel rod does not occur.

B. Fuel center line temperature below melting point of uranium dioxide.

C. Internal gas pressure less than that required to increase the fuel clad diametral gap during MODES 1 and 2 and cause extensive DNB propagation to occur.

D. Clad stresses less than the yield strength of the cladding material.

E. Clad strain for MODES 1 and 2 is limited to 1% from the unirradiated condition. The transient limit is 1% from the pre-transient value.

The ability of fuel designed and operated to these criteria to withstand postulated normal and abnormal service conditions is shown by analyses described in Chapter 15 to satisfy the demands of plant operation well within applicable regulatory limits.

Should pellet/clad gap reopening be predicted to occur during the design cycle, analyses will be performed to show that applicable regulatory limits (17% oxidation) are still met.

The reactor coolant pumps provided for the plant are supplied with sufficient rotational inertia to maintain an adequate flow coastdown in the event of a simultaneous loss of power to all pumps. The flow coastdown inertia is sufficient such that the reduction in heat flux obtained with a low-flow reactor trip prevents core damage.

In the unlikely event of a turbine trip from full power without an immediate reactor trip, the subsequent transient results in a high pressurizer pressure, overtemperature delta T, or low steam generator water level trip and thereby prevents fuel damage for this transient. A loss of external electrical load at 50% of full power or less is normally controlled by rod cluster insertion together with a controlled steam dump to the condenser and atmosphere to prevent a large temperature and pressure increase in the reactor coolant system and thus prevent a reactor trip. In this case, the overpower-overtemperature protection would guard against any

combination of pressure, temperature, and power which could result in fuel centerline melting or DNB during the transient.

Neither the turbine trip nor the loss-of-flow events cause changes in coolant conditions to provoke a large nuclear power excursion because of the large system thermal inertia and relatively small void fraction. Protection circuits actuated directly by the coolant conditions identified with core limits are therefore effective in preventing core damage.

4.2.1.2.2 Suppression of Power Oscillations

CRITERION: The design of the reactor core with its related controls and protection systems shall ensure that power oscillations, the magnitude of which could cause damage in excess of acceptable fuel damage limits, are not possible or can be readily suppressed (AIF-GDC 7).

The design of the reactor core and related protection systems ensures that power oscillations which could cause fuel damage in excess of acceptable limits are not possible or can be readily suppressed.

The potential for possible spatial oscillations of power distribution for this core has been reviewed. In summary it was concluded that the only potential spatial instability of a magnitude which could cause damage in excess of acceptable fuel damage limits was the xenon induced axial instability which may be a nearly free-running oscillation with little or no inherent damping. Part-length control rods were originally provided to suppress these oscillations, if they occurred. They have since been removed. Operating control strategies have been devised that do not require part-length rods and eliminate the potential for axial xenon instabilities.

Out-of-core instrumentation is provided to obtain necessary information concerning axial distributions. This instrumentation is adequate to enable the operator to monitor and control xenon-induced oscillations. In-core instrumentation is used to periodically calibrate and verify the information provided by the out-of-core instrumentation.

4.2.1.2.3 Redundancy of Reactivity Control

CRITERION: Two independent reactivity control systems, preferably of different principles, shall be provided (AIF-GDC 27).

Two independent reactivity control systems are provided, one involving rod cluster control assemblies and the other involving chemical shimming.

4.2.1.2.4 Reactivity MODE 3 (Hot Shutdown) Capability

CRITERION: The reactivity control systems provided shall be capable of making and holding the core subcritical from any hot operating (MODES 1 and 2) condition (AIF-GDC 28).

The reactivity control systems provided are capable of making and holding the core subcritical from any operating or hot standby condition, including those resulting from power

changes. The maximum excess reactivity for the core originally occurred for the cold, clean condition at the beginning-of-life of the initial core.

With the Extended Power Uprate (EPU) and transition to 422V+ fuel, cores with more excess reactivity than the previous cores will be loaded.

The rod cluster control assemblies are divided into two categories comprising a control group and shutdown groups. The control group, used in combination with chemical shim control, provides control of the reactivity changes of the core throughout the life of the core at power conditions. This group of rod cluster control assemblies is used to compensate for short-term reactivity changes at power that might be produced due to variations in reactor power requirements or in coolant temperature. Chemical shim control is used to compensate for the more slowly occurring changes in reactivity throughout core life, such as those due to fuel depletion and fission product buildup.

4.2.1.2.5 Reactivity Shutdown Capability

CRITERION: One of the reactivity control systems provided shall be capable of making the core subcritical under any anticipated operating condition (including anticipated operational transients) sufficiently fast to prevent exceeding acceptable fuel damage limits. Shutdown margin should assure subcriticality with the most reactive control rod fully withdrawn (AIF-GDC 29).

The control rod system provided is capable of making the core subcritical under any condition (including anticipated operational transients) sufficiently fast to prevent exceeding acceptable fuel damage limits. The shutdown margin ensures subcriticality with the most reactive control rod fully withdrawn. The shutdown groups are provided to supplement the control group of rod cluster control assemblies to make the reactor subcritical with the required shutdown margin following trip from any credible operating condition to the hot zero-power condition assuming the most reactive rod cluster control assembly remains in the fully withdrawn position. Manually controlled boric acid addition is used to supplement the rod cluster control assemblies in maintaining the shutdown margin for the long-term conditions of xenon decay or plant cooldown. See Section 9.3.4 concerning details of the chemical and volume control system.

4.2.1.2.6 Reactivity Holddown Capability

CRITERION: The reactivity control systems provided shall be capable of making the core subcritical under credible accident conditions with appropriate margins for contingencies and limiting any subsequent return to power such that there will be no undue risk to the health and safety of the public (AIF-GDC 30).

The reactivity control systems provided are capable of making and holding the core subcritical under accident conditions in a timely fashion with appropriate margins for contingencies. Normal reactivity shutdown capability is provided within two seconds following a trip signal by control rods. Boric acid injection is used to compensate for the long-term xenon decay transient and for plant cooldown. Any time that the reactor is at power, the quantity of boric acid retained in the boric acid storage tanks or refueling water storage tank (RWST) and ready

for injection always exceeds that quantity required to reach MODE 5 (Cold Shutdown) while maintaining the required minimum shutdown margin. This quantity will also exceed the quantity of boric acid required to bring the reactor to MODE 3 (Hot Shutdown) and to compensate for subsequent xenon decay.

Boric acid is pumped from the boric acid storage tanks by one of two boric acid transfer pumps to the suction of one of three charging pumps which injects boric acid into the reactor coolant. Any charging pump and either boric acid transfer pump can be operated from diesel generator power on loss of outside power. Boric acid can be injected by one charging pump operating at the nominal charging flow rate of 46 gpm and shut the reactor down with no rods inserted in approximately 81 minutes. Sufficient boric acid from the Boric Acid Storage Tanks (BAST) o RWST can be injected to compensate for xenon decay beyond the equilibrium level, with one charging pump operating at its minimum speed, and thereby delivering in excess of the required minimum of approximately 9 gpm into the reactor coolant system. If two charging pumps (or one pump at greater than minimum flow in the xenon decay case) and two boric acid transfer pumps are available, these time periods are reduced. Although the charging pumps are larger capacity than the boric acid transfer pumps, it is desirable to operate two charging pumps if two boric acid transfer pumps are operated. Additional boric acid injection is employed if it is desired to bring the reactor to MODE 5 (Cold Shutdown) conditions.

On the basis of the above, the injection of boric acid is shown to afford backup reactivity shutdown capability, independent of control rod clusters which normally serve this function in the short-term situation. Shutdown for long-term and reduced temperature conditions can be accomplished with boric acid injection using redundant components. Alternately, boric acid solution at lower concentration can be supplied from the refueling water storage tank (RWST). This solution can be transferred directly by the charging pumps or alternately by the safety injection pumps. The reduced boric acid concentration lengthens the time required to achieve equivalent shutdown.

4.2.1.2.7 Reactivity Control Systems Malfunction

CRITERION: The Reactor Trip System (RTS) shall be capable of protecting against any single malfunction of the reactivity control systems, such as unplanned continuous withdrawal (not ejection or dropout) of a control rod, by limiting reactivity transients to avoid exceeding acceptable fuel damage limits (AIF-GDC 31).

The Reactor Trip System (RTS) is capable of protecting against any single anticipated malfunction of the reactivity control system, by limiting reactivity transients to avoid exceeding acceptable fuel damage limits.

Reactor shutdown with rods is completely independent of the normal rod control functions since the trip breakers completely interrupt the power to the rod mechanisms regardless of existing control signals.

Details of the effects of continuous withdrawal of a control rod are described in Section 15.4.1 and Section 15.4.2 and Section 15.4.4 describes the effects of continuous deboration.

4.2.1.2.8 Maximum Reactivity Worth of Control Rods

CRITERION: Limits, which include reasonable margin, shall be placed on the maximum reactivity worth of control rods or elements and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change of reactivity cannot (A) rupture the reactor coolant pressure boundary or (B) disrupt the core, its support structures, or other vessel internals sufficiently to lose capability of cooling the core (AIF-GDC 32).

Limits, which include considerable margin, are placed on the maximum reactivity worth of control rods or elements and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change of reactivity cannot (a) rupture the reactor coolant pressure boundary, or (b) disrupt the core, its support structures, or other vessel internals so as to lose capability to cool the core.

The reactor control system employs control rod clusters, greater than half of which are fully withdrawn during power operation, serving as shutdown rods. The remaining rods comprise the controlling group which are used to control load and reactor coolant temperature. The rod cluster drive mechanisms are wired into preselected groups, and are therefore prevented from being withdrawn in other than their respective groups. The rod drive mechanism is of the magnetic latch type and the coil actuation is sequenced to provide variable speed rod travel. The maximum reactivity insertion rate is analyzed in the detailed plant analysis assuming two of the highest worth groups to be accidentally withdrawn at maximum speed with 100% overlap, yielding reactivity insertion rates of the order of 9×10^{-4} Δk/sec which is well within the capability of the overpower-overtemperature protection circuits to prevent core damage.

No credible mechanical or electrical control system malfunction can cause a control rod to be withdrawn at a speed greater than 77 steps per minute.

4.2.1.2.9 Conformance With 1972 General Design Criteria

The adequacy of the Ginna Station reactor system design relative to the following General Design Criteria of 10 CFR 50, Appendix A, is discussed in Section 3.1.2.

GDC 1 Quality Standards and Records.

GDC 10 Reactor Design.

GDC 11 Reactor Inherent Protection.

GDC 12 Suppression of Reactor Power Oscillations.

GDC 13 Instrumentation and Control.

GDC 14 Reactor Coolant Pressure Boundary.

GDC 20 Protection System Functions.

GDC 25 Protection System Requirements for Reactivity Control Malfunctions.

GDC 26 Reactivity Control System Redundancy and Capability.

GDC 27 Combined Reactivity Control Systems Capability.

GDC 28 Reactivity Limits.

GDC 29 Protection Against Anticipated Operational Occurrences.

4.2.1.3 Safety Limits

The reactor is capable of meeting the performance objectives throughout core life under both steady-state and transient conditions without violating the integrity of the fuel elements. Thus the release of unacceptable amounts of fission products to the coolant is prevented.

The limiting conditions for operation specify the highest functional capacity or performance levels permitted to ensure safe operation of the facility.

Design parameters which are established by safety limits are specified below for the nuclear, control, thermal and hydraulic, and mechanical aspects of the design.

4.2.1.3.1 Nuclear Limits

At a full power level (license application power) the heat flux hot-channel factor, F_Q, specified in Table 4.2-1, is not exceeded.

The nuclear axial peaking factor $F^N{}_Z$, and the nuclear enthalpy rise hot-channel factor $F^N{}_{\Delta H}$ are limited to their combined relationship so as not to exceed the F_Q or DNBR limits.

Part-length control rods were provided in the original design in order to control axial xenon oscillations to preclude adverse core conditions. However, constant axial offset operating strategies have been devised that ensure adequate control of axial xenon oscillations without the necessity of using the part-length rods; thus, they were removed. The protection system ensures that the nuclear core limits are not exceeded.

4.2.1.3.2 Reactivity Control Limits

The control system and the operational procedures provide adequate control of the core reactivity and power distribution. The following control limits are met:

A. Sufficient control is available to produce the required MODE 3 (Hot Shutdown) shutdown margin.

B. The shutdown margin is maintained with the most reactive rod cluster control assembly stuck in the fully withdrawn position.

C. The shutdown margin is maintained at cold shutdown by the use of soluble poison.

4.2.1.3.3 Thermal and Hydraulic Limits

The reactor core is designed to meet the following limiting thermal and hydraulic criteria:

A. There is at least a 95% probability that DNB will not occur on the limiting fuel rods during MODES 1 and 2, operational transients, or any condition of moderate frequency at a 95% confidence level.

B. No fuel melting during any anticipated normal operating condition, operational transients, or any conditions of moderate frequency.

To maintain fuel rod integrity and prevent fission product release, it is necessary to prevent clad overheating under all operating conditions. This is accomplished by preventing DNB which causes a large decrease in the heat transfer coefficient between the fuel rods and the reactor coolant resulting in high clad temperatures.

The ratio of the heat flux causing DNB at a particular core location, as predicted by the W-3 correlation or the improved WRB-1 correlation, to the existing heat flux at the same core location is the DNBR. A Design Limit DNBR is defined in Section 4.4.3. Analytical assurance that DNB will not occur is provided by showing calculated DNBR to be higher for all conditions of normal operation, operational transients and transient conditions of moderate frequency. The Design Limit DNBR is chosen by using the Revised Thermal Design Procedure (RTDP) which includes appropriate margin to DNB for all operating conditions sufficient to assure compliance with the DNBR criteria above.

4.2.1.3.4 Mechanical Limits

4.2.1.3.4.1 Reactor Internals

The reactor internal components are designed to withstand the stresses resulting from startup, steady-state operation with any number of pumps running, and shutdown conditions. No damage to the reactor internals occurs as a result of loss of pumping power.

Lateral deflection and torsional rotation of the lower end of the core barrel is limited to prevent excessive movements resulting from seismic disturbances and thus prevent interference with rod cluster control assemblies. Core drop in the event of failure of the normal supports is limited so that the rod cluster control assemblies do not disengage from the fuel assembly guide thimbles.

The structural internals are designed to maintain their functional integrity in the event of a major loss-of-coolant accident. Analysis performed for limited size breaks reported in WCAP-9748 (Proprietary) and WCAP-9749 (Non-Proprietary) June 1980, Westinghouse Owners Group Asymmetric LOCA Load Evaluation - Phase C, showed that the appropriate systems and components will maintain their functional capability to ensure a safe plant shutdown with a coolable core geometry. The systems and components examined were the reactor vessel assembly including internals, fuel, control rod drive mechanisms, vessel and component supports, reactor coolant loop piping, and attached emergency core cooling piping. Furthermore, in the resolution of Unresolved Safety Issue A-2, Asymmetric Loading, it was concluded that an acceptable basis has been provided so that asymmetric blowdown loads resulting from double-ended pipe breaks in main coolant loop piping need not be considered as a design basis for Ginna Station, provided that leakage detection systems exist to detect postulated flaws utilizing guidance from Regulatory Guide 1.45. Conformance with Regulatory Guide 1.45 is discussed in Section 5.2.5.5.

The structural integrity of various reactor internal critical components was evaluated in *Reference 16* for operation over vessel average temperatures ranging from 559.0°F to 581.2°F. The

evaluation concluded that the critical components maintained structural integrity over the temperature range with the revised design transients associated with the replacement steam generators.

In 1998 the Nuclear Regulatory Commission issued Information Notice (IN) 98-11, "Cracking of Reactor Vessel Internal Baffle Former Bolts in Foreign Plants". Rochester Gas and Electric participated with the Westinghouse Owners Group to respond to these concerns and evaluate the structural integrity of Ginna's baffle-former-bolts. In 1999 an ultrasonic inspection was performed on all accessible bolts. As a result of this examination 56 bolts (from a total population of 728) were replaced. The replacement bolts were manufactured from type 316 stainless steel rather than the type 347 stainless steel used in the original bolts. Type 316 steel is believed to be less susceptible to the flaw initiation mechanism.

In 2011, ultrasonic inspections performed on the 56 bolts replaced in 1999 showed no defects. At the same time, ultrasonic inspections performed on 99 of the remaining original bolts only showed one bolt to have a defect. An additional 25 original bolts were also replaced with the improved type 316 stainless steel bolts and ultrasonic inspections performed on 24 of those removed original bolts showed no defects (one was destroyed during removal process). At three locations, bolt holes were left empty because damage occurred during the bolt removal process which prevented insertion of new replacement bolts. The presence of these empty bolt holes was evaluated with respect to structural integrity of the baffle assembly and possible damage to adjacent fuel assemblies. The evaluations, performed in accordance with NRC-approved methods, showed no adverse impact on either area.

In addition to the evaluations described above, the Ginna reactor internals components were evaluated for plant license renewal. System and component materials of construction, operating history and programs used to manage aging effects are documented in NUREG-1786, Safety Evaluation Report (SER) Related to the License Renewal of R. E. Ginna Nuclear Power Plant, May 2004.

The impact of the Extended Power Uprate (EPU) on the conclusions reached in the Ginna License Renewal Application for the reactor internals and core supports were assessed. The NRC SER for the EPU found the proposed EPU acceptable with respect to the design of the reactor internal and core supports.

4.2.1.3.4.2 Fuel Assemblies

The fuel assemblies are designed to perform satisfactorily throughout their lifetime. The loads, stresses, and strains resulting from the combined effects of flow induced vibrations, earthquakes, reactor pressure, fission gas pressure, fuel growth, thermal strain, and differential expansion during both steady-state and transient reactor operating conditions have been considered in the design of the fuel rods and fuel assembly. The assembly is also structurally designed to withstand handling and shipping loads prior to irradiation, and to maintain sufficient integrity at the completion of design burnup to permit safe removal from the core and subsequent handling during cooldown, shipment, and fuel reprocessing.

The fuel rods are supported at several locations along their length within the fuel assemblies by grid assemblies which are designed to maintain control of the lateral spacing between the

rods throughout the design life of the assemblies. The magnitude of the support loads provided by the grids are established to minimize possible fretting without overstressing the cladding at the points of contact between the grids and fuel rods. The grid assemblies also allow axial thermal expansion of the fuel rods without imposing restraint of sufficient magnitude to result in buckling or distortion of the rods.

The fuel rod cladding is designed to withstand operating pressure loads without collapse or rupture and to maintain encapsulation of the fuel throughout the design life.

4.2.1.3.4.3 Control Rods

The criteria used for the design of the cladding on the individual absorber rods in the control rods are similar to those used for the fuel rod cladding. The cladding is designed to be free-standing under all operating conditions and maintain encapsulation of the absorber material throughout the absorber rod design life. Allowance for wear during operation is included in the rod cluster control assembly cladding thickness. Adequate clearance is provided between the absorber rods and the guide thimbles which position the rods within the fuel assemblies so that coolant flow along the length of the absorber rods is sufficient to remove the heat generated without overheating of the absorber cladding. The clearance is also sufficient to compensate for any misalignment between the absorber rods and guide thimbles and to prevent mechanical interference between the rods and guide thimbles under any operating conditions.

4.2.1.3.4.4 Control Rod Drive Assembly

Each control rod drive assembly is designed as a hermetically sealed unit to prevent leakage of reactor coolant water. All pressure-containing components are designed to meet the requirements of the ASME Code, Section III, Nuclear Vessels for Class 1 Vessel appurtenances.

The control rod drive assemblies provide rod cluster control assembly insertion and withdrawal rates consistent with the required reactivity changes for reactor operational load changes. This rate is based on the worths of the various rod groups, which are established to limit power-peaking flux patterns to design values. The maximum reactivity addition rate is specified to limit the magnitude of a possible nuclear excursion resulting from a control system or operator malfunction.

Also, the control rod drive assemblies provide a fast insertion rate during a trip of the rod cluster control assemblies which results in a rapid shutdown of the reactor for conditions that cannot be handled by the reactor control system. This rate is based on the results of various reactor emergency analyses, including instrument and control delay times and the amount of reactivity that must be inserted before the rod cluster control assembly enters the dash-pot region.

4.2.2 FUEL SYSTEM DESIGN DESCRIPTION

The fuel assemblies are arranged in a roughly circular cross-sectional pattern. The assemblies are essentially identical in configuration, but contain fuel of different enrichments depending on the location of the assembly within the core.

The fuel is in the form of slightly enriched uranium dioxide ceramic pellets. The pellets are stacked to an active height of approximately 141 to 144 in. within ZIRLO™ tubular cladding, which is plugged and seal-welded at the ends to encapsulate the fuel. The typical enrichments of the fuel for the axial blanket and enriched regions are given in Table 4.2-2. The enrichments of the fuel for the various regions of the core are determined during the core design process for each reload to obtain the desired cycle energy. Heat generated by the fuel is removed by light water which flows upward through the fuel assemblies and acts as both moderator and coolant. Refueling takes place generally in accordance with a low leakage loading pattern with some fuel assemblies remaining in the core for up to four cycles.

The control rods, designated as rod cluster control assemblies, consist of groups of individual absorber rods which are held together by a spider at the top end and actuated as a group. In the inserted position, the absorber rods fit within hollow guide thimbles in the fuel assemblies. The guide thimbles are an integral part of the fuel assembly skeleton and occupy locations within the regular fuel rod pattern where fuel rods have been deleted. In the withdrawn position, the absorber rods are guided and supported laterally by guide tubes which form an integral part of the upper core support structure. Figure 4.2-1 shows a typical rod cluster control assembly.

The fuel assemblies are positioned and supported vertically in the core between the upper and lower core plates. The core plates are provided with pins which index into closely fitting mating holes in the fuel assembly top and bottom nozzles. The pins maintain the fuel assembly alignment which permits free movement of the control rods from the fuel assembly into the guide tubes in the upper support structure without binding or restriction between the rods and their guide surfaces.

Operational or seismic loads imposed on the fuel assemblies are transmitted through the core plates to the upper and lower support structures and ultimately to the internals support ledge at the pressure vessel flange in the case of vertical loads, or to the lower radial support and internals support ledge in the case of horizontal loads. The internals also provide a form-fitting baffle surrounding the fuel assemblies which confines the upward flow of coolant in the core area to the fuel-bearing region.

4.2.3 CORE COMPONENTS DESIGN DESCRIPTION

4.2.3.1 Fuel Assembly

The overall configuration of the fuel assemblies is shown in Figures 4.2-2 and 4.2-3. The assemblies are square in cross section, nominally 7.763 in. on a side, and have an overall height of approximately 160 in. The fuel rods in a fuel assembly are arranged in a square array with 14 rod locations per side and a nominal centerline-to-centerline pitch of 0.556 in. between rods. Of the total possible 196 rod locations per assembly, 16 are occupied by guide thimbles for the rod cluster control rods and one for in-core instrumentation. The remaining 179 locations contain fuel rods. In addition to fuel rods, a fuel assembly is composed of a top nozzle, a bottom nozzle, nine grid assemblies, 16 absorber rod guide thimbles, and one instrumentation thimble.

The guide thimbles in conjunction with the grid assemblies and the top and bottom nozzles comprise the basic structural fuel assembly skeleton. The top and bottom ends of the guide thimbles are fastened to the top and bottom nozzles respectively. The grid assemblies, in turn, are fastened to the guide thimbles at each location along the height of the fuel assembly at which lateral support for the fuel rods is required. Within this skeletal framework the fuel rods are contained and supported and the rod-to-rod centerline spacing is maintained along the assembly.

Defective fuel rods can be replaced with filler (dummy) rods fabricated from Zircaloy-4, ZIRLO™ or stainless steel to create a reconstituted fuel assembly. Reconstitution is accomplished by removing either the top or bottom nozzle, removing the defective rod(s), replacing the failed rod(s) with filler rod(s), and reattaching the nozzle. If a fuel assembly skeleton is damaged, serviceable fuel rods and dummy rods can be transferred to a new skeleton to form a reconstituted fuel assembly. The reconstituted fuel assemblies meet the same design requirements and satisfy the same design criteria as the original fuel assemblies.

4.2.3.1.1 Top Nozzle, Springs, and Clamps

The top nozzle, adapter plate, holddown springs, and clamps are shown in Figure 4.2-4. The perforated adapter plate directs the core flow through the nozzle enclosure into the upper internals. Two alignment holes are located in the top nozzle and mate with the pins in the upper core plate. The fuel assembly holddown provision consists of four sets of double-leaf springs that are clamped by screws at diagonally opposite corners of the top nozzle with the screws being secured in place by welded lock wires. During assembly a preload is normally applied to the springs, though it is possible to meet all required clearance tolerances with zero preload. The springs also prevent fuel-induced liftoff. The top nozzle, adapter plate, and clamp are stainless steel (type 304) whereas the springs are fabricated from Inconel.

4.2.3.1.2 Bottom Nozzle

The bottom nozzle is fabricated from type 304 stainless steel and consists of a perforated plate with four support legs (see Figure 4.2-5). The perforated plate directs the flow of the coolant upward toward the fuel rods. Indexing and positioning of the fuel assembly is controlled by alignment holes in two diagonally opposite legs which mate with two locating pins on the lower core plate.

Debris filter bottom nozzles (DFBN) were used commencing with the feed assemblies for cycle 21. The debris filter bottom nozzle is similar to the prior bottom nozzles except that it is designed to inhibit debris from entering the active fuel region of the core and minimize debris-related fuel failures. Composite debris filter bottom nozzles with reinforcing skirts have been used since the feed assemblies for cycle 29. The DFBN is a two-piece design incorporating a machined stainless steel adapter plate welded to a single low cobalt investment casting of the legs and skirts which enhances reliability during postulated adverse handling conditions while refueling. The composite DFBN is structurally and hydraulically equivalent to the previous DFBN design and meets all fuel assembly design criteria.

4.2.3.1.3 Guide Thimbles

The guide thimble tubes are fabricated from ZIRLO™ tubing. The guide tubes are structural members which also provide channels for core components, e.g., control rods, sources, burnable absorbers, and thimble plugs.

For Optimized Fuel Assemblies (OFA) as shown in Figure 4.2-6, there are two sections with a large diameter and two with a smaller diameter. The larger diameter at the top permits rapid insertion of the control rods during a reactor trip. The lower short expanded diameter accommodates the mechanical fastening of the second grid from the bottom. Both reduced-diameter sections produce a dashpot action near the end of the control rod travel during a reactor trip which decelerates the control rod and reduces the impact forces between the control rod spider hub and fuel assembly adapter plate at the end of travel. Orifice holes are provided in the tube wall to allow water to exit, thus controlling the rod drop time. The thimble tube has an end plug welded to the bottom end.

The newer 422V+ fuel assembly incorporates the tube-in-tube internal dashpot design which consists of a constant diameter outer guide thimble assembly with a separate smaller diameter dashpot assembly which is inserted into the outer guide thimble assembly. Both the guide thimble diameters and the dashpot inside diameter are the same as the OFA. The dashpot assembly is retained by a press fit with the guide thimble plug at the bottom and to the guide thimble tube with two restraint bulges just above the bottom grid. The tube-in-tube design operates in the same manner as the OFA fuel currently in service but eliminates the swaged reduced diameter portion of the guide thimble tube. This provides increased structural integrity and eliminates a potential incomplete rod insertion contributor.

4.2.3.1.4 Instrumentation Tube

The instrumentation tube is fabricated from ZIRLO™. It is a constant diameter tube as shown in Figure 4.2-7 and is designed to accept the in-core instrumentation. The instrumentation tube is supported at the various grid elevations in the same manner as the fuel rods and is supported and positioned at the lower end by the bottom nozzle.

4.2.3.1.5 Grid Assemblies

The fuel rods are supported at intervals along their length by grid assemblies which maintain the lateral spacing between the rods throughout the design life of the assembly. Each fuel rod is given support at six contact points within each grid cell by a combination of support dimples and springs. The grid assembly consists of individual interlocking slotted straps that are joined either by brazing or welding depending on the material. The straps provide support springs and dimples along with hydraulic flow mixing vanes.

The top and bottom grid material is Alloy 718, which was chosen because of its corrosion resistance, high strength properties, and resistance to irradiation-induced stress relaxation. The middle seven grids are fabricated from Zircaloy-4 or ZIRLO™ which was chosen because of its good neutron economy properties. The Inconel grids are furnace-brazed while the zircaloy grids are laser-welded. The magnitude of the grid restraining force on the fuel rod is set high enough to minimize potential fretting without overstressing the cladding at the

points of contact between the grids and fuel rods. The grid assemblies also allow axial thermal expansion and irradiation-induced growth of the fuel rods while imposing minimal axial restraining forces on the fuel rods.

The dimple contact area in the 422V+ ZIRLO™ mid-grids have been increased to provide significantly reduced contact stresses and to reduce wear rates on the cladding. A balanced vane pattern has also been introduced to eliminate a known mechanism for fuel assembly vibration. The OFA Zircaloy-4 mid-grid design remains unchanged.

The outside straps on all grids contain guide vanes and guide tabs which, in addition to their mixing function, aid in guiding the fuel assemblies past projecting surfaces during handling or loading and unloading of the core.

4.2.3.1.6 Fuel Rods

The fuel rods consist of a stack of uranium dioxide ceramic pellets contained in slightly cold worked ZIRLO™ tubing. The tubing is plugged and seal welded at the ends to encapsulate the fuel.

The top and bottom of the uranium stack (nominal 6.0 in.) contain the axial blanket pellets fabricated from slightly-enriched uranium. Commencing with Region 29 the blanket pellets are annular versus solid. In the center portion of the rod, the pellets are fabricated from slightly-enriched uranium. The tubing and end plug material is ZIRLO™. The function of the tube and end plugs is to contain the pellets and fission products. The bottom end plug has an internal pull grip feature for insertion and removal of the fuel rod to and from the assembly. Inside the top end of the fuel rod there is a stainless steel spring which is designed to prevent axial movement of the stack during shipment and handling of the fuel prior to irradiation.

4.2.3.1.7 Fuel Assembly Joints and Connections

All grids in the optimized fuel assembly design, with the exception of the bottom grid, are mechanically fastened to the thimble tubes by bulging into cylindrical sleeves which are attached to the grid straps. The sleeves used to attach the middle seven grids are fabricated from Zircaloy-4 and are welded to the grid straps. An illustration of a mid-grid joint is shown in Figure 4.2-8. The top grid sleeves are fabricated from stainless steel and is brazed to the Inconel grid straps. The top grid sleeve is then attached to the top nozzle adapter plate, as shown in Figure 4.2-9.

The bottom grid is fastened by clamping it between the thimble tube end plug and bottom nozzle via stainless steel inserts. The inserts are welded to the bottom grid straps. The bottom nozzle is fastened to the skeleton structure by means of stainless steel screws, which mate with internal threads in the thimble tube end plug. Reconstitutable design features have been incorporated into the bottom nozzle/thimble screw design.

As shown in Figure 4.2-9, the Ginna OFA/VANTAGE+ and 422V+ fuel assemblies are fitted with a top nozzle that is easily removable in the field to facilitate reconstruction of the defective fuel. It consists of a stainless steel nozzle insert, which is mechanically connected to the top nozzle adapter plate by means of a pre-formed circumferential bulge near the top of the

insert. The insert engages a mating groove in the wall of the adapter plate guide thimble through hole.

The insert has four equally spaced axial slots which will narrow to allow the insert to deflect inwardly at the location of the bulge, thus facilitating the installation or removal of the nozzle. The insert bulge is positively held in the adapter plate mating groove by placing a lock tube (same OD and ID as the guide thimble tube) into the insert. The lock tube is secured in place by local deformations that fit into the concave side of the insert's bulge and under a shoulder in the top nozzle adapter.

The bottom grid joint configuration is similar to the top grid design with stainless steel sleeves brazed in-place which allow the bottom grid to be mechanically attached to the guide thimble by two bulges located above the grid.

4.2.3.1.8 Fuel Assembly Identification

In addition to identifying the fuel assemblies in accordance with applicable NRC regulations, identification numbers are placed on assembly face three and top of the assembly. The numbers on face three are 2 in. in height and the numbers on the top are as large as practicable. The identification markings on the top of the fuel assembly are limited to a combined total of three digits, including numbers and letters.

4.2.3.2 Control Rods

The control rods or rod cluster control assemblies each consist of a group of individual absorber rods fastened at the top end to a common hub or spider assembly. These assemblies, one of which is shown in Figure 4.2-1, are provided to control the reactivity of the core under operating conditions.

The absorber material used in the control rods is silver-indium-cadmium alloy which is essentially "black" to thermal neutrons and has sufficient additional resonance absorption to significantly increase its worth. The alloy is in the form of extruded single-length rods which are sealed in stainless steel tubes to prevent the rods from coming in direct contact with the coolant.

The overall control rod length is such that when the assembly has been withdrawn through its full travel, the tips of the absorber rods remain engaged in the guide thimbles so that alignment between rods and thimbles is always maintained. Since the rods are long and slender, they are relatively free to conform to any small misalignments with the guide thimble.

The spider assembly is in the form of a center hub with radial vanes containing cylindrical fingers from which the absorber rods are suspended. Handling detents and detents for connection to the drive shaft are machined into the upper end of the hub. A spring pack is assembled into a skirt integral to the bottom of the hub to stop the rod cluster control assembly and absorb the impact energy at the end of a trip insertion. The radial vanes are joined to the hub, and the fingers are joined to the vanes by furnace brazing. A centerpost which will hold the spring pack and its retainer is threaded into the hub within the skirt and welded to prevent loosening in service. All components of the spider assembly are made from type 304 stainless steel except for the springs which are Alloy X-750 alloy and the retainer which is of 17-4

PH material. The absorber rods are secured to the spider so as to ensure trouble free service. The rods are first threaded into the spider fingers and then pinned to maintain joint tightness, after which the pins are welded in place. The end plug below the pin position is designed with a reduced section to permit flexing of the rods to correct for small operating or assembly misalignments.

In construction, the silver-indium-cadmium rods are inserted into cold-worked stainless steel tubing which is then sealed at the bottom and the top by welded end plugs. Sufficient diametral and end clearance are provided to accommodate relative thermal expansions and to limit the internal pressure to acceptable levels. The bottom plugs are made bullet-nosed to reduce the hydraulic drag during a reactor trip and to guide smoothly into the dashpot section of the fuel assembly guide thimbles. The upper plug is threaded for assembly to the spider and has a reduced end section to make the joint more flexible.

Stainless steel clad silver-indium-cadmium alloy absorber rods are resistant to radiation and thermal damage, thereby ensuring their effectiveness under all operating conditions. Ginna augmented the original rod cluster control assemblies with enhanced-performance rod cluster control assemblies. These assemblies have a thin chrome electroplate applied to a length of the stainless steel cladding in contact with the reactor internal guides to provide increased resistance to cladding wear. The enhanced-performance control rods also have reduced diameter absorber at the tips to allow more room within the clad for irradiation-induced swelling.

Withdrawal and insertion of the control rods into the core is accomplished by the control rod drive system described in Section 3.9.4.

4.2.3.3 Neutron Source Assemblies

The following is a historical discussion of neutron sources. Four neutron source assemblies were utilized initially in the core. These consisted of two assemblies with four secondary source rods each, and two assemblies with three secondary source rods and one primary source rod each. The rods in each assembly were fastened to a spider at the top end similar to the rod cluster control assembly spiders. The primary sources were discharged after the initial core. In subsequent cores, two or four secondary sources were used.

The secondary sources were removed at the cycle 20/21 refueling. The neutron emissions naturally occurring from the irradiated fuel provide a sufficient neutron source for startup.

4.2.3.4 Plugging Devices

The following is a historical discussion of thimble plugs. In order to limit bypass flow through the guide thimbles in fuel assemblies that do not contain either control rods or source assemblies, the fuel assemblies at those locations are fitted with plugging devices. The plugging devices consist of a flat spider plate with short rods suspended from the bottom surface and a spring pack assembly. At installation in the core, the plugging devices fit within the fuel assembly top nozzles and rest on the adapter plate. The short rods project into the upper ends of the thimble tubes to reduce the bypass flow area. The spring pack is compressed by the upper core when the upper internals package is lowered into place. Similar short rods are also used on the source assemblies to fill the ends of all vacant fuel assembly guide thimbles.

All components in the plugging device, except for the springs, are constructed from type 304 stainless steel. The springs (one per plugging device) are wound from an age-hardenable nickel base alloy to obtain higher strength.

All thimble plugs had been removed from the core by the start of cycle 23.

4.2.3.5 Fuel Pellet and Cladding Design Considerations

The consequences of a breach of cladding are greatly reduced by the ability of uranium dioxide to retain fission products including those which are gaseous or highly volatile. This retentiveness decreases with increasing temperature or fuel burnup, but remains a significant factor even at full power operating temperature in the maximum burnup element.

Perforation of fuel rod cladding which could release fission products or fuel material is directly related to cladding stress and strain under normal operating and overpower conditions. The stress limit during MODES 1 and 2 or conditions of moderate frequency is the 0.2% offset yield strength of the cladding. Cladding strain for MODES 1 and 2 is limited to 1% from the unirradiated condition. The transient clad strain limit is 1% from the pre-transient value.

For most of the fuel rod life the actual stresses and strains are considerably below the design limits so significant margins exist between actual operating conditions and the design limits.

Other parameters having an influence on cladding stress and strain and the design limits of these parameters are as follows:

A. Internal gas pressure. Rod internal gas pressure shall be limited to a value which would not cause (1) the fuel-clad diametral gap to increase due to outward cladding creep during steady-state operation and (2) extensive DNB propagation to occur.

B. Cladding temperature. The clad surface temperature (oxide to metal interface) shall not exceed that which is required to preclude a condition of accelerated clad oxidation.

C. Burnup. Fuel burnup affects both swelling of the fuel pellet and the release of fission gases during transient conditions. Also, the in-reactor time can affect the metallurgical properties of the cladding to varying degrees. Therefore, cladding stress and strain limits must be evaluated as a function of burnup to determine the limitations that exist.

D. Fuel temperature and kW/ft. During events of moderate frequency there shall be at least a 95% probability that fuel rods operating at the peak kW/ft will not exceed the uranium dioxide melting temperature. The melting temperature of unirradiated uranium dioxide is taken as 5080°F and decreases with burnup.

4.2.3.6 Reload Fuel Design

4.2.3.6.1 Reload Fuel Design - Westinghouse Optimized Fuel

Starting with cycle 14 Ginna Station began the transition to an all Westinghouse optimized fuel assembly fueled core. By cycle 21 the core consisted of all Westinghouse optimized fuel assemblies. Table 4.2-3 provides parameters associated with the optimized fuel assembly.

The 14 x 14 optimized fuel assembly is shown in Figure 4.2-3.

4.2.3.6.2 Reload Fuel Design - Westinghouse OFA/VANTAGE + Fuel

Starting with cycle 28, Ginna Station began the transition to an OFA fuel assembly with VANTAGE + fuel features. Table 4.2-3 provides parameters associated with the OFA/VANTAGE + fuel assembly.

4.2.3.6.3 Reload Fuel Design - Westinghouse 422V+ Fuel

Starting with cycle 33, Ginna Station began the transition to an all Westinghouse 422V+ fuel assembly core. Table 4.2-3 provides parameters associated with the 422V+ fuel assembly and the fuel assembly is shown in Figure 4.2-3.

4.2.3.7 Fuel Assembly and Rod Cluster Control Assembly Tests

To prove the mechanical adequacy of the original core fuel assembly and rod cluster control assembly, functional test programs were conducted on full scale San Onofre mock-up versions of the fuel assembly and control rods. (*Reference 1*).

4.2.3.7.1 Reactor Evaluation Center Tests

The prototype assemblies were tested under simulated reactor operating conditions (1900 psig, 575°F, 14 fps flow velocity) in the Westinghouse Reactor Evaluation Channel.

The components were subjected to a total environmental exposure of 4132 hours during which the rod cluster control assembly experienced a total travel of 38,927 linear feet. The travel was made up of 27,217 ft of normal driven travel and 11,710 ft of reactor trip travel, resulting from 1461 trips, which is equivalent to over two plant service lifetimes.

The fuel assembly remained in excellent mechanical condition. No measurable signs of wear on the fuel tubes or control rod guide tubes were found.

The control rod was also found to be in excellent condition, having maximum wear measured on absorber cladding of approximately 0.001 in.

4.2.3.7.2 Loading and Handling Tests

Tests simulating the loading of the prototype fuel assembly into a core location were also successfully conducted to determine that proper provisions had been made for guidance of the fuel assembly during MODE 6 (Refueling) operation.

The change to the short top nozzle 422V+ assembly design has necessitated changes to the manipulator crane gripper, the RCCA change fixture, the new fuel handling tool, the spent fuel handling tool, the portable RCCA change tool, and the RCCA stop on the fuel transfer cart. The new tooling is consistent with proven designs in service at the other plants.

4.2.3.7.3 Axial and Lateral Bending Tests

In addition, axial and lateral bending tests were performed in order to simulate mechanical loading of the assembly during MODE 6 (Refueling) operation.

Although the maximum column load expected to be experienced in service is approximately 1000 lb, the fuel assembly was successfully loaded to 2200 lb axially with no damage resulting. This information was also used in the design of fuel handling equipment to establish the limits for inadvertent axial loads during refueling.

4.2.4 DESIGN EVALUATION

4.2.4.1 Fuel and Cladding Evaluation - Original Core

The fission gas release and the associated buildup of internal gas pressure in the fuel rods was calculated by the FIGHT code based on experimentally determined rates. The increase of internal pressure in the fuel rod due to this phenomena was included in the determination of the maximum cladding stresses at the end of core life when the fission product gap inventory is a maximum.

The maximum allowable strain in the cladding, considering the combined effects of internal fission gas pressure, external coolant pressure, fuel pellet swelling, and clad creep was limited to less than 1% throughout core life. The associated stresses were below the yield strength of the material under all normal operating conditions.

To ensure that manufactured fuel rods met a high standard of excellence from the standpoint of functional requirements, many inspections and tests were performed both on the raw material and the finished product. These tests and inspections included chemical analysis, tensile testing of fuel tubes, dimensional inspection, X-ray of both end plug welds, ultrasonic testing, and helium leak tests.

In the event of cladding defects, the high resistance of uranium dioxide fuel pellets to attack by hot water protects against fuel deterioration or a decrease in fuel integrity. Thermal stress in the pellets, while causing some fracture of the bulk material during temperature cycling, does not result in pulverization or gross void formation in the fuel matrix. As shown by operating experience and extensive experimental work in the industry, the thermal design parameters conservatively account for any changes in the thermal performance of the fuel element due to pellet fracture.

4.2.4.2 Design Evaluation - Reload Optimized Fuel Assembly, OFA/VANTAGE+ Fuel Assembly, and 422 VANTAGE+ Fuel Assembly Designs

4.2.4.2.1 Introduction

The design and safety analysis of the optimized fuel assembly is discussed in WCAP-9500 (*Reference 2*) which the NRC has reviewed and found acceptable. However, the staff SER of WCAP-9500 requires that certain items be addressed on a plant specific basis. *Reference 3* includes the Ginna responses to staff questions related to plant specific items portions of which are discussed below. These sections have been selectively updated as required to reflect cores beyond cycle 14.

The design and safety analysis of the VANTAGE + fuel assembly is discussed in WCAP-12610-P-A (*Reference 20*) which the NRC has reviewed and found acceptable. However, the staff SER of WCAP-12610-P-A requires that certain plant specific analyses be completed

prior to the implementation of the VANTAGE + fuel product in a plant. *Reference 21* covers the plant specific analyses that are required to demonstrate acceptability of the VANTAGE + fuel at Ginna Station. *Reference 21* has been reviewed by the NRC staff and has been found to be acceptable.

4.2.4.2.2 Fuel Design

Table 4.2-3 presents fuel assembly, fuel rod, and fuel pellet design information for the OFA/ VANTAGE+ and 422V+ assemblies. Table 4.2-3 includes information on materials used and dimensions.

4.2.4.2.3 Design for Seismic and Loss-of-Coolant Accident Forces

Westinghouse has performed a structural integrity evaluation for the Ginna 422V+ 9-grid implementation. A homogeneous core of 422V+ and transition cores of OFA/VANTAGE+ and 422V+ fuel were analyzed for the combined seismic and LOCA loads and it was shown that the mid-grid impact forces for 422V+ and OFA are well below crush limits and a coolable core geometry is maintained. The stress analysis indicates that adequate margins for both fuel rods and thimble tubes for the 422V+ and OFA exist, so the fragmentation of the thimble tubes and fuel rods will not occur for combined seismic and LOCA loads.

Therefore, the 422V+ Ginna design is structurally adequate for Ginna seismic/LOCA loads in the homogeneous core and transition cores with OFA fuel. The OFA still satisfies all requirements in mixed core conditions during the 14x14 422V+ Ginna fuel transition.

4.2.4.2.4 Emergency Core Cooling System (ECCS) Calculation Loss-of-Coolant Accident Cladding Models

The WCOBRA/TRAC UPI Large Break evaluation model for Ginna includes NRC supplied loss-of-coolant accident cladding models as described in NUREG 0630, burst/blockage models. Additional information regarding the models are in *Reference 17*, *Reference 18*, *Reference 19*, *Reference 21*, and *Reference 20*.

4.2.4.2.5 Initial Fuel Conditions for Transient Analysis

The initial fuel temperatures used in the Ginna transient and accident analyses were calculated using the NRC approved Westinghouse fuel performance code, PAD-4.0 (*Reference 22*). In using PAD to generate fuel temperatures for input to safety analyses calculations, a conservative thermal safety model was used. Calculations of initial fuel stored energy used in safety analyses were also based on the results of conservative fuel average temperature calculations at the time of maximum densification. As a result, fuel temperatures at the end of one cycle are significantly less than those occurring at the time of maximum densification.

4.2.4.2.6 Predicted Clad Collapse Time

The Ginna evaluation was performed using *Reference 23*. Clad flattening or creep collapse depends on several design and physical events happening within a given time period. The first of these is fuel pellet hangup, then the formation of an axial gap in the fuel stack due to fuel densification below the hangup pellet, followed by cladding creep into the axial gap,

resulting in clad flattening. Fuel pellet hangup and cladding collapse are thought to be due to a combination of pellet cocking, fuel densification, and cladding creep. Therefore, those fuel design parameters that are important to clad flattening are fuel-to-cladding gap, initial fuel rod fill gas pressure, fuel densification, pellet cocking, and cladding creep rate.

Axial gaps greater than 0.5 inches can lead to cladding collapse and significant flux and power spiking. Axial gaps less than 0.5 inches have been shown to not result in cladding collapse. Current Westinghouse fuel data demonstrates that no large axial gaps (i.e. ≥ 0.3 inches) form in current generation fuel designs during in-reactor performance.

4.2.4.2.7 Nuclear Design

Nuclear design and analysis of Ginna cores are performed using the standard Westinghouse reload safety evaluation methodology. No changes in the nuclear design methodology or models were necessary due to the transition to OFA/ VANTAGE+ or 422V+ fuel assemblies. The most important nuclear design parameter change is the positive moderator temperature coefficient, for which the maximum value of +5.0 pcm/°F is expected to occur at the beginning-of-cycle condition. In particular, conservatively positive values of the moderator temperature coefficient were assumed in the accident evaluations. In general, the neutronic parameters used as input to the safety evaluation were chosen to bound the values obtained from the transition cycles. The required shutdown margin was computed using the negative temperature coefficient corresponding to the end-of-cycle condition and assuming all but the most reactive rod has inserted into the core. The required value of the shutdown margin was found to be 1.3% Δp. CENG will perform whole core power distribution measurements at startup (in addition to administrative procedures) to ensure against fuel misloading. Likewise, CENG will ensure that future cycles comply with the calculated values and bounds of this analysis. Table 4.2-4 is a listing of the neutronic parameters used in the safety analysis to provide bounding values against which cycle dependent parameters may be compared.

4.2.4.2.8 Fuel Assembly Hydraulic Lift-Off

From the precision flow calorimetric in cycle 13, the value for the reactor system flow obtained was approximately 195,000 gpm. The hold-down springs of the optimized fuel assembly and the VANTAGE + fuel assembly are designed to withstand lift-off of the assembly up to a flow rate of 100,000 gpm/loop or 200,000 gpm system flow and should therefore resist lift-off. Additional conservatism has also been built into the analysis to account for uncertainties in thermal and hydraulic parameters, fuel assembly hydraulic resistance, and worst case inlet flow maldistribution factors. The spring rate for the Ginna box nozzle which was implemented in cycle 27 (region 29) is lower than the previous nozzle design. This tends to make the fuel assembly less susceptible to guide thimble bow and distortion. The maximum and minimum contact force requirements are still met with this design.

A top nozzle holddown spring force analysis demonstrated that the functional requirement for fuel assembly holddown is met for both the transition core fuel assembly designs (OFA/VANTAGE+ and 422V+) as well as for a full-core application of 422V+ under extended power uprate conditions.

4.2.4.2.9 Thermal-Hydraulic Analysis

The thermal-hydraulic analysis of the 422V+ and OFA/VANTAGE+ mixed core was performed using the Revised Thermal Design Procedures (RTDP) (*Reference 6*) and the VIPRE code (*References 7*. The WRB-1 (*Reference 9*) critical heat flux correlations was used for both fuel assemblies. The RTDP and the VIPRE code used with the critical heat flux correlation have previously been approved by the NRC. Additional components of this application are noted below.

4.2.4.2.9.1 Sensitivity Factors

For Ginna, the VIPRE code and the WRB-1 DNB correlation have been used for the calculation of sensitivity factors for both the OFA/VANTAGE+ and 422V+ fuel. All parameter values are within the ranges of the codes and correlations used, and sensitivity factors have been determined specific to the fuel type over the range of Ginna parameters. Note that the parameter uncertainties used in the calculations conservatively bound actual Ginna parameters.

4.2.4.2.9.2 WRB-1 Correlation

WRB-1 correlation was approved for the 17 x 17 optimized fuel assemblies and 17 x 17 and 15 x 15 standard LOPAR fuel assemblies with a DNBR limit of 1.17 for the R-grid.

Ginna provided information to the NRC to justify the use of the WRB-1 critical heat flux correlation for the nine-grid 14 x 14 optimized fuel assemblies. The 14 x 14 optimized fuel assembly DNB test results were provided to the NRC in *Reference 12* which contains Supplement 1 to WCAP-8762 (*Reference 13*). These test results were used to demonstrate that the WRB-1 critical heat flux correlation correctly accounted for the geometry changes from the 0.422-in. R-grid design to the 14 x 14 optimized fuel assembly design. The DNB safety analyses for Ginna have been performed with the grid spacing term in the WRB-1 correlation set equal to 22 in., the longest grid spacing in the assembly. The WRB-1 correlation has been shown to accurately predict the 0.422 R-grid critical heat flux performance with grid spacings of 13 to 32 in. (*Reference 12*). The WRB-1 correlation is applicable to the Ginna 14 x 14 optimized fuel assembly and the 14 x 14 VANTAGE + fuel assembly fuel since the range of data covers the spacing for the nine-grid design for Ginna.

Based on the comparison to the FCEP parameters of the WRB-1 database of licensed fuel assembly designs, the Ginna 422V+ fuel assembly design was concluded to be licensable to the WRB-1 critical heat flux correlation (*Reference 24*). The geometric and fluid parameters that affect the applicability of the CHF correlation for the 422V+ nine-grid fuel assembly are bounded by, or have been shown by engineering evaluation to be "similar to or bracketed by" the ranges of the licensable database for the WRB-1 CHF correlation. Therefore the WRB-1 correlation with 1.17 limit DNBR and the associated ranges is acceptable for the 422V+ nine-grid fuel assembly design. Use of the WRB-1 correlation in DNB analyses for 422V+ is discussed in Section 4.4.3.1.

4.2.4.2.9.3 Rod Bow Penalties

Rod bow can occur between mid-grids, reducing the spacing between adjacent fuel rods and reducing the margin to DNB. Rod bow must be accounted for in the DNBR safety analysis of

Condition I and Condition II events. Westinghouse has conducted tests to determine the impact of rod bow on DNB performance; the testing and subsequent analyses were documented in *Reference 14*.

Currently, the maximum rod bow penalty for the OFA fuel assembly is 1.0% DNBR at an assembly average burnup of 24,000 MWD/MTU (*Reference 14* and 25). No additional rod bow penalty is required for burnups greater than 24,000 MWD/MTU since credit is taken for the effect of $F^{N}_{\Delta H}$ burndown due to the decrease in fissionable isotopes and the buildup of fission products (Reference 26). Based on the testing and analyses of various fuel array designs documented in *Reference 14*, including the 14x14 STANDARD assembly, the 14x14 OFA and the 14x14 422V+ fuel assemblies should have the same rod bow penalty applied to the analysis basis as that used for 14x14 STANDARD fuel assemblies.

For the OFA/VANTAGE+ and 422V+ fuel assemblies, sufficient margin (7.5% and 10.1% respectively) exists between the safety analysis limit DNBR and the design limit DNBR to accommodate this penalty as shown below.

	Westinghouse 14 x 14 OFA/ VANTAGE+ and 422V+Fuel Assembly	
	OFA/VANTAGE+	422V+
Correlation	WRB-1	WRB-1
Correlation limit DNBR (STDP)	1.17	1.17
Design limit DNBR (RTDP)	1.24	1.24
Safety analysis limit DNBR (RTDP)	1.34	1.38
DNBR Margin (RTDP)	7.5%	<10.1%

The DNBR margin is defined as:

Safety analysis DNBR value = Design DNBR value / (1 - Margin)

4.2.4.2.9.4 DNBR Design Limits

The core thermal-hydraulic analysis was performed using 1775 MWt core power, 2250 psia system pressure, a nominal T_{AVG} of 576°F, and 177,300 gpm primary system minimum measured flow. Use of a nominal T_{AVG} of 576°F bounds operation at lower nominal values of T_{AVG}. The DNBR design limits using RTDP are shown in the table above and are valid for both typical and thimble cells. For the OFA/VANTAGE+ 422V+ fuel assemblies the WRB-1 correlation was used with a design DNBR limit of 1.24 (RTDP). The safety analysis limit (SAL) DNBR calculated was 7.5% and 10.1% above the associated design limit for OFA/ VANTAGE+ and 422V+ fuel respectively. This margin is more than enough to account for the rod bow penalty, steam generator tube plugging, and thimble plug removal.

4.2.4.3 Design Evaluation of Reconstituted Fuel Assemblies

Filler rods were originally used in fuel assemblies to replace those fuel rods damaged by the baffle jetting problem in Westinghouse reactors. This concept was extended further to replace rods during reconstitution of fuel assemblies in other locations. In order to satisfy generic fuel design criteria, the dummy rods, which are now required to be solid filler rods, require thermal-hydraulic analyses to demonstrate that inclusion of these rods in a specific fuel cycle is acceptable with respect to the overall fuel performance and safety-significant conclusions. Such an analysis will follow the methodology described in *Reference 15*. Should more than 30 rods in the core, or 10 rods in any assembly, be replaced per refueling, a report describing the number of rods replaced and associated cycle-specific evaluation shall be submitted to the Nuclear Regulatory Commission prior to criticality.

4.2.5 CORE COMPONENTS TESTS AND INSPECTIONS

Fuel assemblies are manufactured and inspected in accordance with the Vendor’s Quality Assurance Program.

Since cycle 14 was the first substantial application of 14 x 14 nine-grid Westinghouse optimized fuel assembly fuel (excluding lead test assemblies), a visual surveillance was performed. This was conducted in the containment area for a reasonable number of optimized fuel assemblies until they completed their fuel cycles and were put into the spent fuel pool (SFP).

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Table 4.2-1
NUCLEAR DESIGN DATA

STRUCTURAL CHARACTERISTICS[a]

1.	Fuel weight (UO_2), lb[b]	117,682-120,481
2.	ZIRLO™ weight, lb	28,427
3.	Core diameter, in.	101.44
4.	Core height, in.[c]	143.25
	Reflector Thickness and Composition	
5.	Side-water plus steel (not counting baffle)	
	Corner	25.25 in.
	Flat	15.22 in.
6.	Number of fuel assemblies	121
7.	UO_2 rods per assembly	179

PERFORMANCE CHARACTERISTICS

8.	Heat output, MWt (as initially licensed)	1300
9.	Heat output, MWt (Previous Reactor Power Rating)	1520
10.	Heat output, MWt (current reactor and maximum calculated turbine rating)	1775
11.	Typical fuel burnup at BOL, MWd/MTU	20,400
12.	Typical average enrichment, wt % [a]	4.80
13.	Heat flux hot-channel factor (max), F_Q	2.60
14.	Nuclear enthalpy rise hot-channel factor (max), $F^N_{\Delta H}$	1 .72 (422V + Ginna fuel 1.60 (OFA/VANTAGE+ fuel)

CONTROL CHARACTERISTICS

Rod Cluster Control Assemblies

15.	Material	5% cadmium; 15% indium; 80% silver
16.	Full-length, rod cluster control assemblies, number	29
17.	Number of absorber rods per rod cluster control assembly	16

a. For full core of Westinghouse 422V+ Ginna Fuel.

b. Fuel weight may vary depending on number of fuel assemblies with annular pellets in the blanket region and/or fabrication tolerances.

c. For full core of Westinghouse 422V+ Ginna fuel. The core height for full core of Westinghouse OFA/ VANTAGE+ is 414.4 inches.

Table 4.2-2
CORE MECHANICAL DESIGN PARAMETERS*

	OFA/VANTAGE+	422V+
ACTIVE PORTION OF THE CORE		
Equivalent diameter, in	101.44	
Active fuel height, in.	141.4	143.25
Length-to-diameter ratio	1.46	
Total cross section area, ft^2	50.6	
FUEL ASSEMBLIES		
Number	121	121
Rod array	14 x 14	14x14
Rods per assembly[a]	179	179
Rod pitch, in	0.556	0.556
Fuel weight (as UO_2), lb	103,748 - 105,996[b]	117,682-120,481
Number of grids per assembly	9	9
Number of guide thimbles	16	16
Diameter of guide thimbles (upper part), in	0.492 I.D. x 0.526 O.D.	0.492 I.D. x 0.526 O.D.
Diameter of guide thimbles (lower part), in	0.4465 I.D. x 0.4815 O.D.	0.492 I.D. x 0.526 O.D.
Diameter of dashpot	NA	0.4465 I.D. x 0.480 O.D.
FUEL RODS		
Number	21,659	21,659
Outside diameter, in.	0.400	0.422
Diametral gap, in	0.0070	0.0075
Clad thickness, in.	0.0243	0.0243
Clad material	Zircaloy-4/ZIRLO™	ZIRLO™
Overall length, in	149.162	152.763

	OFA/VANTAGE+	422V+
FUEL PELLETS		
Material	UO_2 sintered	UO_2 sintered
Density (% of theoretical)	95	95
Fuel enrichments wt %, (typical)		
Axial blanket region	Natural or slightly enriched uranium, solid or annular	Slightly enriched uranium, annular
Enriched region	2.60, 3.30, 3.40, 3.60, 3.80, 3.90, 4.00, 4.2, 4.3, 4.6, 4.8	2.60, 3.30, 3.40, 3.60, 3.80, 3.90, 4.00, 4.2, 4.6, 4.8, 4.95
Diameter, in.	0.3444	0.3659
Length, in.		
Axial blanket region (natural uranium)	0.500	NA
Axial blanket region (slightly enriched)	0.500	NA
Enriched region	0.413	0.439
Annular blanket region (natural or slightly enriched)	0.500	0.500
ROD CLUSTER CONTROL ASSEMBLIES		
Neutron absorber	5% cadmium, 15% indium, 80% silver	5% cadmium, 15% indium, 80% silver
Cladding material	Type 304 SS - cold worked	Type 304 SS - cold worked
Clad thickness, in.	0.019	0.019
Number of clusters, full length	29	29
Number of control rods per cluster	16	16
Length of rod control	156.639 in. overall	156.639 in. overall
	148.759 in. insertion length[c]	148.759 in. insertion length
Length of absorber section	142.01 in. (full length)	142.01 in. (full length)

	OFA/VANTAGE+	422V+
CORE STRUCTURE		
Core barrel, in.		
I.D.	109.0	109.0
O.D.	112.5	112.5
Thermal shield, in.		
I.D.	115.3	115.3
O.D.	121.8	121.8

* All dimensions are for Westinghouse Optimized, VANTAGE + and 422V+ fuel assembly cold conditions.

a. Sixteen positions are occupied by guide thimbles to provide passage for control rods and one position contains an instrument thimble for in-core instrumentation.

b. Core fuel weight may vary depending on number of assemblies with annular blankets and/or fabrication tolerances.

c. From top of adaptor plate to bottom of end plug

Table 4.2-3
FUEL DESIGN

Basis	OFA/VANTAGE+	422V+
Fuel assemblies		
Number of fuel assemblies	121	121
UO_2 rods per assembly	179	179
Rod pitch, in.	0.556	0.556
Assembly pitch	7.803	7.803
Number of grids per assembly	9	9
Material	7- Zircaloy 2- Inconel	7- ZIRLO™ 2- Inconel
Guide tube material	Zircaloy/ZIRLO™	ZIRLO™
Fuel rods		
Number	21,659	21,659
Clad O.D., in	0.400	0.422
Diametral gap, in.	0.0070	0.0075
Clad thickness, in.	0.0243	0.0243
Clad material	Zircaloy/ZIRLO™	ZIRLO™
Fuel pellets		
Material	UO_2	UO_2
Density, % theoretical	95	95
Diameter, in.	0.3444	0.3659
Length		
Axial blanket region (natural uranium)	0.500	NA
Axial blanket region (slightly enriched)	0.413	NA
Enriched region	0.313	0.439
Annular blanket region (natural or slightly enriched)	0.500	0.500

Table 4.2-4
KINETIC PARAMETERS USED IN TRANSIENT ANALYSIS (WESTINGHOUSE OFA/ VANTAGE+ AND 422V+ GINNA FUEL ASSEMBLY 14 x 14 FUEL)

Parameter	Bounding Value
Most positive moderator temperature coefficient, pcm/°F	≤+5.0 (for power < 70%) ≤0.0 (for power ≥ 70%)
Most positive moderator density coefficient, Δk/gm/cc	≤0.45
Doppler temperature coefficient, pcm/°F fuel	-0.91 to -2.90
Zero Power Doppler - only power coefficient, pcm/% power	-12.0 + 0.045Q to -24.0 + 0.100Q
B_{EFF} (fraction)	0.0043 to 0.0072
Normal operation $F_{\Delta H}$ (with uncertainties)	≤ 1.72 (422V+ Ginna) ≤ 1.60 (VANTAGE+)
Maximum total peaking factor, F_Q	≤ 2.60

4.3 RELOAD CORE NUCLEAR DESIGN

This section describes the nuclear design and evaluation of reload cores. The design bases for the nuclear design of the fuel and reactivity control system are described in Section 4.2.1. The design objectives and bases are reviewed and each of the design and evaluation phases of a reload core is discussed. The capability of the reactor to achieve these objectives while performing safely under operational modes, including both transient and steady-state, is demonstrated in this section. Relevant design procedures and methods are briefly described and design codes are referenced where appropriate.

The objective of the nuclear design process is to determine the number and enrichment of the feed assemblies and a preliminary loading pattern that meets the required energy output of the refueled core as defined in the design initialization. Constraints from the design initialization specify the approximate MODE 6 (Refueling) dates, the burnup window of the previous cycle, and sometimes an upper and/or lower bound on the number of feed assemblies (or alternatively on the feed enrichment).

Once the loading pattern is set, the nuclear evaluation phase begins. The primary objective of this phase is to determine whether all nuclear-related key safety parameters are within the bounding values used in the reference analysis. These parameters are used in the safety evaluation.

4.3.1 PRELIMINARY DESIGN PHASE

The detail and scope of the preliminary design process depends to a large extent on how similar the refueled core is to previous reload cores. When it differs significantly from previous reloads, detailed calculations are used, as outlined later in this section. When the reload is very similar to ones already designed, simpler calculational models may be used. These simpler calculational models are benchmarked to the more detailed models.

When a preliminary loading pattern that meets the required energy output is established, an evaluation is performed to ensure that the following criteria are satisfied.

1. The $F_{\Delta H}$ values with all-rods-out and D-bank-in to the insertion limit are below specified limits, with allowance for variation in the actual burnup of the previous cycle.
2. The moderator temperature coefficient satisfies Technical Specification requirements.
3. Sufficient rod worth is available to meet the N-1 rods shutdown margin criteria at all times.

During the preliminary design phase, operating history is used as much as possible and where this is not available, the best prediction of the operating history is used. Some of the parameters that comprise the operating history are power level, control rod position, average coolant temperature, and other parameters that may affect the nuclear models. Operating history is used to ensure that the nuclear model of the core represents the actual condition of the core.

With the completion of the preliminary design phase, the preliminary loading pattern including the number and enrichment of feed assemblies and the number of burnable absorber rods

if any, is fixed. Also, the three criteria specified above are met. The remaining effort consists of determining the nuclear related key safety parameters.

4.3.2 DETERMINATION OF NUCLEAR-RELATED KEY SAFETY PARAMETERS

A reload core can affect nuclear-related key safety parameters in three basic areas: core kinetic characteristics, control rod worths, and core power distributions. Key safety parameters can be determined by a comparison of the current reload core characteristics with the characteristics of previously analyzed reload cores, scoping studies that typically utilize efficient spatially dependent nuclear calculations, or explicit calculations using detailed techniques and models.

Each of the above methods is used in varying degrees for any particular reload evaluation. For example, if a reload core is identical to a previous reload (where plant operating parameters, fuel enrichment, cycle burnup, fuel arrangement, control rod pattern, etc., remain the same), a simple comparison would demonstrate that the previously evaluated parameters are applicable and that additional calculations are not required. This example, of course, is an ideal situation. Conversely, a reload core may possess characteristics unlike any previously evaluated core. For this example, comprehensive scoping calculations and explicit worst-case condition calculations would be required to evaluate limiting safety analysis parameters.

Most reload cores cannot be categorized by the above two examples. That is, reload cores possess varying degrees of similarity with previously evaluated reload cores and the evaluation methods recognize this fact.

The following discussion describes the methods for determining the nuclear related key safety parameters for the reload core. Three areas are addressed: control rod worth parameters, core reactivity parameters and coefficients, and other nuclear-related key safety parameters for specific events. Nuclear-related key safety parameters are identified and, where appropriate, a description of core conditions that are assumed in the evaluation of these parameters is discussed.

4.3.2.1 Reactivity Control Aspects

Reactivity control is provided by (1) a soluble chemical neutron absorber in the reactor coolant (boric acid, also called chemical shim), and (2) movable neutron absorbing control rods.

The concentration of boric acid is varied as necessary during the life of the core to compensate for (1) changes in reactivity which occur with change in temperature of the reactor coolant from MODE 5 (Cold Shutdown) to the hot operating, zero power conditions, (2) changes in reactivity associated with changes in the fission product poisons, xenon and samarium, (3) reactivity losses associated with the depletion of fissile inventory and buildup of long-lived fission product poisons (other than xenon and samarium), and (4) changes in reactivity due to burnable absorber burnout.

The control rods provide reactivity control for (1) fast shutdown, (2) reactivity changes associated with changes in the average coolant temperature above hot zero power (core average coolant temperature is increased with power level), (3) reactivity associated with any void formation, and (4) reactivity changes associated with the power coefficient of reactivity.

The control rods are divided into two categories according to their function. The rods which compensate for changes in reactivity due to variations in operating conditions of the reactor, such as coolant temperature, power level, boron concentration, or xenon concentration, comprise the control group of rods. The other rods provide additional shutdown reactivity and are termed shutdown rods. The total shutdown worth of all the control rods is specified to provide adequate shutdown at all operating and hot zero-power conditions with the most reactive rod stuck out of the core. The distribution of the various control group rods and shutdown rods within the core is shown in Figure 4.3-1.

A reload core can typically alter individual rod cluster control assembly worths and control and shutdown bank worths. These changes can be attributed to changes in the neutron flux distribution (and thus, reactivity importance) that are produced by the loading pattern of burned and fresh fuel assemblies and the fuel depletion which occurs during the reload fuel cycle. Changes in control rod worths may also affect rod insertion limits, trip reactivity, differential rod worths, and shutdown rod worth.

Prior to the evaluation of limiting control rod worth parameters, an initial evaluation of limiting control rod worth parameters is performed by rod worth calculations obtained using two-group three-dimensional models. These calculations are performed for the beginning and end of the reload fuel cycle at full and zero power conditions. The total worth of all the shutdown banks is also calculated at zero power conditions. In addition, the impact of the previous cycle burnup (burnup window) on the rod worth calculation is also evaluated for completeness. These calculations form the basis for the evaluation of the limiting control rod worth parameters.

4.3.2.1.1 Insertion Limits

Control rod insertion limits define the deepest individual control bank insertion that can be allowed, as a function of the reactor power level. One of the purposes for these limits is to physically restrict the value of the inserted integral rod worth in the core at any power level. This will ensure that the minimum shutdown margin requirement can be satisfied regardless of the core configuration during MODES 1 and 2. It should be recognized, however, that control rod insertion limits are not defined by reactivity constraints alone. The final determination of control rod insertion limits is dependent on peaking factor constraints that must be satisfied during MODES 1 and 2 and during certain accident conditions.

Insertion limits are calculated using two-group, one-dimensional axial models. The core is depleted using a three-dimensional model. The three-dimensional model is collapsed into an equivalent one-dimensional axial model. The calculations are performed at the beginning and end of the reload cycle. Subsequently, the axial model is used to compute power levels for various rod positions (with normal bank overlap) that would represent a pre-defined value of inserted integral rod worth (commonly referred to as the rod insertion allowance). Rod insertion limits are conservatively constructed by limiting the amount of rod insertion at any power level to a value that is less than the calculated amount.

4.3.2.1.2 Total Rod Worth

The total integral rod worth is evaluated by assuming that all the control and shutdown banks are inserted and that the most reactive individual rod cluster control assembly is fully withdrawn from the core. Calculations are performed at the beginning and end of the reload fuel cycle at hot zero power conditions. Two-group three-dimensional calculations are used to determine the worth of the most reactive stuck rod. Individual rods are withdrawn from an all-rods-in condition until the most reactive rod is identified. The stuck rod worth is subtracted from the total worth of all control and shutdown banks and the resultant quantity (called the N-1 rod worth) is further reduced for conservatism. This evaluation of the minimum N-1 rod worth is used to determine the shutdown margin that is available at both the beginning and end of the reload fuel cycle.

4.3.2.1.3 Trip Reactivity

The minimum trip reactivity at or near full power conditions and the trip reactivity shape (i.e., the inserted rod worth versus rod position) are control rod worth parameters evaluated for each reload core. The minimum trip reactivity is evaluated at the beginning and end of the reload fuel cycle to ensure that the previously established limit is valid for power levels near full power and for the entire cycle length.

The most limiting trip reactivity shape (accounting for the worst axial power distribution) is evaluated each reload fuel cycle to determine the minimum inserted rod worth versus rod position that would be produced by N-1 control rods entering the core at full power. This evaluation is performed with two-group one-dimensional axial calculations. The axial model is established by collapsing the three-dimensional model into an equivalent one-dimensional axial model. It is assumed that the control rods can be inserted as deep as the full power insertion limit and that the power distribution is within Technical Specifications limits. Using the most limiting axial power shape, a single shutdown bank, equal in worth to the minimum trip reactivity, is inserted into the core in a stepwise fashion. The results of these calculations are used to evaluate the minimum inserted rod worth versus rod position.

4.3.2.1.4 Differential Rod Worths

Maximum differential rod worths at full power and zero power conditions are evaluated for each reload core. These evaluations are performed at the beginning and end of the fuel cycle. Two-group, one-dimensional axial calculations are used to determine maximum differential rod worths.

The differential rod worths are obtained using the equivalent axial model, which has been obtained by collapsing the three-dimensional model with control bank cross sections that yield the total worth determined by the three-dimensional analyses for the bank fully inserted. Full power calculations are performed to determine the maximum differential worth of any control bank that could be moving during power operation. The control banks are assumed to move in normal sequence with programmed control bank overlap. At zero power conditions, the maximum differential rod worth of any two sequential control banks is determined by assuming that the banks are moving with 100% overlap. That is, both control banks are withdrawn simultaneously as in a postulated startup accident from a subcritical condition.

4.3.2.1.5 Summary

The control rod worth parameters are evaluated each reload fuel cycle. These key safety parameters are then factored into the reload safety evaluation.

4.3.2.2 Core Reactivity Parameters and Coefficients

The kinetic characteristics of the reactor core determine the response of the core to changing plant conditions or to operator adjustments made during MODES 1 and 2, as well as the core response during abnormal or accidental transients. These kinetic characteristics are quantified in terms of reactivity coefficients. The reactivity coefficients reflect the changes in the neutron multiplication due to varying plant conditions such as changes in power, moderator, or fuel temperatures. Since reactivity coefficients change during the life of the core, ranges of coefficients are employed in transient analysis to determine the response of the plant throughout life.

Reactivity coefficients are calculated on a core-wide basis using three-dimensional two-group calculations. For some accidents, power distributions during the transient do not change significantly from those occurring during normal operating conditions, ensuring negligible changes in the values of reactivity coefficients. However, for accidents leading to significant power distribution changes from those occurring during normal operating conditions (e.g., worst stuck rod configuration), reactivity coefficients are determined using the power distribution occurring during the accident. The exact values of the reactivity coefficient used in the safety analysis depend on whether the transient of interest is examined at beginning-of-life or end-of-life, whether the most negative or the most positive (least negative) coefficients produce conservative results, and whether spatial non-uniformity must be considered in the analysis. Conservative values of reactivity coefficients, considering various aspects of analysis, are used in the transient analysis. Table 4.2-4 illustrates the reactivity parameters and coefficients and the limiting values which are evaluated for each reload core.

Reactivity parameters and coefficients are evaluated by considering the following conditions.

A. Beginning, middle, and end of the reload fuel cycle.

B. Full power, part power, and zero power operation.

C. Rodded core configurations allowed by the Technical Specifications during power operation.

In addition to the above conditions, consideration is also given to the impact that the previous cycle burnup has on core reactivity parameters and coefficients. The evaluated reactivity parameters and coefficients are discussed below.

4.3.2.2.1 Moderator Temperature Coefficient

The moderator temperature (density) coefficient is defined as the change in reactivity per degree change in moderator temperature (density). The value of this coefficient is sensitive to changes in the moderator density, the moderator temperature (keeping the density constant), the soluble boron concentration, the fuel burnup, and the presence of control rods and/or burnable absorbers which reduce the required soluble boron concentration and increase the

"leakage" of the core. The moderator coefficient is calculated for the various plant conditions discussed above by performing two-group three-dimensional neutronic calculations, varying the moderator temperature (and density) by several degrees about each of the mean temperatures of interest.

4.3.2.2.2 Fuel Temperature Coefficient

The fuel temperature (doppler) coefficient is defined as the change in reactivity per degree change in effective fuel temperature. It is primarily a measure of the doppler broadening of Uranium-238 and Plutonium-240 resonance absorption peaks. The fuel temperature coefficient is calculated by performing two-group three-dimensional calculations. Moderator temperature is held constant and power level is varied. The spatial variation of fuel temperature is taken into account by calculating the effective fuel temperature as a function of local power density throughout the core. The doppler only contribution to the power coefficient is derived from the same calculations and is defined as the change in reactivity per percent change in power.

4.3.2.2.3 Boron Worth

The boron worth is defined as the change in reactivity per ppm change in the boron concentration. The value of this parameter depends on the boron concentration, on the moderator temperature (density), and on the presence of control rods and/or burnable absorbers. It is calculated for the various plant conditions discussed above by performing two-group neutronic calculations, varying the boron concentration about the reference values of interest.

4.3.2.2.4 Delayed Neutrons

Delayed neutrons play an important role in determining the dynamic response of the core. The delayed neutrons are emitted from fission products, called precursors, a short time after a fission event. The delayed neutron fraction in each of the precursor groups is, in general, different for different fissionable isotopes. The effective delayed neutron fraction for the entire core is obtained by weighting the delayed neutron fraction for different isotopes and precursor groups by the region-wise fraction of fissions in each isotope and the region-wise power sharing in the core. Region-wise power sharings for various core conditions described earlier are obtained from two-group three-dimensional neutronic calculations. The fraction of fissions in each isotope is obtained from region-wise macroscopic few-group cross-section calculations.

4.3.2.2.5 Prompt Neutron Lifetime

The prompt neutron lifetime value is obtained in a manner similar to the calculation of the effective delayed neutron fraction. Values of the prompt neutron lifetime are obtained from region-wise few-group cross-section calculations. These values are weighted by region-wise power sharings taken from two-group three-dimensional neutronic calculations for various core conditions to determine the core average prompt neutron lifetime.

4.3.2.2.6 Summary

Core reactivity parameters and coefficients evaluated in reload cores depend on the previous cycle burnup, the number and enrichment of fresh fuel assemblies, the loading pattern of burned and fresh fuel, the number and location of any burnable absorbers, etc. These coefficients and parameters do, however, exhibit predictable trends which are dependent on such core average parameters as burnup, boron concentration, moderator and fuel temperatures, and power level. As a result of these trends and past reload evaluation experience, reactivity parameters and coefficients can be evaluated using differing degrees of sophistication.

4.3.2.3 Reactor Core Power Distribution

In order to meet the performance objectives without violating safety limits, the peak to average power density must be within the limits set by the nuclear hot-channel factors. For the peak power point in the core, the heat flux hot-channel factor, F_Q, was established as specified in Table 4.2-1. For the hottest channel the nuclear enthalpy rise hot-channel factors, $F^N_{\Delta H}$, was established as specified in Table 4.2-1.

Power capability of a PWR core is determined largely by consideration of the power distribution and its interrelationship to limiting conditions involving

- The linear power density.
- The fuel cladding integrity.
- The enthalpy rise of the coolant.

To determine the core power capability, local as well as gross core neutron flux distributions have been determined for various operating conditions at different times in core life.

The presence of control rods, burnable absorbers, and chemical shim concentration all play significant roles in establishing the fission power distribution, in addition to the influence of thermal-hydraulic and temperature feedback considerations. The computer programs used to determine neutron flux distributions include a model to simulate nonuniform water (and chemical shim) density distributions.

Thermal-hydraulic feedback considerations are especially important late in cycle life where the magnitude of the flux redistribution and reactivity change with change in core power or rod movement are strongly influenced by enthalpy rise up the core and by the fuel burnup distribution. Consequently, extensive X-Y and Z power distribution analyses have been performed to evaluate fission power distributions. In-core instrumentation is employed to evaluate the core power distributions throughout core lifetime to ensure that the thermal design criteria are met.

4.3.3 EVALUATION OF RELOADS WITH OFA/VANTAGE+ AND 422V+ FUEL ASSEMBLIES

The key safety parameters evaluated for the transition to 422V+ fuel at extended power uprate conditions show that the expected ranges of variation for many of the parameters will lie within the normal cycle-to-cycle variations. The parameters which fall outside of these

ranges are those which are sensitive to fuel type, e.g., the moderator temperature coefficient. The accident evaluations, documented in Chapter 15, have considered ranges of parameters which are appropriate for the transition cycles and beyond.

The Advanced Nodal Code (ANC) (*Reference 1*) was implemented in the reload design analysis beginning with cycle 19. ANC is an advanced nodal analysis theory code capable of two- or three-dimensional calculations. Beginning with cycle 22, PHOENIX-P (*Reference 2*) computer code was implemented in the reload design analysis. PHOENIX-P is a two-dimensional transport theory based code that calculates lattice physics constants. These models supplement the "Standard Reload Safety Evaluation Methodology" (*Reference 3*). These are the same methods and models that have been used in other Westinghouse reload cycle designs.

A number of changes to the Technical Specifications were approved as part of the transition to 422V+ fuel assemblies and extended power uprate. These changes include (1) a reduction in the required shutdown margin to 1.3% Δp, (2) a reduction in the $F^N_{\Delta H}$ limit to 1.72 (422V+) and 1.60 (OFA/VANTAGE+), and (3) an increase in $F^N_Q(z)$ to 2.60.

Power distributions and peaking factors are primarily loading-pattern dependent. The usual methods, such as enrichment variation can be employed to ensure compliance with the peaking factor Technical Specifications.

4.3.4 TESTS FOR REACTIVITY ANOMALIES

Tests for reactivity anomalies or design errors are obtained during the reload startup tests. Review acceptance criteria are applied to the comparison of measured and predicted results at startup to identify reactivity anomalies.

Monitoring for reactivity anomalies over depletion of the fuel is accomplished by obtaining a measurement of the boron concentration, correcting the measurement to a set of reference plant operating conditions, and plotting the results versus fuel burnup. A reactivity anomaly can be identified by departure of the corrected measured boron concentration from the predicted boron value or path.

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4.4 THERMAL AND HYDRAULIC DESIGN

This section presents an evaluation of the characteristics and design parameters which are significant to the thermal-hydraulic design objectives. The capability of the reactor to achieve these objectives while performing safely under operational modes, including both transient and steady-state, is demonstrated in this section.

4.4.1 DESIGN BASIS

The design basis for the thermal and hydraulic design of the reactor is presented in Section 4.2.1.

4.4.2 DESCRIPTION AND EVALUATION OF THE THERMAL-HYDRAULIC DESIGN AND ANALYSIS OF RELOAD CORES

This section describes the thermal and hydraulic analysis of reload cores. The design objectives and bases are reviewed, and each of the design and evaluation phases of a reload core is discussed. Relevant design procedures and methods are briefly described and design codes are referenced where appropriate. Constraints from the design initialization specify the hydraulic conditions to be considered for the reload core analysis. The safety-related design bases for the thermal and hydraulic analysis are as specified in Section 4.2.

4.4.2.1 Hydraulic Evaluation

The hydraulic evaluation of the reload core requires a review of the fuel assembly design (nozzles, grids, fuel rods, etc.) that is to be inserted into the core. This design is compared with the design of the fuel assemblies that remain in the core. This comparison is made to ensure that the new fuel assemblies are hydraulically compatible with the fuel assemblies remaining in the core. In general the reload fuel assembly design is identical to the previous fuel assembly design. The best estimate flow rate and mechanical design flow rate are considered in evaluating the core pressure drop and fuel assembly hydraulic loads respectively. This evaluation is performed to verify the conservatism of the core pressure drop and the hydraulic loads upon which the fuel assembly holddown springs are designed.

4.4.2.2 Thermal and Hydraulic Key Safety Parameters

A list of thermal-hydraulic key safety parameters is given in Table 4.4-1. The core power, system pressure, inlet temperature, thermal design flow rate, and core bypass flow are defined during the design initialization phase of the reload design effort. The design radial power distribution for steady-state operation (a peak-to-average $F^N_{\Delta H}$ of 1.72 including measurement uncertainties) and design axial power shape are also defined during the initialization phase.

The values of these parameters are usually identical to the previous cycle design. The departure from nucleate boiling (DNB) correlation to be used in the DNB analyses is also defined during this initialization phase. Fuel density and sintering temperature are important to assess the effects of fuel densification. Changes in the above parameters are evaluated in the determination of the key safety parameters.

4.4.2.2.1 Engineering Hot-Channel Factors

Engineering hot-channel factors account for the influence of the variations of fuel pellet diameter, density, and enrichment. The heat flux engineering hot-channel factor, $F^E{}_Q$, which is applied in determining the peak kW/ft and in fuel pellet temperature evaluations, has been conservatively determined and generally will not vary for the reload case. $F^E{}_Q$ does not need to be considered in DNB evaluation as stated in *Reference 1*. The enthalpy rise engineering hot-channel factor, $F^E{}_{\Delta H}$, is directly considered in RTDP by the convolution which sets the design limit DNBR.

4.4.2.2.2 Axial Fuel Stack Shrinkage

Axial fuel stack shrinkage due to fuel densification increases the linear kW/ft used for the fuel temperature calculations and heat flux used in DNB evaluations. The stack height factor is a multiplier on the linear kW/ft and heat flux which accounts for the fuel stack shrinkage. An acceptable model for determining the fuel stack shrinkage is given in *Reference 1*.

4.4.2.2.3 Fuel Temperatures

Fuel temperatures for safety analyses are computed for each first-core design. A summary of the computed quantities is given below:

- Fuel centerline temperature versus kW/ft.
- Fuel average temperature versus kW/ft.
- Fuel surface temperature versus kW/ft.

Temperatures are computed with the PAD 4.0 code (*26*).

Fuel parameters for reload fuel are evaluated to determine if the temperatures that were computed for the reference analysis are applicable to the current reload. Major fuel parameters of interest are pellet density, pellet sintering temperature, helium backfill pressure, and fuel pellet and rod dimensions. The dimensions are generally the same as the prior fuel design. If the reference analysis temperatures are not applicable, a new fuel temperature analysis is performed. If the reference analysis continues to apply, no further evaluation is required.

4.4.2.2.4 Rod Internal Pressure

The rod internal gas pressure of the lead rod in a reactor is limited to a value below that which could cause (1) the diametral gap to increase due to the outward cladding creep during steady-state operation and (2) extensive DNB propagation to occur. This precludes the outward clad creep rate from exceeding the fuel solid swelling rate and, therefore, ensures that the fuel-clad diametral gap will not reopen following contact or increase in size during steady-state operation. Restricting the fuel-clad gap from opening will prevent accelerated fission gas release at high burnup and preclude high burnup fuel from becoming limiting from a loss-of-coolant accident standpoint.

Fuel rod internal pressure is important in evaluating the possibility of clad flattening in pile (*Reference 6*) as well as assessing the degree of burst and blockage which may occur after a loss-of-coolant accident.

Pressures are computed with the PAD 4.0 code (*26*).

Fuel parameters for reload fuel are evaluated to determine if the pressures that were computed for the reference analysis are applicable to the current reload. Major fuel parameters of interest are pellet density, pellet sintering temperature, helium backfill pressure, power history, and fuel pellet and rod dimensions. The dimensions are generally the same as the prior fuel design. If first core or previous reload pressures are not applicable, a new fuel pressure analysis is performed. If the reference analysis continues to apply, no further evaluation is required.

4.4.2.2.5 Core Thermal Limits

Core thermal limits represent the locus of points of core thermal power, primary system pressure, and coolant inlet temperature which ensure that the DNB design basis (see Section 4.2) is satisfied. The design radial power distribution utilized is characterized by the enthalpy rise hot-channel factor $F^N_{\Delta H}$, which increases with decreasing power level. A typical value of

$F^N_{\Delta H}$ versus power is:

$$F^N_{\Delta H} = 1.72\,[1 + 0.3(1 - P)]$$

The design axial power distribution is a 1.75 chopped cosine. This power distribution is used in the VIPRE analysis to determine the core thermal limits. *Reference 7* further describes these criteria, assumptions, and methods.

The method for determining core thermal limits considers the variations in plant operating parameters, nuclear and thermal parameters, fuel fabrication parameters, and code uncertainties, coupled with the correlation uncertainties, to statistically obtain a DNB ratio (DNBR) uncertainty factor. Applying this factor leads to a limit DNBR value to be used for determining core thermal limits. These core thermal limits are used in accident analysis in conjunction with input parameters at their nominal or best estimate values. This method is described in detail in *Reference 23*.

The parameters listed in Table 4.4-1 are important in analyzing the core thermal limits. These parameters of the reload core are compared to those used in the reference analysis of the core thermal limits. A change in a parameter which results in the previous thermal limits being conservative (e.g., a decrease in the radial peaking factor, $F^N_{\Delta H}$) would not require a reevaluation. A change in a parameter which results in the previous thermal limits being nonconservative (e.g., an increase in the radial peaking factor, $F^N_{\Delta H}$) requires a reevaluation. For the reload core design which utilized the statistical method in the previous cycle, the variations in these parameters are reviewed to ensure that the DNBR uncertainty factor and limit DNBR remains applicable. If the variations in these parameters are changed or a new DNB correlation is used, the DNBR uncertainty factor, limit DNBR, and core thermal limits would be reevaluated. The reevaluation may be of two types: a simple quantitative evaluation performed to assess the effect of a change in a parameter on the core thermal limits, or a full reanalysis.

4.4.2.2.6 Key Safety Parameters for Specific Events

This section describes the DNB analysis for specific events. The methods used in reload evaluation of DNB are the same as those discussed in the previous section.

The DNB analysis of the loss of flow accident considers the plant parameters listed in Table 4.4-1, the heat flux and flow variations with time, and the design steady-state radial and axial power distribution in the VIPRE transient analysis to ensure that the DNB design basis is met.

An important parameter in the evaluation of the rod cluster control assemblies misalignment accidents is the radial power distribution. This radial power distribution (characterized by the enthalpy rise hot-channel factor, $F^N_{\Delta H}$) and the design axial power shape, as well as the other plant parameters listed in Table 4.4-1, are considered in the VIPRE analysis.

A DNB analysis of the single rod withdrawal at power accident is performed to determine the number of rods in DNB, as appropriate. The plant parameters listed in Table 4.4-1 which include the design steady-state radial and axial power distribution are considered in the VIPRE analysis. To determine the number of rods in DNB, the radial peaking factor $F^N_{\Delta H}$ is determined which satisfies the following conditions:

A. Minimum allowable DNBR is met.

B. Hot-channel exit quality is within the range of the DNB correlation.

A fuel census curve is then used to determine the percent of rods with powers greater than this $F^N_{\Delta H}$ and thus assumed to be in DNB.

In the DNB analysis of the hypothetical steam line break, the transient state points (combinations of reactor coolant pressure, inlet temperature, flow rate, and core power level) along with the radial power distribution, $F^N_{\Delta H}$ and axial power profile are included in the VIPRE analysis to ensure that the DNB design basis is met.

4.4.2.3 VIPRE Code

VIPRE is a subchannel code which has been developed to account for hydraulic and nuclear effects on the enthalpy rise in the core. The behavior of the hot assembly is determined by superimposing the power distribution among the assemblies upon the inlet flow distribution while allowing for flow mixing and flow distribution between assemblies. The average flow and enthalpy in the hottest assembly is obtained from the core-wide, assembly-by-assembly analysis. The local variations in power, fuel rod and pellet fabrication, and mixing (engineering hot-channel factors) within the hottest assembly are then superimposed on the average conditions of the hottest assembly in order to determine the conditions in the hot channel.
Further descriptions of this code and its applications are given in *Reference 9*.

4.4.2.3.1 Steady-State Analysis

The VIPRE computer program determines the coolant density, mass velocity, enthalpy, vapor void, static pressure, and DNBR distribution along parallel flow channels within a reactor core under all expected operating conditions. The core region being studied is considered to be made up of a number of contiguous elements extending the full length of the core. An element may represent any region of the core from several assemblies to a subchannel.

4.4.2.3.2 Transient Analysis

The VIPRE code is also used for transient DNB analysis (e.g., loss of flow and locked rotor transients).

The conservation equations needed for the transient analysis are included in VIPRE by including the necessary accumulation terms to the conservation equations used in the steady-state analyses. The input includes one or more of the following time dependent arrays: (1) inlet flow variation, (2) heat flux distribution, (3) system pressure history, and (4) inlet temperature variation.

4.4.3 THERMAL-HYDRAULIC METHODOLOGY FOR OFA/VANTAGE+ AND 422V+ FUEL ASSEMBLY DESIGN EVALUATION

4.4.3.1 General

The calculational methods used in the analysis of the OFA/VANTAGE+ and 422V+are (1) the VIPRE computer code, (2) the WRB-1 DNB correlation and (3) the revised thermal design procedure. In addition, the PAD 4.0 thermal model (*Reference 26*) is used to generate fuel temperatures for safety analysis.

The VIPRE code is used to perform steady-state thermal-hydraulic calculations. VIPRE calculates coolant density, mass velocity, enthalpy, void fractions, static pressure, and DNBR distributions along flow channels within a reactor core under all expected operating conditions. VIPRE is described in detail in *Reference 9*.

The WRB-1 DNB correlation (*Reference 13*) provides a significant improvement in critical heat flux predictions over previous DNB correlations.

The 17 x 17 optimized fuel assembly DNB tests showed that the WRB-1 correlation correctly accounted for the geometry changes in going from the 17 x 17 0.374-in. rod O.D. design to the 17 x 17 0.360-in. rod O.D. design, and that the correlation limit of 1.17 was still applicable (*Reference 14*). The 14 x 14 optimized fuel assembly design involved very similar geometry changes from the seven-grid 14 x 14 standard fuel design, namely, the reduction of the rod O.D. from 0.422-in. to 0.400-in. and the incorporation of a grid design with an increased height and strap thickness due to the change from Inconel to zircaloy. Confirmatory DNB tests performed on the 14 x 14 optimized fuel assembly typical cell geometry verified that the WRB-1 correlation accurately predicted critical heat flux values for this geometry type and that the correlation limit of 1.17 was still appropriate.

The WRB-1 correlation with a 95/95 correlation limit of 1.17 was also used in the DNB analyses for the Ginna 14 x 14 422V+ fuel. The use of the WRB-1 DNB correlation for this fuel design is based on the change notification which introduced the 14 x 14 422V+ mid-grid design (*Reference 27*). The basic change was reverting back to the larger O.D. fuel rod used with the original 14 x14 STANDARD fuel but with a new low pressure drop mid-grid design. The applicability of WRB-1 to this mid-grid was justified under the Westinghouse FCEP process (*Reference 28*).

The W-3 DNBR correlation (*References 15* and *16*) was used where the WRB-1 correlation is not applicable. The WRB-1 correlation was developed based on mixing vane data and therefore is only applicable in the heated rod spans above the first mixing vane grid. The W-3 correlation, which does not take credit for mixing vane grids, is used to calculate DNBR values in the heated region below the first mixing vane grid. In addition, the W-3 correlation is applied in the analysis of accident conditions where the system pressure is below the range of the WRB-1 correlation. For system pressures in the range of 500 to 1000 psia, the W-3 correlation limit is 1.45 (*Reference 25*). For system pressures greater than 1000 psia, the W-3 correlation limit is 1.30.

The design method employed to meet the DNB design basis is the revised thermal design procedure (*Reference 23*). Uncertainties in plant operating parameters, nuclear and thermal parameters, fuel fabrication parameters, and computer codes are considered statistically. To this convolution of plant system and performance uncertainties is added the uncertainties of the correlation itself as defined from the test basis. These two major components of calculational uncertainty are combined to define an overall DNBR limit such that there is at least a 95% probability that the minimum DNBR of the limiting rod will be greater than or equal to this value to satisfy the DNB design criterion. This DNBR uncertainty establishes a design limit DNBR value that must be met in plant safety analyses. Since the parameter uncertainties are considered in determining the design limit DNBR value, the plant safety analyses are performed using values of input parameters without uncertainties. In addition, margin is allocated to the design limit DNBR values to set values designated as the safety analysis limit DNBR values. The plant allowance available between the safety analysis limit DNBR values and the design limit DNBR values will be used to offset DNBR penalties and for flexibility in the design and operation of this plant.

The table below indicates the relationship between the correlation limit DNBR, design limit DNBR, and the safety analysis limit DNBR values used for the Ginna fuel designs.

	Westinghouse 14 x 14 OFA/ VANTAGE+ and 422V+ Fuel Assembly	
	OFA/VANTAGE+	422V+
Correlation	WRB-1	WRB-1
Correlation limit DNBR (STDP)	1.17	1.17
Design limit DNBR (RTDP)	1.24	1.24

	Westinghouse 14 x 14 OFA/ VANTAGE+ and 422V+ Fuel Assembly	
Safety analysis limit DNBR (RTDP)	1.34	1.38
DNBR Margin (RTDP)	7.5%	10.1%

The margin between the design limit and the safety analysis limit DNBR is more than enough to offset the DNBR penalties associated with the Ginna core, which include rod bow, steam generator tube plugging, thimble plug removal, and transition core penalties.

4.4.3.2 Rod Bow

Rod bow can occur between mid-grids, reducing the spacing between adjacent fuel rods and reducing the margin to DNB. Rod bow must be accounted for in the DNBR safety analysis of Condition I and Condition II events. Westinghouse has conducted tests to determine the impact of rod bow on DNB performance; the testing and subsequent analyses were documented in *Reference 18*.

Currently, the maximum rod bow penalty for the OFA fuel assembly is 1.0% DNBR at an assembly average burnup of 24,000 MWD/MTU (*References 18 and 29*). No additional rod bow penalty is required for burnups greater than 24,000 MWD/MTU since credit is taken for the effect of $F^N_{\Delta H}$ burndown due to the decrease in fissionable isotopes and the buildup of fission products (*Reference 30*). Based on the testing and analyses of various fuel array designs documented in *Reference 18*, including the 14 x 14 STANDARD assembly, the 14 x 14 OFA and the 14 x 14 422V+ fuel assemblies should have the same rod bow penalty applied to the analysis basis as that used for 14 x 14 STANDARD fuel assemblies.

4.4.4 THERMAL AND HYDRAULIC TESTS AND INSPECTIONS

General hydraulic tests on models were initially used to confirm the design flow distributions and pressure drops (*References 19* and *20*). Fuel assemblies and control rod drive mechanisms were also tested. Onsite measurements were made to confirm the design flow rates.

Vessel and internals inspections were also reviewed to check such thermal and hydraulic design values as bypass flow. An extensive program of preoperational physics testing was performed using the in-core instrumentation system to verify that actual power distributions in the core were satisfactory.

4.4.5 REACTOR COOLANT FLOW MEASUREMENT

In the design of the reactor coolant system, design margin was applied to both the calculated system pressure drop and the pump design head as contrasted to "best estimate" calculations to ensure a system flow rate equal to or greater than design flow rate. Straightforward hydraulics techniques were employed together with detailed model tests using scaling techniques in accordance with Hydraulic Institute Standards. This design approach has been substantiated by measurements in all operating Westinghouse-designed plants.

Core safety limits are not particularly sensitive to the absolute value of reactor coolant system flow. In the course of plant startup, data pertinent to determining coolant flow both directly and indirectly were obtained to verify that flow is at or greater than design. A definite exact measurement of flow is not necessary for plant operation or for protection system purposes, as described later. Further there are no design provisions to vary flow, i.e., throttling devices; therefore, variations in absolute flow are not of concern during operation. Protection in the event of a loss of flow resulting from loss of power to one or both pumps is analyzed in Section 15.3.

In the original FSAR several methods were discussed that could be used to determine that actual coolant system flow was greater than the assumed design flow. The methods are described below and consist of a pump power measurement, a secondary heat balance coupled with coolant temperatures, elbow tap differential pressure measurements, core outlet thermocouple measurements, and a pump power-differential pressure iterative procedure. By operating each pump alone and both together, two different flow rates can be evaluated with the above methods as further confirmation of the pump flow characteristics.

4.4.5.1 Pump Power

Reactor coolant system flow rate can be determined from a measurement of reactor coolant pump input power by determining from the pump input power versus pump capacity characteristic curve the pump output in flow for the input power measured. Pump input power can be measured accurately. Pump power measurements are made on the actual pump motors prior to installation in the plant to determine motor characteristics.

4.4.5.2 Secondary Heat Balance

System flow rate is calculated by accurately measuring the secondary system power generation together with the corresponding measured hot- to cold-leg temperature differential in the reactor coolant system (loop delta T). Flow is equal to power divided by the reactor coolant enthalpy decrease. Further discussion of this method is in Section 4.4.5.8.

4.4.5.3 Elbow Tap Differential Pressure

Measurement of the elbow tap flow meter differential pressure provides a highly repeatable measure of system flow rate. The flow rate is determined from the measured 90-degree elbow differential pressure by documented (*Reference 21*) standard elbow characteristics.

4.4.5.4 Core Exit Thermocouple

The core differential temperature can be determined from the cold-leg temperature and a core exit thermocouple map. This is then compared with total generated secondary system power generation to determine total core flow. The core exit thermocouple system has 36 thermocouples positioned to measure fuel assembly coolant outlet temperatures at preselected core locations and three thermocouples to measure temperatures in the reactor vessel head area. The core exit thermocouple signals are converted to degrees Fahrenheit and input to the plant computer and a control room display. The core exit thermocouple system meets the requirements of NUREG 0737 and Regulatory Guide 1.97, Revision 3, for post-accident monitoring.

4.4.5.5 Pump Power-Differential Pressure

This procedure has been used experimentally in an existing plant. The results have produced calculated flow rates in close agreement with the analytically predicted most probable flow and consistent flow rates to within ±0.3% for a number of pumps. It is a refinement of the pump power method that utilizes a procedure to establish the actual pump input power performance curve more accurately. The actual operating curve is established from its known shape, determined from model tests, by interrelating pump input power and a relative change in system pressure drop under conditions of one and two pumps running. This procedure reduces the uncertainties associated with the absolute relation of the pump input power curve to flow. This procedure is described in more detail than the more familiar methods mentioned previously. Figure 4.4-1 is an example of a typical pump input power curve and is included to describe the procedure which is as follows:

A. With the reactor coolant system pressurized, all pumps are started. The flow within the loop to be measured is assumed to be equal to the design (represented by line 1 on Figure 4.4-1) and pump power (represented by line 2 on Figure 4.4-1) and a reference differential pressure measured. The intersection of lines 1 and 2 establishes a point on the assumed pump power input curve. This allows construction of the assumed curve by shifting the model test curve vertically until it intersects this point.

B. The other pump is stopped. The flow within the active loop increases because of the reduced flow through the reactor vessel. This increased flow above the assumed design flow is determined from the relative increase in the measured differential pressure.

C. This increased flow is then plotted on Figure 4.4-1 (line 3). Its intersection with the previously assumed pump curve will yield the amount of anticipated input power (line 4). If the anticipated input power equals the measured input power with one pump running, the originally assumed flow rate was correct.

The above procedure is all that is necessary to establish whether actual flow is less than, equal to, or greater than design flow. The sense of the difference between anticipated one loop operation input power and measured one loop input power will indicate this. If anticipated power is greater than measured power, the actual flow rate was greater than design. (This can be seen by following the construction of lines 5, 6, and 7 in Figure 4.4-1.) If it is desired to know the actual flow rate, the flow with all pumps operating must again be assumed and the construction of the lines repeated until anticipated one loop power equals measured one loop input power.

This procedure makes use of elbow tap (or steam generator) differential pressure readings. These readings are not used as absolute quantities but only in reference to each other in order to determine the magnitude of the change in flow from one point to another. Therefore, calibration or accurate knowledge of elbow characteristics and dimensions are not required.

The accuracy of this procedure is affected by the accuracy of measured input power, the accuracy of determining the relative change in flow, and the accuracy of the shape of the input power curve. From a review of data from full scale tests of smaller earlier model pumps and the accuracies associated with model tests and hydraulic scaling theory, it has been judged that an accuracy of ±0.5% is a conservative tolerance to apply to the accuracy of the shape of the curve. The relative change in flow between the two pump running condition

and the one pump running condition can be determined to an accuracy of ±0.5% by the use of pretest deadweight tester calibrated Foxboro differential pressure cells and a Hewlett-Packard digital voltmeter. Pump input power can be measured to an accuracy of ± 0.5% by use of procedures and instrumentation available from a test organization at the Westinghouse Large Rotating Apparatus Division. Typical instrumentation that would be used consists of a Weston Model 329 wattmeter and Westinghouse Model PA 151 volt and ammeters. These accuracies result in an expected total flow rate measurement accuracy of ± 2.5%.

4.4.5.6 Experience

Each of the above methods is employed to obtain an independent assessment of flow. Each is evaluated in terms of consistency with one another as well as between loops. Possible error allowances are established on the basis of various in-plant calibrations, i.e., loop temperature and in-core thermocouple isothermal calibrations. Experience has shown that all methods indicate greater than design flow with good agreement between loops and reasonable agreement between methods sufficient to validate greater than design flow. An example of measured data is listed below.

	Pump Power	***Elbow Taps***	***Core Thermocouple***
Loop A	113%	107%	
Loop B	110%	107%	105%

4.4.5.7 Low Flow Trip Setpoint

Elbow taps are used in the primary coolant system as an instrument device that indicates the status of the reactor coolant flow. The basic function of this device is to provide information as to whether or not a reduction in flow rate has occurred. The correlation between flow rate reduction and elbow tap read-out has been well established by the following equation (*Reference 21*):

$$\frac{deltaP}{deltaP_0} = \left(\frac{\omega}{\omega_0}\right)^2$$

(Equation 4.4-1)

where delta P_o is the referenced pressure differential, with the corresponding referenced flow rate ω_o and delta P is the pressure differential with the corresponding flow rate ω. The full-flow reference point is established during initial plant startup. The low-flow trip point is then established by extrapolating along the correlation curve. The technique has been used in providing core protection against low coolant flow in Westinghouse PWR plants. Field results have shown the repeatability of the setpoint to be within ± 1%. Transient analysis for a loss of flow assumes instrumentation error of ≤ 4%.

4.4.5.8 Precision Calorimetric Measurement for Reactor Coolant System Flow

The Improved Thermal Design Procedure, which was first used beginning with cycle 14, as well as the Revised Thermal Design Procedure, utilized beginning with cycle 26, require a reactor coolant system flow measurement with a high degree of accuracy such that flow measurement can be performed by determining the steam generator thermal output, corrected for the reactor coolant pump heat input and the loop's share of primary system heat losses, and the enthalpy rise (delta h) of the primary coolant. Assuming that the primary and secondary sides are in equilibrium, the reactor coolant system total vessel flow is the sum of the individual primary loop flows, i.e.

$$W_{RCS} = \Sigma W_L$$

The individual primary loop flows are determined by correcting the thermal output of the steam generator for steam generator blowdown (if not secured), subtracting the reactor coolant pump heat addition, adding the loop's share of the primary side system losses, dividing by the primary side enthalpy rise, and multiplying by the specific volume of the reactor coolant system cold leg. The equation for this calculation is

$$W_L = \frac{\gamma \cdot \left\{Q_{SG} - Q_p + \frac{Q_L}{N}\right\} V_C}{\left[h_H - h_C\right]}$$

(Equation 4.4-2)

where: W_L = loop flow (gpm)

γ = 0.1247 gpm/(ft^3/hr)

Q_{SG} = steam generator thermal output (Btu/hr)

Q_p = reactor coolant pump heat adder (Btu/hr)

Q_L = primary system net heat losses (Btu/hr)

V_c = specific volume of the cold leg at T_C (ft^3/lb)

N = number of primary side loops

h_H = hot-leg enthalpy (Btu/lb)

h_c = cold-leg enthalpy (Btu/lb)

As an alternative to the individual loop methodology, it is also possible to obtain a calorimetric for both steam generators combined and calculate hot and cold leg average enthalpies to arrive at total reactor coolant system flow. The thermal output of the steam generator is determined by the same calorimetric measurement as for reactor power, which is defined as

Q_{SG} = $(h_s - h_f)\ W_f$

where

h_s = steam enthalpy (Btu/lb)

h_f = feedwater enthalpy (Btu/lb)

W_f = feedwater flow (lb/hr)

The steam enthalpy is based on measurement of steam generator outlet steam pressure, assuming saturated conditions. The feedwater enthalpy is based on the measurement of feedwater temperature and steam pressure. The feedwater flow is determined by multiple measurements and the same calculation as used for reactor power measurements, which is based on the following:

$$W_f = (K)(F_a) \cdot \sqrt{\rho_f \cdot \Delta p}$$

(Equation 4.4-3)

where: K = feedwater venturi flow factor

F_a = feedwater venturi correction for thermal expansion

ρ_f = feedwater density (lb/ft^3)

Δp = feedwater venturi pressure drop (in. H_2O)

The feedwater venturi flow coefficient is the product of a number of constants including as-built dimensions of the venturi and calibration tests performed by the vendor. The thermal expansion correction is based on the coefficient of expansion of the venturi material and the difference between feedwater temperature and calibration temperature. Feedwater density is based on the measurement of feedwater temperature and feedwater pressure. The venturi

pressure drop is obtained from the output of the differential pressure cell connected to the venturi.

The reactor coolant pump heat adder is determined by calculation, based on the best estimates of coolant flow, pump head, and pump hydraulic efficiency.

The primary system net heat losses are determined by calculation, considering the following system heat inputs and heat losses:

- Charging flow
- Letdown flow
- Seal injection/seal return flow
- Reactor coolant pump thermal barrier cooler heat removal
- Pressurizer spray flow
- Pressurizer surge line flow
- Component insulation heat losses
- Component support heat losses
- Control rod drive mechanism heat losses

A single calculated sum for full power operation is used for these losses/heat inputs.

The hot-leg and cold-leg enthalpies are based on the measurement of the hot-leg temperature, cold-leg temperature, and the pressurizer pressure. The cold-leg specific volume is based on measurement of the cold-leg temperature and pressurizer pressure.

The reactor coolant system flow measurement is thus based on the following plant measurements.

- Steam line pressure (P_s)
- Feedwater temperature (T_f)
- Feedwater venturi differential pressure (delta P)
- Hot-leg temperature (T_H)
- Cold-leg temperature (T_C)
- Pressurizer pressure (P_p)
- Steam generator blowdown (if not secured)

and on the following calculated values.

- Feedwater venturi flow coefficients (K)
- Feedwater venturi thermal expansion correction (F_a)
- Feedwater density (ρ_f)

- Feedwater enthalpy (h_f)
- Steam enthalpy (h_s)
- Primary system net heat losses (Q_L)
- Reactor coolant pump heat adder (Q_p)
- Hot-leg enthalpy (h_H)
- Cold-leg enthalpy (h_c)

This measurement is performed for each cycle starting with cycle 13, verifying the conservatism of the design flow used in the safety analysis. The uncertainty of this measurement was calculated to be less than that assumed in determining the design limit for DNBR using the Revised Thermal Design Procedure.

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17. Deleted.

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22. Deleted.

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29. Letter from Rahe, E. P. (Westinghouse) to Miller, J. R. (NRC), "Partial Response to Request Number 1 for Additional Information on WCAP-8691, Revision 1," NS-EPR-2515, October 9, 1981; and Letter from Rahe, E. P., (Westinghouse) to Miller, J. R. (NRC), "Remaining Response to Request Number 1 for Additional Information on WCAP-8691, Revision 1," NS-EPR-2572, March 16, 1982.

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Table 4.4-1
THERMAL AND HYDRAULIC DESIGN PARAMETERS[a]

Total heat output, Mwt	1775
Total heat output, Btu/hr	6057×10^6
Heat generated in fuel, %	97.4
Maximum thermal overpower, %	18
Nominal system pressure, psia	2250
Hot-channel factors	
Heat flux	
Nuclear, F^N_Q	2.52
Engineering, F^E_Q	1.03
Total	2.60
Enthalpy rise	
Nuclear, $F^N_{\Delta H}$	1.72 (422V+) 1.60 (VANTAGE+)
Coolant flow	
Total flow rate, lb/hr (TDF)	60.5×10^6
Coolant temperature, °F	
Nominal inlet	540.2
Average rise in vessel	71.6
Average rise in core	76.0
Average in core	580.3
Average in vessel	576.0

Heat transfer	
Active heat transfer surface area, ft^2	98,507
Average heat flux, Btu/hr-ft^2	206,950
Maximum heat flux, Btu/hr-ft^2	538,070
Maximum thermal output, kW/ft	18.2
DNBR	
Minimum DNBR at nominal operating conditions	1.839 typical cell; 1.818 thimble cell
Pressure drop, psi	
Across core at flow of 193,600 gpm (B.E.)	24.7 (422V+) 26.6 (VANTAGE+)

a. These parameters correspond to zero percent steam generator tube plugging and high T_{AVG} (576°F). An evaluation has been performed (*Reference 17* of UFSAR Section 4.4) which shows that all safety limits are satisfied for 15% steam generator tube plugging.

4.5 REACTOR MATERIALS

Reactor vessel materials are discussed in Section 5.3.1. Control rod drive system structural materials and reactor internals materials are discussed below.

4.5.1 CONTROL ROD DRIVE SYSTEM STRUCTURAL MATERIALS

All parts exposed to reactor coolant, such as the pressure vessel, latch assembly, and drive rod, are made of metals which resist the corrosive action of the water.

Three types of metals are used exclusively: stainless steels, Alloy X-750, and cobalt-based alloys. Wherever magnetic flux is carried by parts exposed to the main coolant, stainless steel is used. Cobalt-based alloys are used for the pins and latch tips.

Alloy X-750 is used for the springs of both latch assemblies and type 304 stainless steel is used for all pressure containment. Hard chrome plating provides wear surfaces on the sliding parts and prevents galling between mating parts (such as threads) during assembly.

Outside of the pressure vessel, where the metals are exposed only to the containment environment and cannot contaminate the main coolant, carbon and stainless steels are used. Carbon steel, because of its high permeability, is used for flux return paths around the operating coils. It is zinc-plated 0.001-in. thick to prevent corrosion.

Additional information on the control rod drive system materials is presented in Section 3.9.4.

4.5.2 REACTOR INTERNALS MATERIALS

Information on reactor internals materials is presented in Section 3.9.5.

4.6 FUNCTIONAL DESIGN OF REACTIVITY CONTROL SYSTEM

Information on the functional design and evaluation of the control rod drive system is presented in Section 7.7. The mechanical design is discussed in Section 3.9.4. Information on the functional design and evaluation of the chemical and volume control system is presented in Section 9.3.4. Evaluation of the combined performance of reactivity control systems pertaining to the response of the plant to postulated process disturbances and to postulated malfunctions or failures of equipment are presented in Chapter 15 and Section 7.7.

Figure 4.2-1 Typical Rod Cluster Control Assembly

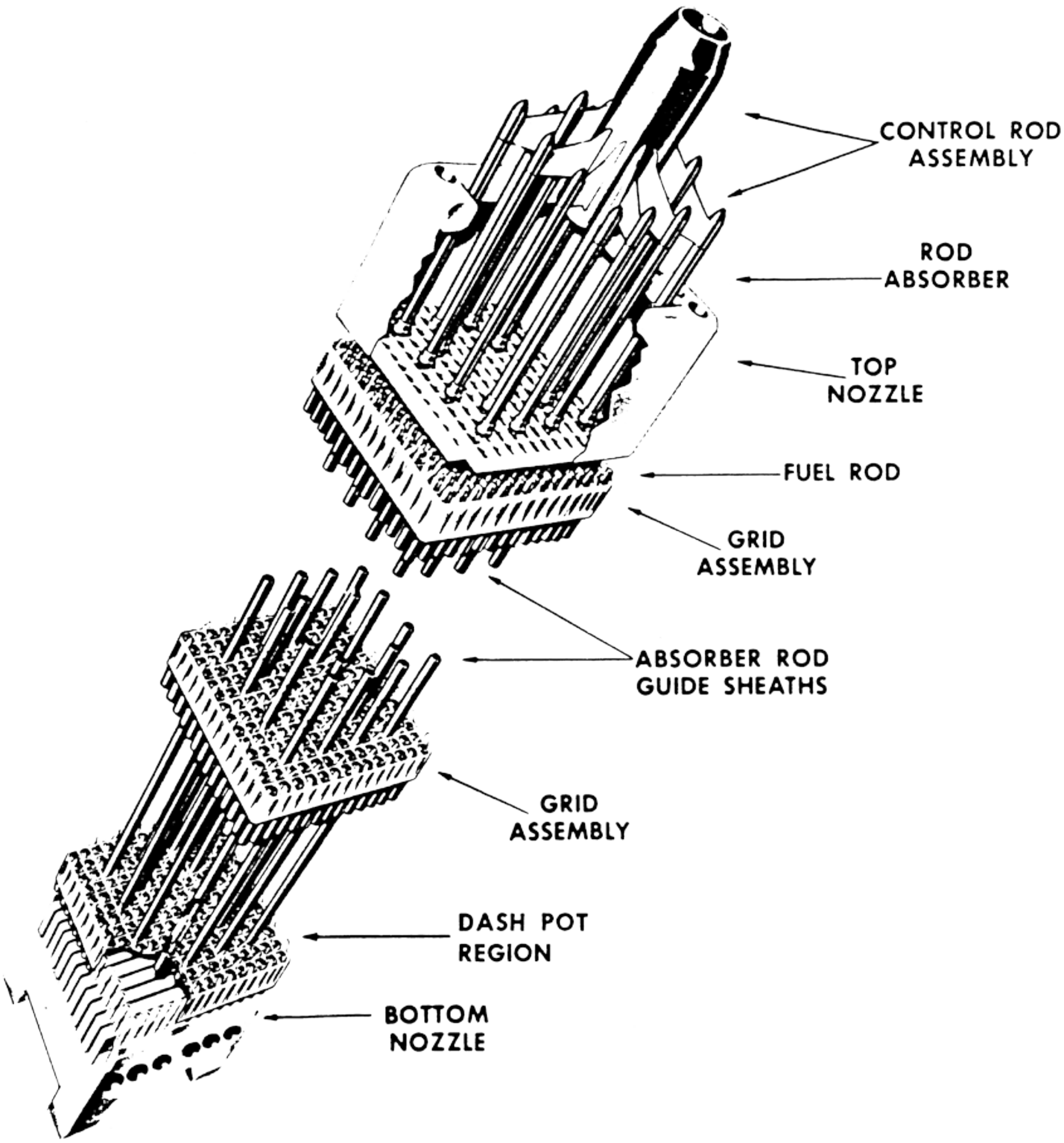

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Figure 4.2-1 Typical Rod Cluster Control Assembly

Figure 4.2-2 Fuel Assembly and Control Cluster Cross Section

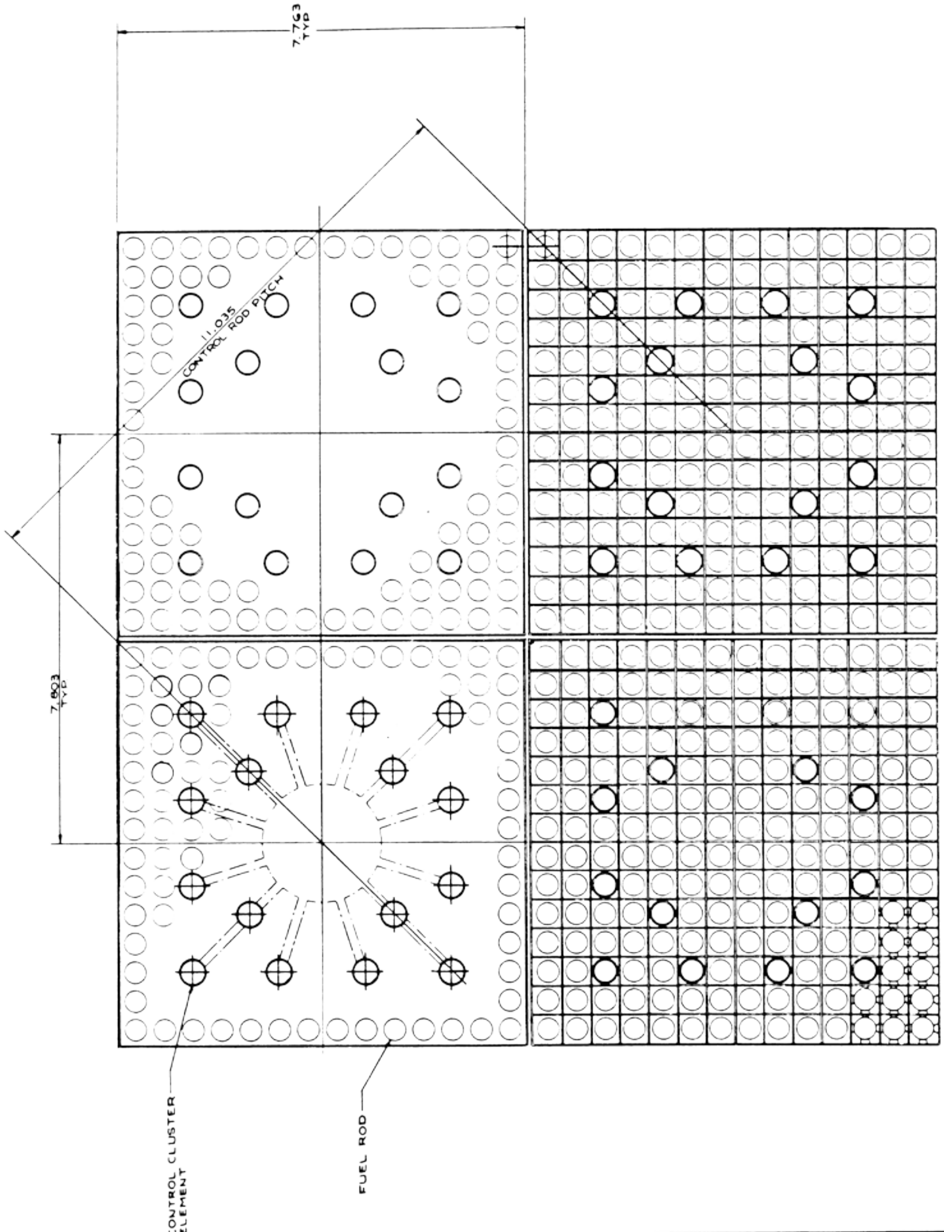

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Figure 4.2-2

Fuel Assembly and Control Cluster Cross Section

Figure 4.2-3 14 x 14 OFA and 422V+ Fuel Assemblies

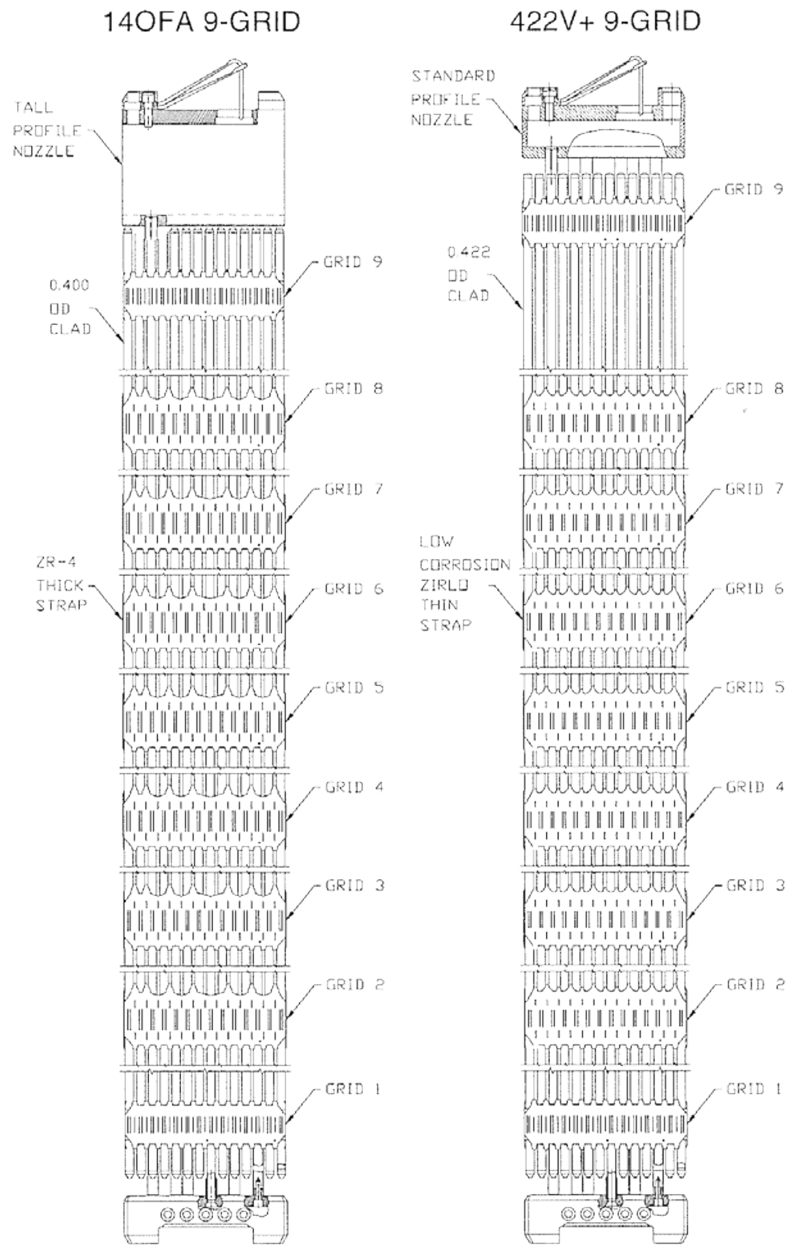


Figure 4.2-4 OFA and 422V+ Top Nozzle Assemblies

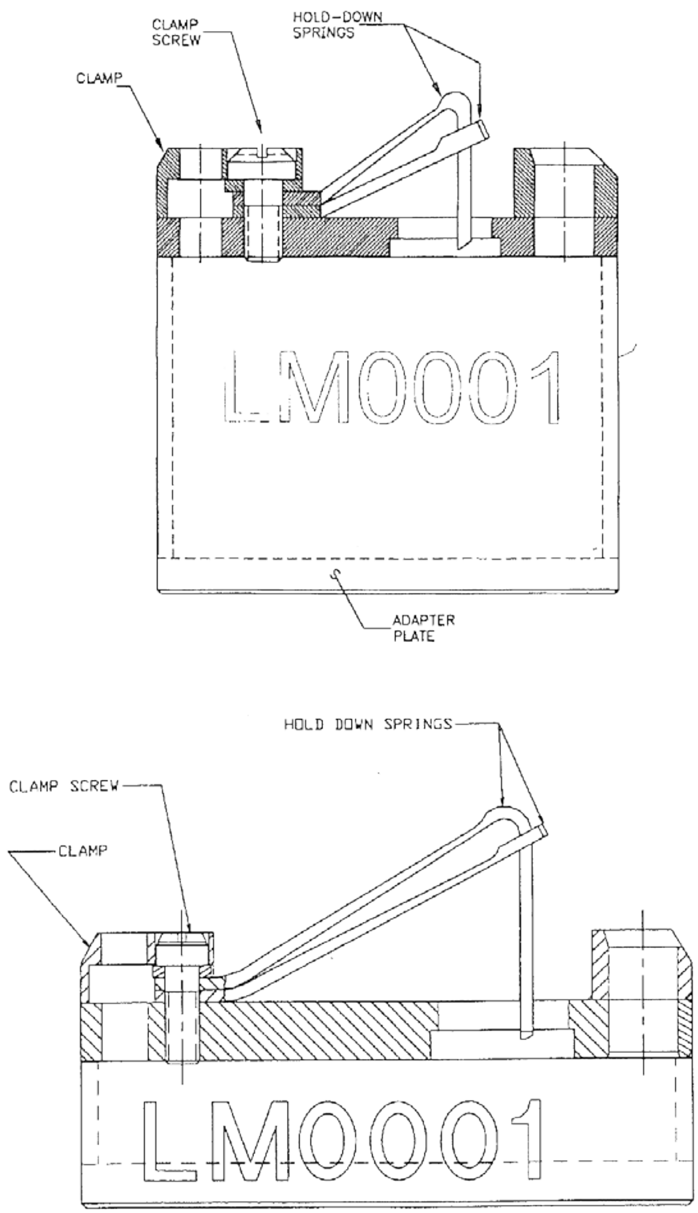
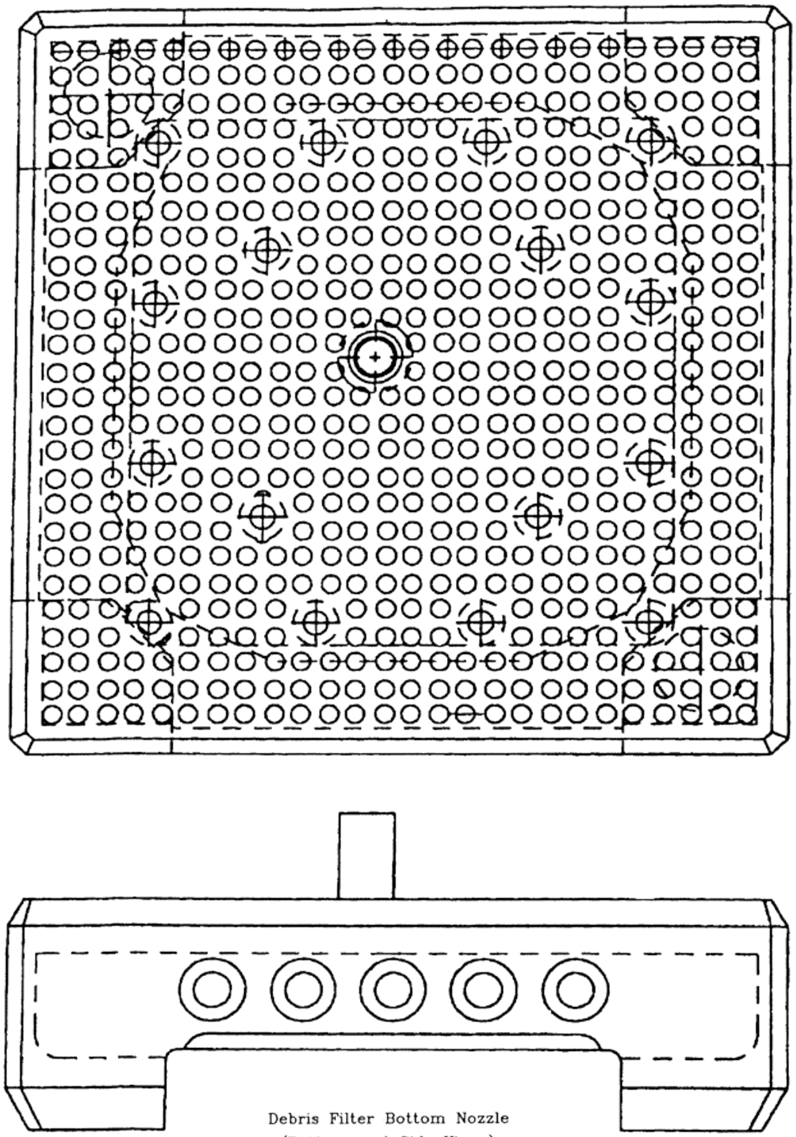


Figure 4.2-5 Debris Filter Bottom Nozzle



Debris Filter Bottom Nozzle
(Bottom and Side Views)

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Figure 4.2-5 Debris Filter Bottom Nozzle

Figure 4.2-6 Optimized Guide Thimble Assembly

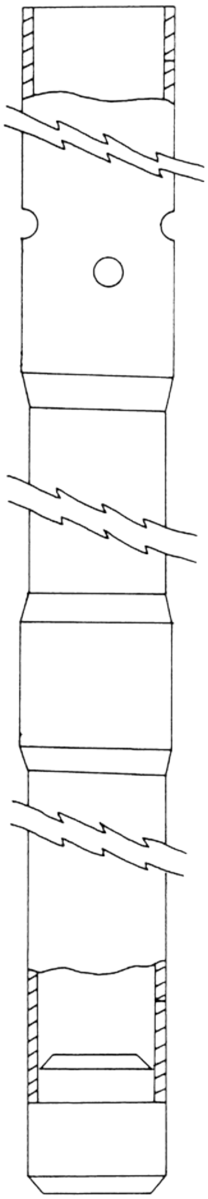

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Figure 4.2-6
Optimized Guide Thimble Assembly

Figure 4.2-7 Optimized Instrumentation Tube

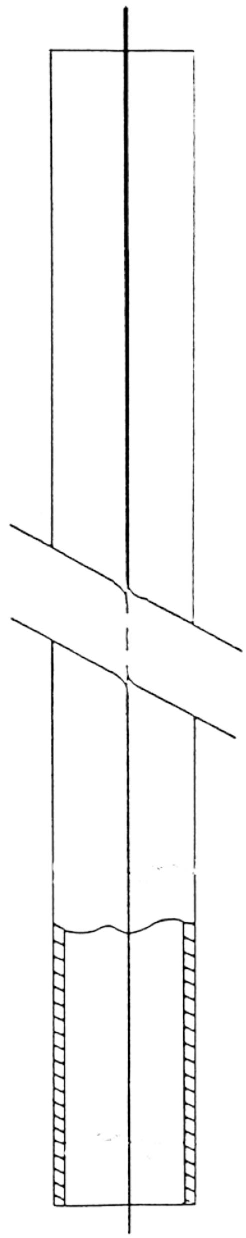

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Figure 4.2-7
Optimized Instrumentation Tube

Figure 4.2-8 Mid-Grid Connection

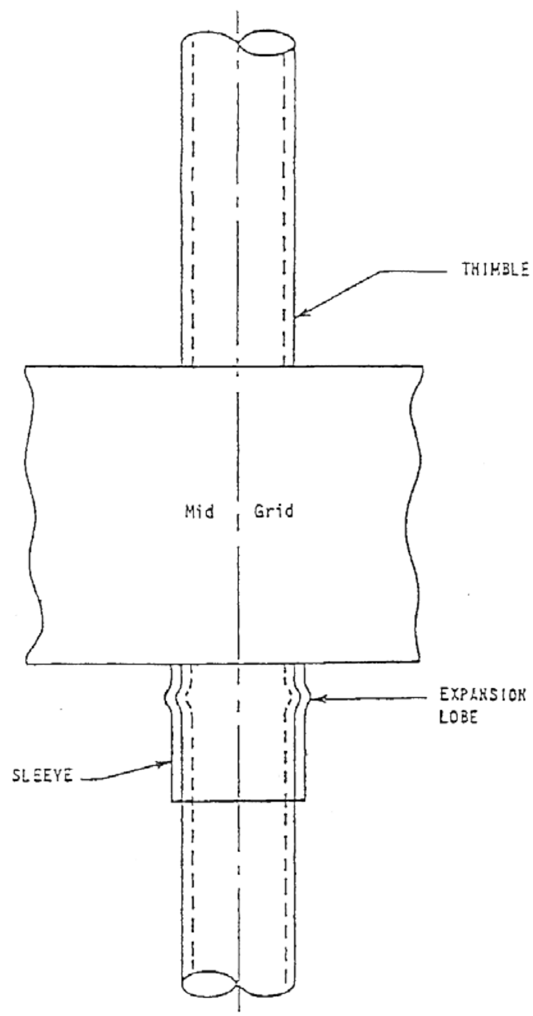
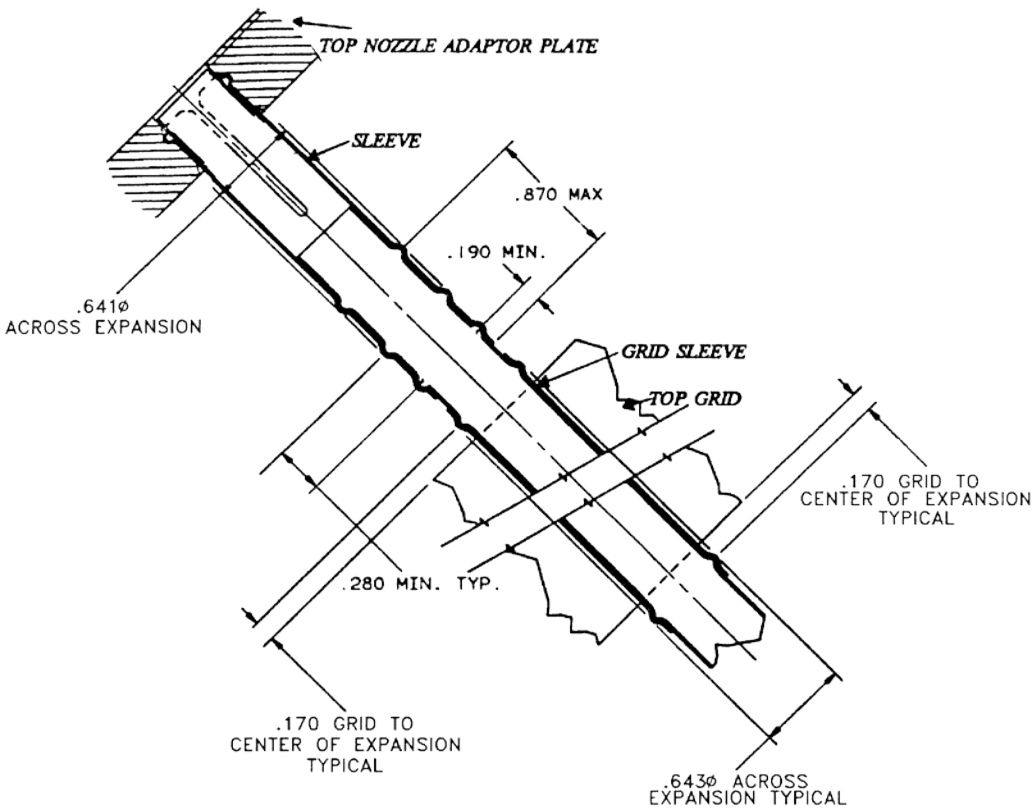
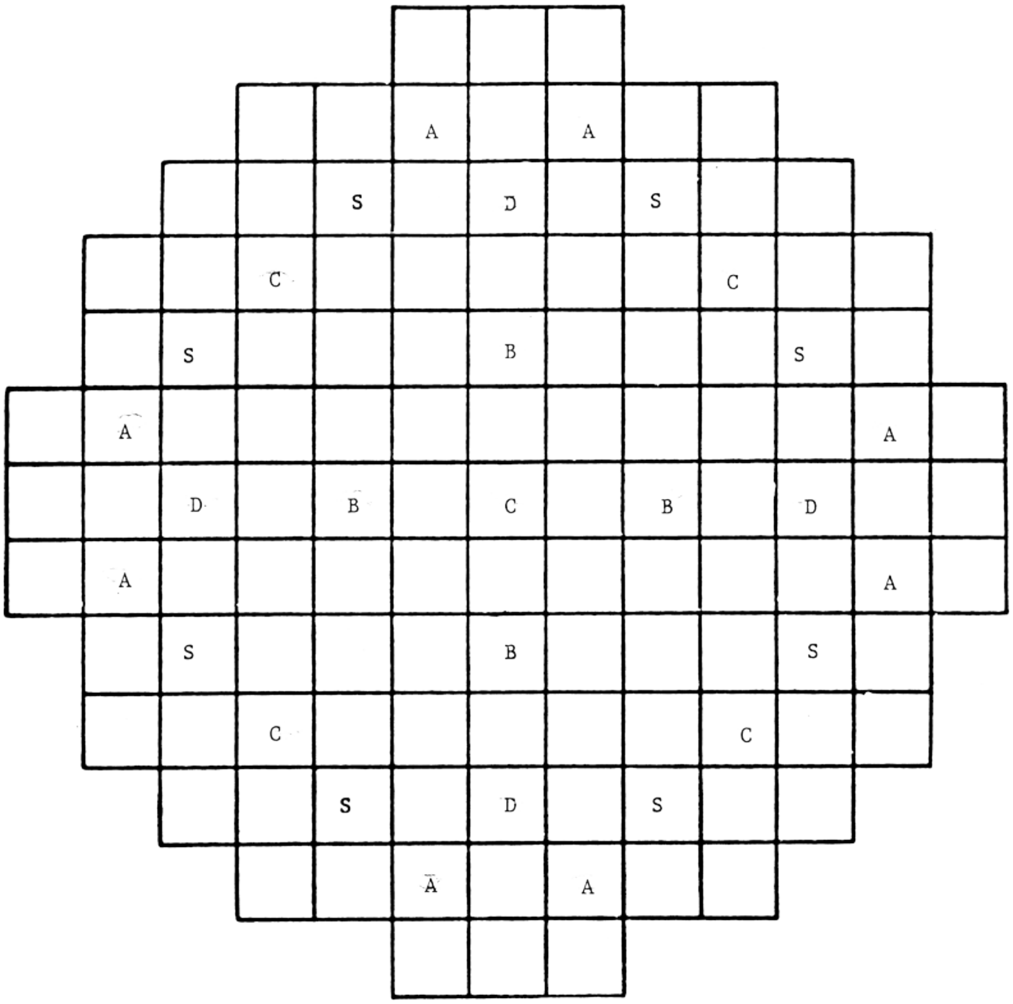


Figure 4.2-9 Removable Top Nozzle and Top Grid Connection



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 4.2-9 Removable Top Nozzle and Top Grid Connection

Figure 4.3-1 Control Rod Cluster Groups



Control Group 1	(A)	8
Control Group 2	(B)	4
Control Group 3	(C)	5
Control Group 4	(D)	4
Shutdown	(S)	8
TOTAL		29

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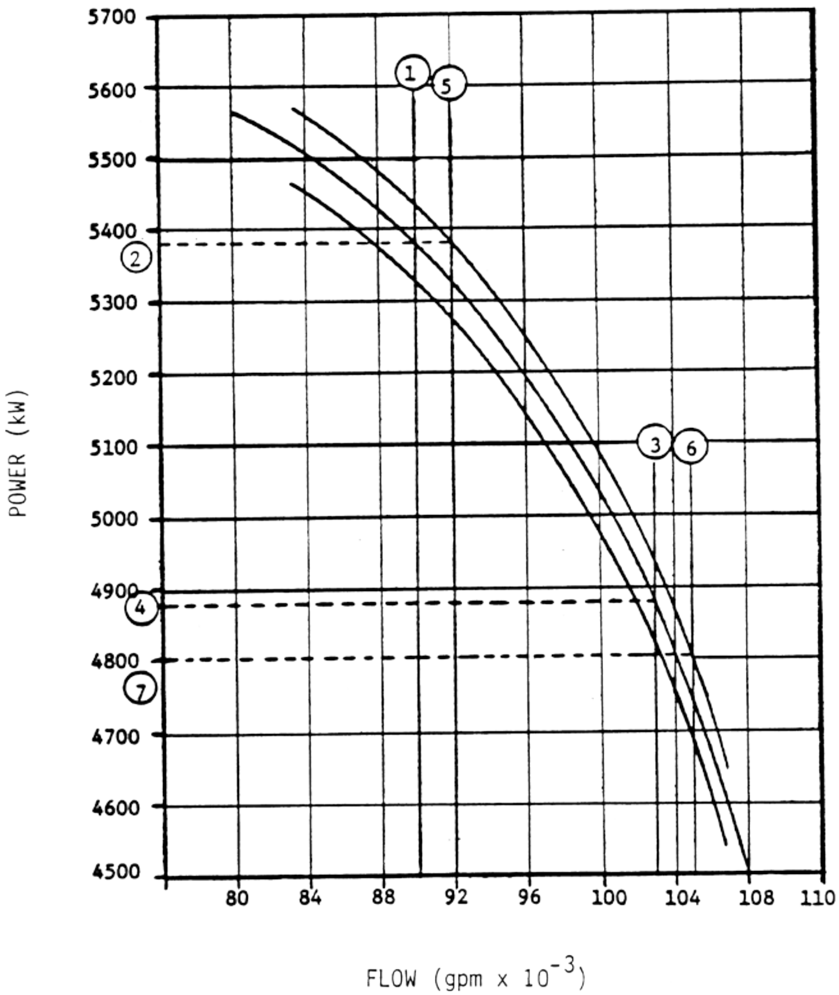
Figure 4.3-1

Control Rod Cluster Groups

Figure 4.4-1 Typical Pump Power Versus Flow Curves



NOTE: SEE SECTION 4.4.5.5 FOR EXPLANATION OF CIRCLED NUMBERS

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Figure 4.4-1 Typical Pump Power Versus Flow Curves

5 REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS

5.1 SUMMARY DESCRIPTION

5.1.1 GENERAL

The reactor coolant system, shown in Drawings 33013-1258 and 33013-1260, consists of two identical heat transfer loops connected in parallel to the reactor vessel. Each loop contains a circulating pump and a steam generator. The system also includes a pressurizer, pressurizer relief tank, connecting piping, and instrumentation necessary for operational control. The pressurizer is connected to the B loop. Auxiliary system piping connections into the reactor coolant piping are provided as necessary.

Pressure in the system is controlled by the pressurizer, where water and steam pressure is maintained through the use of electrical heaters and sprays. Steam can either be formed by the heaters or condensed by a pressurizer spray to minimize pressure variations due to contraction and expansion of the coolant. Spring-loaded steam safety valves and Pressurizer Power Operated Relief Valves (PORV) are connected to the pressurizer and the discharge to the pressurizer relief tank, where discharged steam is condensed and cooled by mixing with water.

Major components which are located inside the containment are indicated in Drawings 33013-1258 and 33013-1260 by the containment boundary. The intersection of a process line with this boundary indicates a functional penetration.

Reactor coolant system design data are listed in Tables 5.1-1 through 5.1-3.

5.1.2 PERFORMANCE OBJECTIVES

The reactor coolant system transfers the heat generated in the core to the steam generators, where steam is generated to drive the turbine generator. Demineralized water is circulated at the flow rate and temperature that are consistent with achieving the reactor core thermal-hydraulic performance presented in Chapter 4. The water also acts as a neutron moderator and reflector, and as a solvent for the neutron absorber used in chemical shim control.

The reactor coolant system provides a boundary for containing the coolant under operating temperature and pressure conditions. It serves to confine radioactive material and limits to acceptable values its uncontrolled release to the secondary system and other parts of the plant. During transient operation, the heat capacity of the system attenuates thermal transients that are generated by the core or extracted by the steam generators. The reactor coolant system accommodates coolant volume changes within the protection system criteria.

By appropriate selection of the inertia of the reactor coolant pumps, the thermal-hydraulic effects are reduced to a safe level during the pump coastdown which would result from a loss-of-flow situation. The layout of the system ensures the natural circulation capability following a loss of flow to permit plant cooldown without overheating the core.

5.1.3 DESIGN CRITERIA

The design criteria discussed in Sections 5.1.3.1 through 5.1.3.9 were used during the licensing of Ginna Station. They represent the Atomic Industrial Forum (AIF) version of

proposed criteria issued by the AEC for comment on July 10, 1967. Conformance with the General Design Criteria (GDC) of 10 CFR 50, Appendix A, is discussed in Section 5.1.3.10.

The following design criteria apply to the reactor coolant system.

5.1.3.1 Quality Standards

CRITERION: Those systems and components of reactor facilities which are essential to the prevention, or the mitigation of the consequences, of nuclear accidents which could cause undue risk to the health and safety of the public shall be identified and then designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed. Where generally recognized codes and standards pertaining to design, materials, fabrication, and inspection are used, they shall be identified. Where adherence to such codes or standards does not suffice to assure a quality product in keeping with the safety function, they shall be supplemented or modified as necessary. Quality assurance programs, test procedures, and inspection acceptance criteria to be used shall be identified. An indication of the applicability of codes, standards, quality assurance programs, test procedures, and inspection acceptance criteria used is required. Where such items are not covered by applicable codes and standards, a showing of adequacy is required (AIF-GDC 1).

The reactor coolant system is of primary importance with respect to its safety function in protecting the health and safety of the public.

Quality standards of material selection, design, fabrication, and inspection conform to the applicable provisions of recognized codes and good nuclear practice (Section 5.2.1.2). Details of the quality assurance programs, test procedures, and inspection acceptance levels are given in Section 5.2.3. Particular emphasis is placed on quality assurance in the selection of reactor vessel materials that have properties which are uniformly within tolerances appropriate to the application of the design methods of the code.

5.1.3.2 Performance Standards

CRITERION: Those systems and components of reactor facilities which are essential to the prevention or to the mitigation of the consequences of nuclear accidents which could cause undue risk to the health and safety of the public shall be designed, fabricated, and erected to performance standards that will enable such systems and components to withstand, without undue risk to the health and safety of the public the forces that might reasonably be imposed by the occurrence of an extraordinary natural phenomenon such as earthquake, tornado, flooding condition, high wind or heavy ice. The design bases so established shall reflect: (A) appropriate consideration of the most severe of these natural phenomena that have been officially recorded for the site and the surrounding area and (B) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design (AIF-GDC 2).

All piping, components, and supporting structures of the reactor coolant system are designed as Seismic Category I equipment, i.e., they are capable of withstanding the following stresses with no loss of function:

A. Code-allowable working stresses for the design seismic ground acceleration.

B. The maximum potential seismic ground acceleration acting in the horizontal and vertical direction simultaneously.

Details are given in Section 5.4.11.

The reactor coolant system is located in the containment, the design of which, in addition to being a Seismic Category I structure, also considers accidents or other applicable natural phenomena. Details of the containment design are given in Sections 3.8 and 6.2.

5.1.3.3 Records Requirements

CRITERION: The reactor licensee shall be responsible for assuring the maintenance throughout the life of the reactor of records of the design, fabrication, and construction of major components of the plant essential to avoid undue risk to the health and safety of the public (AIF-GDC 5).

Records of the design, fabrication, and construction of the major reactor coolant system components are to be maintained throughout the life of the plant.

5.1.3.4 Missile Protection

CRITERION: Adequate protection for those engineered safety features, the failures of which could cause an undue risk to the health and safety of the public, shall be provided against dynamic effects and missiles that might result from plant equipment failures (AIF-GDC 40).

The dynamic effects during blowdown following a loss-of-coolant accident are evaluated in the detailed layout and design of the high-pressure equipment and barriers which afford missile protection. Fluid and mechanical driving forces are calculated and consideration is given to possible damage due to fluid jets and secondary missiles which might be produced.

The steam generators are supported, guided, and restrained in a manner which prevents rupture of the steam side of a generator, the steam lines, and the feedwater piping as a result of forces created by a reactor coolant system pipe rupture. These supports, guides, and restraints also prevent rupture of the primary side of a steam generator as a result of forces created by a steam or feedwater line rupture.

The mechanical consequences of a pipe rupture are restricted by design such that the functional capability of the engineered safety features is not impaired.

5.1.3.5 Reactor Coolant Pressure Boundary

CRITERION: The reactor coolant pressure boundary shall be designed, fabricated and constructed so as to have an exceedingly low probability of gross rupture or significant uncontrolled leakage throughout its design lifetime (AIF-GDC 9).

The reactor coolant system, in conjunction with its control and protective provisions, is designed to accommodate the system pressures and temperatures attained under all expected modes of plant operation or anticipated system interactions, and maintain the stresses within applicable code stress limits.

Fabrication of the components which constitute the pressure-retaining boundary of the reactor coolant system is carried out in strict accordance with the applicable codes. In addition, there are areas where equipment specifications for reactor coolant system components go beyond the applicable codes. Materials of construction were chosen to lessen the probability of gross leakage or failure. Details are given in Section 5.2.3.

The materials of construction of the pressure-retaining boundary of the reactor coolant system are protected by control of coolant chemistry from corrosion phenomena which might otherwise reduce the system structural integrity during its service lifetime.

System conditions resulting from anticipated transients or malfunctions are monitored and appropriate action is automatically initiated to maintain the required cooling capability and to limit system conditions so that continued safe operation is possible.

The system is protected from overpressure by means of pressure-relieving devices, as required by Section III of the ASME Boiler and Pressure Vessel Code. The Low Temperature Overpressure Protection (LTOP) System is also provided, together with operating precautions to minimize operation under undesirable conditions. (See Section 5.2.2.)

Isolable sections of the system are provided with overpressure-relieving devices discharging to closed systems such that the system code-allowable relief pressure within the protected section is not exceeded.

5.1.3.6 Monitoring Reactor Coolant Leakage

CRITERION: Means shall be provided to detect significant uncontrolled leakage from the reactor coolant pressure boundary (AIF-GDC 16).

Positive indications in the control room of leakage of coolant from the reactor coolant system to the containment are provided by equipment which permits continuous monitoring of containment air activity (R-11 and R-12) and humidity, containment sump A level (LT-2039 and LT-2044), and of runoff from the condensate collection system under the cooling coils of the containment air recirculation (CRFC) units. This equipment provides indication of normal background which is indicative of a basic level of leakage from primary systems and components. Any increase in the observed parameters is an indication of change within the containment and the equipment provided is capable of monitoring this change. The basic design criterion is the detection of deviations from normal containment environmental conditions including air particulate activity, radiogas activity, humidity, condensate runoff, and the liquid inventory in the process systems and containment sump A.

Further details are supplied in Section 5.2.5.

5.1.3.7 Reactor Coolant Pressure Boundary Capability

CRITERION: The reactor coolant pressure boundary shall be capable of accommodating without rupture the static and dynamic loads imposed on any boundary component as a result of an inadvertent and sudden release of energy to the coolant. As a design reference, this sudden release shall be taken as that which would result from a sudden reactivity insertion such as rod ejection (unless prevented by positive mechanical means), rod dropout, or cold water addition (AIF-GDC 33).

The reactor coolant boundary is shown to be capable of accommodating, without further rupture, the static and dynamic loads imposed as a result of a sudden reactivity insertion such as a rod ejection. The rod ejection accident is described in Section 15.4.5.

The operation of the reactor is such that the severity of an ejection accident is inherently limited. Since control rod clusters are used to control load variations only and core depletion is followed with boron dilution, only the rod cluster control assemblies in the controlling groups are inserted in the core at power; at full power these rods are only partially inserted. A rod insertion limit monitor is provided as an administrative aid to the operator to ensure that this condition is met.

By using the flexibility in the selection of control rod groupings, radial locations, and position as a function of load, the design limits the maximum fuel energy for the highest worth ejected rod to a value which precludes any resultant damage to the primary system pressure boundary, i.e., gross fuel dispersion in the coolant and possible excessive pressure surges.

The failure of a rod mechanism housing causing a control rod to be rapidly ejected from the core is evaluated as a theoretical, though not a credible accident. While limited fuel damage could result from this hypothetical event, the fission products are confined to the reactor coolant system and the reactor containment. The environmental consequences of rod ejection are less severe than from the postulated loss-of-coolant accident, for which public health and safety is shown to be adequately protected.

5.1.3.8 Reactor Coolant Pressure Boundary Rapid Propagation Failure Prevention

CRITERION: The reactor coolant pressure boundary shall be designed and operated to reduce to an acceptable level the probability of a rapidly propagating type failure. Consideration is given (A) to the provisions for control over service temperature and irradiation effects which may require operational restrictions, (B) to the design and construction of the reactor pressure vessel in accordance with applicable codes, including those which establish requirements for absorption of energy within the elastic strain energy range and for absorption of energy by plastic deformation and (C) to the design and construction of reactor coolant pressure boundary piping and equipment in accordance with applicable codes (AIF-GDC 34).

The reactor coolant pressure boundary is designed to reduce to an acceptable level the probability of a rapidly propagating type failure.

In the core region of the reactor vessel it is expected that the notch toughness of the material will change as a result of fast neutron exposure. This change is evidenced as a shift in the nil ductility transition temperature which is factored into the operating procedures in such a manner that full operating pressure is not obtained until the affected vessel material is above the now higher design transition temperature and in the ductile material region. The pressure during startup and shutdown at the temperature below nil ductility transition temperature is maintained below the threshold of concern for safe operation.

The design transition temperature is a minimum of nil ductility temperature plus 60°F and dictates the procedures to be followed in the hydrostatic test and in station operations to avoid excessive cold stress. The value of the design transition temperature is increased during the life of the plant as required by the expected shift in the nil ductility transition temperature and as confirmed by the experimental data obtained from irradiated specimens of reactor vessel materials during the plant lifetime. Further details are given in Sections 5.2 and 5.3.

All pressure-containing components of the reactor coolant system are designed, fabricated, inspected, and tested in conformance with the applicable codes. Further details are given in Section 5.2.1.2.

5.1.3.9 Reactor Coolant Pressure Boundary Surveillance

CRITERION: Reactor coolant pressure boundary components shall have provisions for inspection, testing, and surveillance of critical areas by appropriate means to assess the structural and leaktight integrity of the boundary components during their service lifetime. For the reactor vessel, a material surveillance program conforming with current applicable codes shall be provided (AIF-GDC 36).

The design of the reactor vessel and its arrangement in the system provides the capability for accessibility during service life to the entire internal surfaces of the vessel and certain external zones of the vessel including the nozzle to reactor coolant piping welds and the top and bottom heads. The reactor arrangement within the containment provides sufficient space for inspection of the external surfaces of the reactor coolant piping, except for the area of pipe within the primary shielding concrete.

Monitoring of the nil ductility transition temperature properties of the core region plates, forgings, weldments, and associated heat-treated zones are performed in accordance with ASTM E185 (Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors). Samples of reactor vessel forging materials are retained and cataloged in case future engineering development shows the need for further testing.

The material properties surveillance program includes not only the conventional tensile and impact tests but also fracture mechanics specimens. The fracture mechanics specimens are the wedge-opening loading-type specimens. The observed shifts in nil ductility transition temperature of the core region materials with irradiation will be used to confirm the calculated limits of startup and shutdown transients.

To define permissible operating conditions below design transient temperature, a pressure range is established which is bounded by a lower limit for pump operation and an upper limit

that satisfies reactor vessel stress criteria. To allow for thermal stresses during heatup or cooldown of the reactor vessel, an equivalent pressure limit is defined to compensate for thermal stress as a function of the rate of coolant temperature change. The reactor coolant temperature and pressure and system heatup and cooldown rates (with the exception of the pressurizer) are limited in accordance with the Pressure and Temperature Limits Report (PTLR). The allowable pressure-temperature relationships for the heatup and cooldown rates were developed using Regulatory Guide 1.99, Revision 2, and Appendix G of Section III of the ASME Boiler and Pressure Vessel Code and are discussed in the Technical Specifications and *Reference 1*.

For the pressurizer, the heatup and cooldown rates do not exceed 100°F per hr and 200°F per hr, respectively. An additional limitation is that spray cannot be used if the temperature difference between the pressurizer and the spray fluid is greater than 320°F.

Since the normal operating temperature of the reactor vessel is well above the maximum expected design transient temperature, brittle fracture during MODES 1 and 2 is not considered to be a credible mode of failure. A discussion of reactor vessel integrity under transient conditions is included in Sections 5.3.3.4 and 5.3.3.5.

5.1.3.10 Adequacy of Reactor Coolant System Design Relative to 1972 10 CFR 50, Appendix A, Criteria

The adequacy of the Ginna Station reactor coolant system design relative to the following General Design Criteria (GDC) is discussed in Section 3.1.2:

- GDC 14, Reactor Coolant Pressure Boundary.
- GDC 15, Reactor Coolant System Design.
- GDC 30, Quality of Reactor Coolant Pressure Boundary.
- GDC 31, Fracture Prevention of Reactor Coolant Pressure Boundary.
- GDC 32, Inspection of Reactor Coolant Pressure Boundary.
- GDC 34, Residual Heat Removal.

The use of the following Safety Guides is discussed in Section 1.8:

- Safety Guide 2, Thermal Shock to Reactor Pressure Vessels.
- Safety Guide 14, Reactor Coolant Pump Flywheel Integrity.

5.1.4 DESIGN CHARACTERISTICS

5.1.4.1 Design Pressure

The reactor coolant system design and operating pressure, together with the safety, power relief, and spray valves setpoints and the protection system setpoint pressures, are listed in Table 5.1-1. The design pressure allows for operating transient pressure changes. The selected design margin considers core thermal lag, coolant transport times and pressure drops, instrumentation and control response characteristics, and system relief valve characteristics.

Additional reactor coolant system piping and pressure drop data are listed in Tables 5.1-1 through 5.1-3.

5.1.4.2 Design Temperature

For each component, the design temperature is selected to be above the maximum coolant temperature under all normal and anticipated transient load conditions. The design and operating temperatures of the respective system components are discussed in Sections 5.3.2 and 5.4.

5.1.5 CYCLIC LOADS

All components in the reactor coolant system are designed to withstand the effects of cyclic loads due to reactor system temperature and pressure changes. These cyclic loads are introduced by normal unit load transients, reactor trips, and startup and shutdown operation. The number of thermal and loading cycles used for design purposes is shown in Table 5.1-4. During unit startup and shutdown, the rates of temperature and pressure changes are limited. The number of cycles for plant heatup and cooldown at 100°F/hr was selected as a conservative estimate based on an evaluation of the expected requirements. The resulting number, which averages five heatup and cooldown cycles per year, could be increased significantly; however, it is the intent to represent a conservative realistic number rather than the maximum allowed by the design.

Although loss of flow and loss of load transients are not included in the tabulation since the tabulation is only intended to represent normal design transients, the effects of these transients have been analytically evaluated and are included in the fatigue analysis for primary system components.

The reactor coolant system and its components are designed to accommodate 10% of full power step changes in plant load and 5% of full power per minute ramp changes over the range from 12.8% full power up to and including but not exceeding 100% of full power without reactor trip. The reactor coolant system will accept a complete loss of load from full power with reactor trip. In addition, the turbine bypass and steam dump system make it possible to accept a 50% rapid load reduction (200% per minute runback) from full power without a reactor trip for RCS full power T_{avg} values > 564.6°F. For RCS full power T_{avg} values greater than 570°F, a 50% step load reduction can be accommodated without a reactor trip or turbine trip. Additionally, a turbine trip below 50% power can be accepted without a reactor trip. The ability of the plant to withstand these plant transients at 1775 MWt was determined in Reference 2.

5.1.6 SERVICE LIFE

The service life of reactor coolant system pressure components depends upon the end-of-life material radiation damage, unit operational thermal cycles, quality manufacturing standards, environmental protection, and adherence to established operating procedures.

5.1.7 RELIANCE ON INTERCONNECTED SYSTEMS

The principal heat removal systems which are interconnected with the reactor coolant system are the steam and feedwater systems and the safety injection and residual heat removal systems. The reactor coolant system is dependent upon the steam generators and the steam, feedwater, and condensate systems for decay heat removal from normal operating conditions to a reactor coolant temperature of approximately 350°F. The layout of the system ensures the natural circulation capability to permit plant cooldown following a loss of all main reactor coolant pumps.

Flow diagrams of the steam and feedwater systems are shown in Drawings 33013-1231, 33013-1232, and 33013-1236. In the event that the condenser is not available to receive the steam generated by residual heat, the water stored in the feedwater system may be pumped into the steam generators and the resultant steam vented to the atmosphere. The preferred auxiliary feedwater system will supply water to the steam generators in the event that the main feedwater pumps are inoperative. The system is described in Section 10.5.

The safety injection system is described in Section 6.3. The residual heat removal system is described in Section 5.4.5.

5.1.8 SYSTEM INCIDENT POTENTIAL

The potential of the reactor coolant system as a cause of accidents is evaluated by investigating the consequences of certain credible types of component and control failures as discussed in Sections 15.1 through 15.4 and Section 15.6. Reactor coolant pipe rupture is evaluated in Section 15.6.4.

REFERENCES FOR SECTION 5.1

1. Westinghouse Electric Corporation, Rochester Gas and Electric Reactor Vessel Life Attainment Plan, March 1990.

2. Westinghouse Calculation Note, CN-SCS-05-1, "R.E. Ginna (RGE) 19.5% Uprate Program Plant Operability and Margin to Trip Analysis," Rev. 2.

3. Westinghouse Calculation Note, CN-PCWG-04-10, "Closeout of PCWG Open Items for Additional Best Estimate Performance Calculations based on Plant Data to Support the R.E. Ginna Unit 1 (RGE) Uprate Program," Rev. 0.

Table 5.1-1
REACTOR COOLANT SYSTEM PRESSURE SETTINGS

	Pressure (psig)
Design pressure	2485
Operating pressure	2235
Safety valves	2485
Power relief valves	2335
Spray valves (open)	2260
High-pressure trip	2377
High-pressure alarm	2310
Low-pressure trip	1873
Hydrostatic test pressure	3110

Table 5.1-2
REACTOR COOLANT PIPING DESIGN DATA

Reactor inlet piping, I.D., in.	27-1/2
Reactor outlet piping, I.D., in.	29
Coolant pump suction piping, I.D., in.	31
Pressurizer surge piping, in.[a]	10 - Schedule 140
Design/operating pressure, psig	2485/2235
Hydrostatic test pressure (cold), psig	3110
Design temperature, °F	650
Design temperature (pressurizer surge line), °F	680
Water volume, ft^3	552

a. Surge line fitted with a 14-in./10-in. adapter at the pressurizer.

Table 5.1-3
REACTOR COOLANT SYSTEM DESIGN PRESSURE DROP

	Pressure Drop (psi)[a]
Across pump discharge leg	1.25
Across vessel, including nozzles	42.60
Across hot leg	1.45
Across replacement steam generator	34.95
Across pump suction leg	2.85
Total pressure drop	83.10

a. Best estimate flow for 1775 MWt with T_{AVG}=573° and 0% SG tube plugging as calculated by *Reference 3*.

Table 5.1-4
THERMAL AND LOADING CYCLES

Transient Condition	Design Cycles[a]
Plant heatup at 100 °F/hr	200
Plant cooldown at 100 °F/hr	200
Plant loading at 5 % of full power per min[b]	6,460
Plant unloading at 5 % of full power per min[b]	6,460
Step load increase of 10 % of full power (but not to exceed full power)	2,000
Step load decrease of 10 % of full power	2,000
Step load decrease from 100 % to 50 % of full power	200
Partial loss of flow[c]	80
Loss of load[c]	80
Reactor trip	400
Hydrostatic test	
Pressure 3125 psia at 100 °F	5
Pressure 2500 psia at 400 °F	40

Steady-state fluctuations:

The reactor coolant average temperature for purposes of design is assumed to increase and decrease a maximum of 6 °F in 1 min. The corresponding reactor coolant pressure variation is less than 100 psig. It is assumed that an infinite number of such fluctuations will occur.

a. Estimated for equipment design purposes (40-year life) and not intended to be an accurate representation of actual transients or to reflect actual operating experience.

b. The number of cycles summarized for both plant loading and plant unloading at 5% of full power per minute is the most recent analyzed low-cycle fatigue results, as summarized in LTR-RIDA-12-149, based on analyses performed for the baffle-former bolt replacement/inspection campaign per ECP-10-000422.

c. Not an original Ginna design basis load. Included in uprate assessments performed by Westinghouse to be consistent with the list of design transients included in WCAP-14460.

5.2 INTEGRITY OF THE REACTOR COOLANT PRESSURE BOUNDARY

5.2.1 COMPLIANCE WITH CODES

5.2.1.1 System Integrity

The reactor coolant system serves as a barrier preventing radionuclides contained in the reactor coolant from reaching the atmosphere. In the event of a fuel cladding failure the reactor coolant system is the primary barrier against the uncontrolled release of fission products. By establishing a system pressure limit, the continued integrity of the reactor coolant system is ensured. Thus, the safety limit of 2735 psig (110% of design pressure) has been established. This represents the maximum transient pressure allowable in the reactor coolant system under the ASME Code, Section III, for MODES 1 and 2 and anticipated transient events. Reactor coolant system pressure settings are given in Table 5.1-1.

Release of activity into the reactor coolant in itself does not constitute a significant hazard. Activity in the coolant could constitute a significant hazard only if the reactor coolant system barrier is breached, and then only if the coolant contains excessive amounts of activity which could be released to the environment. The chemical and volume control system maintains primary reactor coolant activity within acceptable levels, as defined in the Technical Specifications.

A rupture of a steam generator tube would allow reactor coolant to enter the secondary system. In this event, a portion of the reactor coolant system gaseous activity could be released to the atmosphere. The radiological consequences of the event are discussed in Section 15.6.3.

As part of the design control on materials, Charpy V-notch toughness test curves were conducted for all ferritic material used in fabricating pressure parts of the reactor vessel and pressurizer to provide assurance for hydrotesting and operation in the ductile region at all times. For the replacement steam generators (RSGs) pressure boundary materials comply with ASME Section II and III requirements. The RT NDT (Reference Temperature for Nil-Ductility Transition Temperature) is used to specify the RSG material toughness. This temperature for each RSG pressure boundary plate, forging or weld is equal to or less than 0°F; typically these range from -70°F to -20°F. This provides assurance that these materials remain ductile for hydrotesting and operation at all times. In addition, drop-weight tests were performed on the reactor vessel material. Reactor vessel materials are discussed in Section 5.3.1. Reactor coolant pressure boundary materials are discussed in Section 5.2.3.

As an assurance of system integrity, all components in the system were hydro-tested at 3110 psig prior to initial operation.

As part of the Systematic Evaluation Program (SEP) the NRC evaluated, in part, the stresses in reactor coolant system components under normal and accident conditions. In the NRC Safety Evaluation Report (*Reference 1*) it was concluded that the control rod drive mechanism, reactor coolant pumps, steam generator and tube supports, and pressurizer and

reactor vessel supports were acceptably designed, with the stress analysis results within established limits.

5.2.1.2 Codes and Classifications

5.2.1.2.1 Code Requirements

All pressure-containing components of the reactor coolant system were originally designed, fabricated, inspected, and tested in conformance with the applicable codes listed in Table 5.2-1.

As part of the SEP, the codes, standards, and classifications to which the station was built were compared to current code requirements. It was generally concluded that changes between original and current code requirements do not affect the safety functions of the systems and components reviewed. Details of the review, which includes the reactor coolant system are presented in Section 3.2.

The reactor coolant system is classified as Seismic Category I, requiring that there will be no loss of function of such equipment in the event of the assumed maximum potential ground acceleration acting in the horizontal and vertical directions simultaneously, when combined with the primary steady state stresses.

Commencing in 1979, RG&E performed a reanalysis of Class I piping systems including the reactor coolant system for the seismic upgrade program. The analytical procedure used for the piping reanalysis is described in Section 3.7.3.7.5. The piping and thermal stresses were calculated using the formulas given in ANSI 31.1-1973, 1973 Summer Addenda requirements. The piping reanalysis is discussed in Section 3.9.2.1.8.

5.2.1.2.2 Quality Control

Quality control techniques used in the fabrication of the reactor coolant system were equivalent to those used in manufacture of the reactor vessel which conforms to Section III of the ASME Code.

Nuclear Piping Code B31.7 is derived from ASME III criteria. Thus, the added quality assurance requirements by Westinghouse to USAS B31.1.0-1967 procured reactor coolant piping ensured that the quality level of a Westinghouse plant was comparable to that of the Nuclear Piping Code USAS B31.7 as itemized below:

A. The material specifications were ASTM specifications approved for nuclear use in the various code cases.

B. The reactor systems materials were nondestructively examined to the levels required of Class A vessels - the same levels set forth in USAS B31.7.

C. Welding procedures and welders were required to be qualified to the requirements of Section IX of the ASME Code. The same requirement prevails in USAS B31.7.

D. All butt welds were examined to the same standards required in USAS B31.7.

E. All nozzle welds were required to be radiographically examined when the branch weld was in excess of 2-in. pipe size. This requirement exceeds that of USAS B31.7.

F. All nozzle, girth, and longitudinal welds were required to be liquid penetrant examined. This requirement is equivalent to USAS B31.7.

G. Hydrostatic testing was performed in completed systems. This requirement is equivalent to USAS B31.7.

5.2.1.2.3 Field Erection Procedures

Field erection and welding procedures were governed by Westinghouse specifications, which ensured that the field fabrication resulted in the same quality consistent with that exercised in the shop fabrication of the same piping. In these specifications for shop fabrication and field erection were references to portions of the ASME Code (Sections III, VIII, and IX), USAS Pressure Piping Code (B31.1) and Nuclear Code Cases N-7 and N-10, and ASTM Standards, as well as a number of Westinghouse documents.

During the erection, Westinghouse onsite personnel continually monitored all operations to ensure conformance to specifications, regulatory codes, and good construction practices. Adequate records are maintained onsite or at Westinghouse and include radiography reports and other nondestructive testing reports.

5.2.1.3 Seismic Loads

The seismic loading conditions were initially established by the design earthquake and maximum potential earthquake. The former was selected to be typical of the largest probable ground motion based on the site seismic history. The latter was selected to be the largest potential ground motion at the site based on seismic and geological factors and their uncertainties.

For the design earthquake loading condition, the nuclear steam supply system was designed to be capable of continued safe operation. Therefore, for this loading condition critical structures and equipment needed for this purpose are required to operate within normal design limits. The seismic design for the maximum potential earthquake was intended to provide a margin in design that ensures capability to shut down and maintain the nuclear facility in a safe condition. In this case, it was only necessary to ensure that the reactor coolant system components do not lose their capability to perform their safety function. This had come to be referred to as the no-loss-of-function criteria and the loading condition as the no-loss-of-function earthquake loading condition.

The analytical method employed in the design is described in Section 3.7 for Seismic Category I structures and components. The natural periods necessary for the determination of the loads were obtained by physical model testing.

The loading combinations and associated stress limits used for the piping systems which are part of the Seismic Piping Upgrade Program are discussed in Section 3.9.2.1.8.

The criteria adopted for allowable stresses and stress intensities in vessels and piping subjected to normal loads plus seismic loads are defined in Sections 3.9.2 and 5.4.11. These criteria ensure the integrity of the reactor coolant system under seismic loading.

For the combination of normal and design earthquake loadings, the stresses in the support structures are kept within the limits of the applicable codes.

For the combination of normal and no-loss-of-function earthquake loadings, the stresses in the support structures are limited to values as necessary to ensure their integrity and to contain the stresses in the reactor coolant system components within the allowable limits as previously established.

As part of the Ginna Station SEP the reactor coolant system has been reevaluated for the design-basis earthquake (safe shutdown earthquake) loadings wherein the ground acceleration is 0.2g. This reevaluation is discussed in Sections 3.7, 3.8, and 3.9.

5.2.2 OVERPRESSURIZATION PROTECTION

5.2.2.1 Normal Operation

During MODES 1, 2, and 3, the reactor coolant system is protected against overpressure by safety valves located on the top of the pressurizer. The safety valves on the pressurizer are sized to prevent system pressure from exceeding the design pressure by more than 10%, in accordance with Section III of the ASME Code. The capacity of the pressurizer safety valves is determined from considerations of (1) the Reactor Trip System (RTS) and (2) accident or transient conditions which may potentially cause overpressure.

The combined capacity of the safety valves is equal to or greater than the maximum surge rate resulting from complete loss of load without a direct reactor trip or any other control, except that the safety valves on the secondary plant are assumed to open when the steam pressure reaches the secondary plant safety valve setting. Details of the analysis are reported in Section 15.2.2. The pressurizer relief discharge system and safety valves are described in Sections 5.4.8.1 and 5.4.10.1.

5.2.2.2 Low Temperature Overpressure Protection (LTOP) System

Low temperature reactor vessel overpressure protection is provided by the two pressurizer power operated relief valves (PORVs) (Section 5.4.10) with a low-pressure setpoint as specified in the Pressure and Temperature Limits Report (PTLR). Whenever the reactor coolant system cold leg temperature is below the temperature setpoint specified for LTOP in the PTLR or the residual heat removal system is in operation, the low-pressure setpoint is manually enabled from the control room. Pressure transients caused by mass addition or heat addition are terminated below the limits of 10 CFR 50, Appendix G, as amended by ASME Code Case Cases N-640 and N-588, by automatic operation of the pressurizer power operated relief valves (PORVs). The system is designed to protect the reactor coolant system pressure boundary from the effects of operating errors during MODES 4, 5, and 6 (as applicable in the Technical Specifications) when the reactor coolant system is in a water-solid condition. The system also supplies protection for the residual heat removal system from overpressurization. The following sections give a more detailed discussion of the Low Temperature Overpressure Protection (LTOP) System.

5.2.2.2.1 Design Bases

The basic purpose of the Low Temperature Overpressure Protection (LTOP) System is to prevent reactor vessel pressure in excess of 10 CFR 50, Appendix G limits (ASME Code Cases N-640 and N-588). Specific criteria for system performance are:

5.2.2.2.3.1 Operator Action: No credit can be taken for operator action for 10 minutes after the operator is aware of a transient.

5.2.2.2.3.1 Single Failure: The system must be designed to relieve the pressure transient given a single failure in addition to the failure that initiated the pressure transient.

5.2.2.2.3.1 Testability: The system must be testable on a periodic basis consistent with the systems employment.

5.2.2.2.3.1 Seismic Criteria: The system safety function is met by equipment categorized as Seismic Category I. The basic objective is that the system should not be vulnerable to a common failure that would both initiate a pressure transient and disable the overpressure mitigating system. Such events as loss of instrument air and loss of offsite power must be considered.

Two kinds of pressure transients are considered:

1. Mass input transients from injection sources such as charging pumps, safety injection pumps, or safety injection accumulators.
2. Heat input transients from sources such as steam generators or decay heat.

On Westinghouse designed plants, a common cause of overpressure transients is isolation of the letdown path (letdown during low-pressure operations is via a flow path through the residual heat removal system). Thus, isolation of the residual heat removal system can initiate a pressure transient if a charging pump is left running. Although other transients occur with lower frequency, those which result in the most rapid pressure increases are of main concern. The most limiting mass input transient is the charging-letdown mismatch with three charging pumps left running with letdown completely isolated. The most limiting thermal expansion transient is the start of a reactor coolant pump with a 50°F temperature difference between the water in the reactor vessel and the water in the steam generator.

The NRC considers the pressurizer power operated relief valve (PORV) with a manually enabled low-pressure setpoint to be an acceptable overpressure mitigating system. Detailed information on system design is contained in *References 2 through 4*.

5.2.2.2.2 System Description

The "Reference Mitigating System" concept developed by Westinghouse and the Westinghouse Owner’s Group was originally adopted by Rochester Gas and Electric Corporation. This concept is acceptable to Ginna LLC. The actuation circuitry of the pressurizer power operated relief valves (PORVs) requires a low-pressure setpoint (setpoint provided in the Pressure and Temperature Limits Report (PTLR)) during startup and shutdown conditions. The low-pressure pressurizer power operated relief valve (PORV) actuation circuitry uses multiple pressure sensors, power supplies, and logic trains to improve system reliability.

Each of the two pressurizer power operated relief valves (PORVs) is manually enabled using two keylock switches, one to line up the nitrogen supply and the other to enable the low-pressure setpoint. When the reactor vessel is at low temperatures with the overpressure protection system enabled, a pressure transient is terminated below the Appendix G limit (ASME Code Cases N-640 and N-588) by automatic opening of the pressurizer power operated relief valves (PORVs). An enabling alarm monitors the reactor coolant system temperature, the position of the keylock switches (two per channel), and the upstream isolation valve position. The overpressure protection system is required to be in operation during plant cooldown prior to reaching the temperature limit specified in the PTLR or on initiation of the residual heat removal system and it is disabled prior to exceeding 350°F during plant heatup. The enabling alarm alerts the operator in the event the reactor coolant system temperature is below the PTLR temperature limit and overpressure protection system valve or switch alignment has not been completed.

The Ginna pressurizer power operated relief valves (PORVs) are spring closed and air or nitrogen opened. Each of the two pressurizer power operated relief valves (PORVs) receives actuating gas from either the plant instrument air system or a backup nitrogen accumulator; however, only nitrogen is used for Low Temperature Overpressure Protection (LTOP) conditions. The accumulators are sized to provide sufficient actuating nitrogen for 10 min of pressurizer power operated relief valve (PORV) operation (about 40 cycles) without operator action during the most limiting transient and a loss of the plant instrument air system. Low-pressure alarms are installed in the control room to alert the operator to a low nitrogen accumulator pressure condition. See Drawing 33013-1263. Performance of secondary side hydrostatic tests are permitted without the pressurizer power operated relief valves (PORVs) or a reactor coolant system vent ≥ 1.1 sq. in. operable, however, no safety injection pump may be capable of injecting into the reactor coolant system during the tests.

An alarm monitors the position of the pressurizer power operated relief valve (PORV) isolation valves (515 and 516), along with the low setpoint enabling switch, to ensure that the overpressure mitigating system is properly aligned for shutdown conditions. An overpressure alarm which incorporates two setpoints is also provided. One setpoint is variable and follows the limit specified in the PTLR. The other setpoint alarms at a preprogrammed differential pressure. Both setpoints alarm and light on the plant process computer system.

The installed pressure and temperature instrumentation at Ginna Station will provide a permanent record over the full range of both pressure and temperature.

5.2.2.2.3 System Evaluation

5.2.2.2.3.1 General

Generic Letter 88-11, "NRC Position on Radiation Embrittlement of Reactor Vessel Materials and Its Impact on Plant Operations," required each licensee to reevaluate the effect of neutron radiation on reactor vessel material using the methods described in Regulatory Guide 1.99, Revision 2. The pressure-temperature limits resulting from the implementation of Regulatory Guide 1.99, Revision 2, required the reevaluation of the pressurizer power operated relief valve (PORV) setpoint. The setpoint reevaluation was performed by Westinghouse and is documented in *Reference 5*. This evaluation was superseded by *Reference 15* which incorporated the characteristics of the replacement steam generators, and the approval to use

ASME Code Case N-514 for Ginna. The use of ASME Code Case N-514 was disallowed by License Amendment 106. ASME Code Cases N-640 and N-588 are now used in place of Code Case N-514. The Appendix G limits were updated in *Reference 29*, and incorporated into *Reference 15*.

The Low Temperature Overpressure Protection system (LTOP) transient analyses were performed using the RELAP5/MOD2 B&W Version 20 (*Reference 16*) computer code. The plant model that was employed for the LTOP analyses is described in *Reference 15*.

For the limiting mass addition case, the primary system was initialized at 60°F and 315 psig with two reactor coolant pumps running. The primary and secondary systems were decoupled since there was no heat transfer in this case. The event was initiated by starting three charging pumps with a total capacity of 180 gpm. The analysis was terminated after 10 minutes when the operator was assumed to secure charging flow. The peak RCS pressure was compared with the acceptance criteria.

For the heat addition cases, the primary system was initialized to isothermal conditions with no reactor coolant flow. The secondary and primary fluid in the steam generators were initialized at a temperature 50 degrees above the primary system. The transient was initiated by starting a reactor coolant pump in the loop that contains the pressurizer. The analysis was run until the peak pressure was obtained. The peak pressures in the reactor vessel and the RHR system were compared with the acceptance criteria.

5.2.2.2.3.2 Mass Addition Case

The mass addition case was initialized at a primary temperature of 60°F and a primary pressure of 315 psig. Using the initial pressure of 315 psig assures that the transient is well defined by the time the power operated relief valve (PORV) is actuated. Two reactor coolant pumps were assumed running and the pressurizer was water solid. It was assumed that the residual heat removal (RHR) system was removing decay heat, so it was conservatively not modeled. The event was initiated by starting three pump charging flow (180 gpm or 25 lb/sec). The analysis was run for ten minutes. The sequence of events for this case is shown in Table 5.2-7.

The peak reactor vessel pressure was 587.4 psia. The allowable pressure, according to ASME Code Cases N-640 and N-588 at 60°F is 621 psig or 635.7 psia. Therefore, there is 48.3 psi margin to the Appendix G acceptance criterion.

To compare the peak pressure in the RHR system with the acceptance criterion, the pressure drop from the hot leg to the RHR pump discharge was added to the peak hot leg pressure. This case yielded a peak RHR pressure of 663.5 psia. The peak allowable pressure in the RHR system is 674.7 psia. This results in a 11.2 psi margin to the acceptance criterion.

5.2.2.2.3.3 Heat Addition at 60°F

The most limiting heat addition case was analyzed at a primary system temperature of 60°F and a primary pressure of 315 psig. The secondary system was assumed to be 50 degrees hotter than the primary system, so the temperature in the secondary system and the primary side of the steam generator was 110°F. Initially, the reactor coolant pumps were not running

and cooling was assumed to be provided by the residual heat removal (RHR) system. The RHR system was operating. The pressurizer was water solid. There was no charging flow for this event. The event was initiated by starting the reactor coolant pump in the loop that contained the pressurizer. The transient was analyzed for 40 seconds. The sequence of events for this case is shown in Table 5.2-8.

The peak pressure in the reactor vessel for this case was 551.3 psia. The allowable pressure limit at this temperature is 635.7 psia. This yields a 84.5 psi margin. This case is the most limiting for Appendix G. (ASME Code Cases N-640 and N-588)

The peak pressure in the RHR System was 650.0 psia as compared with an acceptance criterion of 674.7 psia, for a margin of 24.7.

5.2.2.2.3.4 Heat Addition at 320°F

The heat addition case was also analyzed with steam generator secondary system temperatures of 370°F. This temperature is the maximum temperature, including instrument uncertainty, at which both reactor coolant pumps can be stopped. The primary system was assumed to be 50 degrees colder than the secondary system, so the temperature in the primary system was 320°F. Initially, the reactor coolant pumps were not running and cooling was assumed to be provided by the residual heat removal (RHR) system. The RHR system was operating. The pressurizer was water solid. There was no charging flow for this event. The transient was initiated by starting the reactor coolant pump in the loop that contained the pressurizer.

The peak pressure in the reactor vessel for this case was 563.8 psia. The allowable reactor vessel pressure limit at this temperature is > 2400 psia. This yields a > 1836 psi margin.

The peak pressure in the RHR system was 655.7 psia as compared with an acceptance criterion of 674.7 psia, for a margin of 19.0 psia.

5.2.2.2.3.5 Administrative Controls

To limit the magnitude of postulated pressure transients to within the bounds of the analysis, a defense-in-depth approach is adopted using administrative controls. Specific conditions required to ensure that the plant is operated within the bounds of the analysis are described in the bases for Technical Specification LCO 3.4.12.

A number of provisions for prevention of pressure transients are also contained in the Ginna operating procedures. These procedures require that an acceptable reactor coolant system temperature profile be achieved prior to startup of a reactor coolant pump with the reactor coolant system in a water-solid condition. In addition, plant shutdown and cooldown procedures call for one reactor coolant pump to be run until the reactor coolant system temperature has been lowered to 160°F, thus reducing the possibility of a significant reactor coolant system temperature asymmetry.

Also, plant procedures restrict water-solid operations to only those times when absolutely necessary. For example, the plant must be maintained in a water-solid condition during reactor coolant system filling and venting operations, during hydrostatic testing of the reactor

coolant system, and during plant heatup prior to bringing the reactor coolant system within water chemistry specifications.

The cooldown procedures require the safety injection signal associated with the pressurizer and steam line low pressure be blocked at approximately 2000 psig. At less than 350°F psig, the high-head safety injection discharge valves to the reactor coolant system loops are shut and the high-head safety injection pumps are deenergized by placing their control switches in the "pull-stop" position. In the "pull-stop" position the safety injection pumps cannot automatically start. The safety injection pumps are not reenergized while the reactor coolant system is in a cold and shutdown condition unless special surveillance testing is in progress or a safety injection accumulator is to be filled when only one safety injection pump is energized.

The diesel-generator load and safeguards sequence test conducted during cold or MODE 6 (Refueling) shutdown operates each safeguards train (two pumps). However, the pump discharge valves are closed, the valve power supply breakers are open, and the breaker dc control fuses are removed. During other tests the safety injection pumps are prohibited from starting and, except during valve cycling tests, the discharge valves are shut.

5.2.2.2.4 Tests and Inspections

Operability of the Low Temperature Overpressure Protection (LTOP) System is verified prior to solid system, low temperature operation by use of the remotely operated isolation valve, and the enable/disable switches. The actuation circuitry is tested each MODE 6 (Refueling) outage. Testing requirements are included in the Technical Specifications.

5.2.3 REACTOR COOLANT PRESSURE BOUNDARY MATERIALS

5.2.3.1 Material Specifications

Each of the materials used in the reactor coolant system is selected for the expected environment and service conditions. The major component materials are listed in Table 5.2-2.

5.2.3.1.1 Nondestructive Examination of Materials and Components Prior to Operation

5.2.3.1.1.1 Quality Assurance Program

Table 5.2-3 summarizes the initial quality assurance program for all reactor coolant system components. In this table, all of the nondestructive tests and inspections required by Westinghouse specifications on reactor coolant system components and materials are specified for each component. All tests required by the applicable codes are included in this table. Westinghouse requirements, which were more stringent in some areas than those requirements specified in the applicable codes, are also included.

Table 5.2-3 also summarizes the quality assurance program with regard to inspections performed on primary system components. In addition to the inspections shown in Table 5.2-3, there were those that the equipment supplier performed to confirm the adequacy of material received, and those performed by the material manufacturer in producing the basic material. The inspections of reactor vessel, pressurizer, and steam generator were governed by ASME Code requirements. The inspection procedures and acceptance standards required

on pipe materials and piping fabrication were governed by USAS B31.1 and Westinghouse requirements and were equivalent to those performed on ASME Code vessels.

Procedures for performing the examinations were consistent with those established in the ASME Code, Section III, and were reviewed by qualified Westinghouse engineers. These procedures were developed to provide the highest assurance of quality material and fabrication. They considered not only the size of the flaws, but equally as important, how the material was fabricated, the orientation and type of possible flaws, and the areas of most severe service conditions. In addition, the surfaces most subject to damage as a result of the heat treating, rolling, forging, forming, and fabricating processes, received a 100% surface inspection by magnetic particle or liquid penetrant testing after all these operations were completed. Although flaws in plates are inherently laminations in the center, all reactor coolant plate material is subject to shear as well as longitudinal ultrasonic testing to give maximum assurance of quality. (All forgings received the same inspection.) In addition, 100% of the material volume was covered in these tests as added assurance over the grid basis required in the code.

Westinghouse quality control engineers and RG&E engineers monitored the supplier’s work, and witnessed key inspections not only in the supplier’s shop but in the shops of subvendors of the major forgings and plate material. Normal surveillance included verification of records of material, physical and chemical properties, review of radiographs, performance of required tests, and qualification of supplier personnel.

5.2.3.1.1.2 Welding and Heat Treatment

Equipment specifications for fabrication required that suppliers submit the manufacturing procedures (welding, heat treating, etc.) to Westinghouse where they were reviewed by qualified Westinghouse engineers. This also was done on the field fabrication procedures to ensure that installation welds were of equal quality.

Section III of the ASME Code required that nozzles carrying significant external loads be attached to the shell by full penetration welds. This requirement was carried out in the reactor coolant piping, where all auxiliary pipe connections to the reactor coolant loop were made using full penetration welds.

Preheat requirements, nonmandatory under code rules, were performed on all weldments, including P1 and P3 materials which were the materials of construction in the reactor vessel, pressurizer, and steam generators. Preheat and postheat of weldments both serve a common purpose: the production of tough, ductile metallurgical structures in the completed weldment.

Preheating produces tough ductile welds by minimizing the formation of hard zones, whereas postheating achieves this by tempering any hard zones which may have formed due to rapid cooling. Thus, the reactor coolant system components were welded under procedures that required the use of both preheat and postheat.

5.2.3.1.2 Quality Assurance for Electroslag Welds

5.2.3.1.2.1 Piping Elbows

The 90-degree primary system elbows were electroslag welded. The following efforts were performed for quality assurance of these components:

a. The electroslag welding procedure employing one-wire technique was qualified in accordance with the requirements of ASME Code, Section IX and Code Case 1355, plus supplementary evaluations as requested by Westinghouse. The following test specimens were removed from a 5-in.-thick weldment and successfully tested. They were:

   1. Six transverse tensile bars - as welded.
   2. Six transverse tensile bars - 2050°F, H_2O quench.
   3. Six transverse tensile bars - 2050°F, H_2O quench + 750°F stress relief heat treatment.
   4. Six transverse tensile bars - 2050°F, H_2O quench, tested at 650°F.
   5. Twelve guided side bend test bars.

b. The casting segments were surface conditioned for 100% radiographic and penetrant inspections. The acceptance standards were ASTM E-186 severity level 2 (except no category D or E defectiveness was permitted) and USAS Code Case N-10, respectively.

c. The edges of the electroslag weld preparations were machined. These surfaces were penetrant inspected prior to welding. The acceptance standards were USAS Code Case N-10.

d. The completed electroslag weld surfaces were ground flush with the casting surface. Then the electroslag weld and adjacent base material were 100% radiographed in accordance with ASME Code Case 1355. Also, the electroslag weld surfaces and adjacent base material were penetrant inspected in accordance with USAS Code Case N-10.

e. Weld metal and base metal chemical and physical analyses were determined and certified.

f. Heat treatment furnace charts were recorded and certified.

5.2.3.1.2.2 Reactor Coolant Pump Casings

The Ginna reactor coolant pump casings were electroslag welded. The following efforts were performed for quality assurance of the components.

The electroslag welding procedure employing two-wire and three-wire techniques was qualified in accordance with the requirements of the ASME Code, Section IX and Code Case 1355, plus supplementary evaluations as requested by Westinghouse. The following test specimens were removed from an 8-in.-thick and from a 12-in.-thick weldment and successfully tested for both the two-wire and the three-wire techniques, respectively. They were as follows.

a. Two-wire electroslag process - 8-in.-thick weldment.

   1. Six transverse tensile bars - 750°F postweld stress relief.
   2. Twelve guided side bend test bars.

b. Three-wire electroslag process - 12-in.-thick weldment.
    1. Six transverse tensile bars - 750°F postweld stress relief.
    2. Seventeen guided side bend test bars.
    3. Twenty-one Charpy V-notch specimens.
    4. Full section macroexamination of weld and heat affected zone.
    5. Numerous microscopic examinations of specimens removed from the weld and heat affected zone regions.
    6. Hardness survey across weld and heat affected zone.

c. A separate weld test was made using the two-wire electroslag technique to evaluate the effects of a stop and restart of welding by this process. This evaluation was performed to establish proper procedures and techniques as such an occurrence was anticipated during production applications due to equipment malfunction, power outages, etc. The following test specimens were removed from an 8-in.-thick weldment in the stop-restart-repaired region and successfully tested.
    1. Two transverse tensile bars - as welded.
    2. Four guided side bend test bars.
    3. Full section macroexamination of weld and heat affected zone.

d. All of the weld test blocks in items a, b, and c above were radiographed using a 24-MeV betatron. The radiographic quality level obtained was between 0.5% to 1% (1-1T). There were no discontinuities evident in any of the electroslag welds.
    1. The casting segments were surface conditioned for 100% radiographic and penetrant inspections. The radiographic acceptance standards were ASTM E-186 severity level 2 (except no category D or E defectiveness was permitted for section thickness up to 4.5 in.) and ASTM E-280 severity level 2 for section thicknesses greater than 4.5 in. The penetrant acceptance standards were ASME Code, Section III, paragraph N-627.
    2. The edges of the electroslag weld preparations were machined. These surfaces were penetrant inspected prior to welding. The acceptance standards were ASME Code, Section III, paragraph N-627.
    3. The completed electroslag weld surfaces were ground flush with the casting surface. Then, the electroslag weld and adjacent base material were 100% radiographed in accordance with ASME Code Case 1355. Also, the electroslag weld surfaces and adjacent base material were penetrant inspected in accordance with ASME Code, Section III, paragraph N-627.
    4. Weld metal and base metal chemical and physical analyses were determined and certified.
    5. Heat treatment furnace charts were recorded and certified.

5.2.3.1.2.3 Reactor Coolant Pump Field Erection and Welding

Field erection and field welding of the reactor coolant system were performed so as to permit exact fit-up of the 31-in. I.D. closure pipe subassemblies between the steam generator and the reactor coolant pump. After installation of the pump casing and the steam generator, measurements were taken of the pipe length required to close the loop. Based on these measurements, the 31-in. I.D. closure pipe subassembly was properly machined and then erected and field welded to the pump suction nozzle and to the steam generator exit nozzle. Thus, upon completion of the installation, the system was essentially of zero stress in the installed position.

Cleaning of reactor coolant system piping and equipment was accomplished before and/or during erection of various equipment. Stainless steel piping was cleaned in sections as specific portions of the systems were erected. Pipe and units large enough to permit entry by personnel were cleaned by locally applying approved solvents (Stoddart solvent, acetone, and alcohol) and demineralized water, and by using a rotary disk sander or 18-8 wire brush to remove all trapped foreign particles. Standards for final physical and chemical cleanliness are defined in Section 14.1.1.2.2.

5.2.3.2 Compatibility With Reactor Coolant

All reactor coolant system materials that are exposed to the coolant are corrosion resistant. They consist of stainless steels and Inconel, and they are chosen for specific purposes at various locations within the system for their superior compatibility with the reactor coolant.

All external insulation of reactor coolant system components is compatible with the component materials. The cylindrical shell exterior and closure flanges to the reactor vessel are insulated with metallic reflective insulation. The closure head is insulated with low halidecontent insulating material. All other external corrosion resistant surfaces in the reactor coolant system are insulated with low or halide-free insulating material as required.

The water chemistry is selected to provide the necessary boron content for reactivity control and to minimize corrosion of the reactor coolant system surface. Periodic analyses of the coolant chemical composition are performed to monitor the adherence of the system to the reactor coolant water quality listed in plant procedures. Concentration limits of lithium and lithium hydroxide as a function of boron concentration are determined from plant procedures. Maintenance of the water quality to minimize corrosion is performed by the chemical and volume control system and sampling system, which are described in Sections 9.3.4 and 9.3.2.

Generic Letter 88-05 (*Reference 6*) directed PWR licensees to have a program that addresses the corrosive effects of reactor coolant system leakage below Technical Specifications limits wherein the coolant containing dissolved boric acid comes in contact with and degrades low alloy carbon steel components. The concern is that concentrated boric acid solution or boric acid crystals, formed by evaporation of water from the leaking reactor coolant, is more corrosive than the coolant and will corrode the reactor coolant pressure boundary. The boric acid corrosion prevention program at Ginna Station addresses both reactor coolant system leaks and leaks from other systems containing boric acid that may contact any reactor

coolant system carbon steel components. The program meets the intent of Generic Letter 88-05 (*Reference 7*).

5.2.4 INSERVICE INSPECTION AND TESTING OF THE REACTOR COOLANT SYSTEM PRESSURE BOUNDARY

5.2.4.1 Inservice Inspection Program

The Inservice Inspection Program for Ginna Station is designed to verify that the structural integrity of the reactor coolant pressure boundary is maintained throughout the life of the station. The program is scheduled for 10-year inspection intervals. The current 10-year inspection interval is specified in the Inservice Inspection Program document.

The inservice inspection program for the reactor vessel includes a visual examination of accessible internal surfaces, nozzles, and internal components of the reactor vessel and ultrasonic examinations of the vessel welds. The program is performed in accordance with Ginna Station procedures. The inservice inspection program for steam generator tubes was developed to meet the Ginna Technical Specifications, and the requirements of the Electric Power Research Institute PWR Steam Generator Program Guidelines. The program is described in the Ginna Station Engineering procedures. Special reporting requirements are described in the Steam Generator Program. The inservice inspection program for the reactor coolant pump flywheels was developed to meet the guidance provided in Regulatory Guide 1.14, Revision 1. The program is also described in the Ginna Station procedures, and is specified in the Inservice Inspection (ISI) Program document.

NRC Bulletin 88-09 (*Reference 8*) requested licensees to establish an inspection program to monitor thimble tube performance because of recently identified thimble tube thinning and leakage. Since no inservice inspection or testing requirements for thimble tubes existed, the NRC believed that this may have resulted in significant thimble tube degradation having gone undetected, creating a condition that may be adverse to safety. To comply with NRC Bulletin 88-09, a "Thimble Tube Inspection Program" has been established to ensure that the acceptance criterion of 65% through-wall wear is not exceeded and that appropriate corrective action is performed for any tube whose inspection indicates equal to or greater than 55% through-wall in the wear area as documented in *Reference 9*.

5.2.4.2 Inspection Areas and Components

5.2.4.2.1 Accessible Components and Areas

The following components and areas are available and accessible for visual and/or nondestructive examination:

1. Reactor vessel.
    a. Longitudinal and circumferential shell welds.
    b. Circumferential welds in bottom head. Replacement reactor vessel closure head provided by PCR 2001-0042 is a one piece forging.
    c. Vessel-to-flange circumferential welds.

   d. Primary nozzle-to-vessel welds and inside nozzle section.
   e. Penetrations, including control rod drive and instrumentation penetrations.
   f. Nozzle-to-safe-end welds.
   g. Closure head studs, nuts, washers, and pressure retaining bolts.
   h. Integrally welded attachments.
   i. Interior surface.
   j. Core support structures.
   k. Control rod drive housings.

2. Pressurizer.
   a. Longitudinal and circumferential welds.
   b. Nozzle-to-vessel welds and nozzle-to-vessel radiused section.
   c. Heater penetrations.
   d. Nozzle-to-safe-end welds.
   e. Bolts, studs, and nuts.
   f. Integrally welded attachments.

3. Steam Generators.
   a. Longitudinal and circumferential welds, including tubesheet-to-head or shell welds on the primary side.
   b. Nozzle-to-safe-end welds.
   c. Bolts, studs, washers, and nuts.
   d. Integrally welded attachments.
   e. Tubing.

4. Reactor Coolant Pumps.
   a. Pump casing welds.
   b. Supports.
   c. Bolts, studs, and nuts.
   d. Integrally welded attachments.
   e. Flywheel.

5. Pressure Boundary Piping.
   a. Safe-end to piping welds and safe-end in branch piping welds.
   b. Circumferential and longitudinal pipe welds.
   c. Branch pipe connection welds.
   d. Socket welds.

e. Supports.

f. Bolts, studs, and nuts.

g. Integrally welded attachments.

6. Pressure Boundary Valves.

a. Valve-body welds.

b. Supports.

c. Bolts, studs, and nuts.

d. Integrally welded attachments.

5.2.4.2.2 Accessible Areas During Refueling

The internal surface of the reactor vessel is inspected periodically using optical devices over the accessible areas. During refueling, the vessel cladding can be inspected in certain areas between the closure flange and the primary coolant inlet nozzles and, if deemed necessary by this inspection, the core barrel could be removed making the entire inside vessel surface accessible. Ultrasonic testing methods are employed as required. In order to facilitate this test program, critical areas of the reactor vessel were mapped during the fabrication phase to serve as a reference base for subsequent ultrasonic tests.

Externally, the control rod drive mechanism nozzles on the closure head, the instrument nozzles on the bottom of the vessel, and the extension spool pieces on the primary coolant outlet nozzles are accessible for visual, magnetic particle, or dye penetrant inspection during refuelings.

The closure head is examined visually during each refueling. Optical devices permit a selective visual inspection of the cladding, control rod drive mechanism nozzles, and the gasket seating surface. The knuckle transition piece, which is the area of highest stress of the closure head, also is accessible on the outer surface for inspection by visual and dye penetrant means.

The closure studs are inspected periodically using magnetic particle tests and/or ultrasonic tests. Additionally, it is possible to perform strain tests during the tensioning, which assists in verifying the material properties.

5.2.4.3 Accessibility

The considerations that are incorporated into the reactor coolant system design to permit these inspections are as follows:

A. All reactor internals are completely removable. The tools and storage space required to permit these inspections are provided.

B. The closure head is stored dry on the reactor operating deck during MODE 6 (Refueling) to facilitate direct visual inspection.

C. All reactor vessel studs, nuts, and washers are removed to dry storage during refueling.

D. Removable plugs are provided in the primary shield just above the coolant nozzles, and the insulation covering the nozzle welds is readily removable.

E. Access holes are provided in the lower internals barrel flange to allow remote access to the reactor vessel internal surfaces between the flange and the nozzles without removal of the internals.

F. A removable plug is provided in the lower core support plate to allow access for inspection of the bottom head without removal of the lower internals.

G. The storage stands that are provided for storage of the internals allow for inspection access to both the inside and outside of the structures.

H. The station that is provided for changeout of control rod clusters from one fuel assembly to another is especially designed to allow inspection of both fuel assemblies and control rod clusters.

I. The control rod mechanism is especially designed to allow removal of the mechanism assembly from the reactor vessel head.

J. Manways are provided in the steam generator, steam drum, and channel head to allow access for internal inspection.

K. A manway is provided in the pressurizer top head to allow access for internal inspection.

L. Insulation on the primary system components (except the reactor vessel) and piping (except for the penetration in the primary shield) included in the inservice inspection program is removable.

5.2.4.4 Examination Methods

The reactor coolant pressure boundary areas and components identified in Section 5.2.4.2 will be examined by the required visual, surface, or volumetric methods. These examinations will include one or a combination of visual, liquid penetrant, magnetic particle, ultrasonic, eddy current, or radiographic examination. These methods will be in accordance with the rules of IWA-2000 of the ASME Code, Section XI as specified in the Inservice Inspection (ISI) Program document.

Steam generator tubes will be examined by a volumetric method (e.g., eddy-current) or an alternative acceptable method. In response (*References 12* and *13*) to Generic Letter 95-03 (*Reference 14*), RG&E provided information to the NRC about techniques which were used (and will be used) in the performance of eddy-current testing of the replacement steam generators. In response (*References 18* and *19*) to Generic Letter 97-05 (*Reference 20*), RG&E provided additional information to the NRC regarding steam generator inspection techniques used at Ginna Station.

Reactor coolant pump flywheels will be examined by the required surface and volumetric methods in accordance with the requirements of IWA-2200 of the ASME Code, Section XI.

The edition and addenda of the ASME Code sections cited in UFSAR Sections 5.2.4.4 through 5.2.4.8 are as specified in the Inservice Inspection (ISI) Program document.

In 1981, RG&E performed a 10-year inservice inspection of the reactor coolant pump bowl successfully utilizing the portable radiographic linear accelerator prototype MINAC, developed by the Electric Power Research Institute, and a manipulator/control system developed by RG&E. The system was placed onto the reactor coolant pump and a radiographic examination was made of the middle weld (ranging in thickness from 5 in. to 9 in.), bottom weld (ranging in thickness from 8.5 in. to 9 in.), and the top weld (ranging in thickness from 10.25 in. to 10.5 in.). A sensitivity level of 1T was obtained in most exposures and all radiographs were acceptable. Video enhancement equipment was used in conjunction with the MINAC head-mounted camera during the visual examination of the inside surface of the welds and also as an aid to verify the position of the ground weld and MINAC head alignment for each of the exposures of the three welds.

5.2.4.5 Evaluation of Examination Results

The evaluation of nondestructive examination results will be in accordance with Article IWB-3000 of the ASME Code, Section XI and the Inservice Inspection (ISI) Program document. All reportable indications will be subject to comparison with previous data to aid in their characterization and in determining their origin.

The evaluation of the nondestructive examination results from the steam generator tube examination will dictate certain action in terms of resumption of operation and corrective measures, depending on the type and extent of degradation. Specific criteria are included in the Steam Generator Program.

5.2.4.6 Repair Requirements

Repair of reactor coolant pressure boundary components will be performed in accordance with the applicable subsections of the ASME Code, Section XI. Examinations associated with repairs or replacements will meet the applicable design and code requirements described in the Inservice Inspection (ISI) Program document.

Repair of steam generator tubes that have unacceptable defects will be performed by using a tube plugging technique or sleeving. Steam generator tube and sleeve repair criteria are included in the Technical Specifications and in the Inservice Inspection Program. Repair of a reactor coolant pump flywheel that has unacceptable defects will be performed in accordance with Regulatory Guide 1.14, Revision 1 and the Inservice Inspection (ISI) Program document.

5.2.4.7 Pressure Testing

The reactor coolant system pressure test will be conducted in accordance with the Inservice Inspection (ISI) Program document.

5.2.4.8 Exemptions

In accordance with paragraphs IWB-1220 and IWC-1220 of the ASME Code, Section XI, components may be exempt from examinations where certain conditions exist. Detailed descriptions of the exemptions at Ginna Station appear in the Inservice Inspection (ISI) Plan document. The majority of exemptions cover areas where a later edition of the ASME Code

provides better assurance and is more practicable; in these cases relief from the earlier version has been approved by the NRC.

5.2.5 DETECTION OF LEAKAGE THROUGH REACTOR COOLANT PRESSURE BOUNDARY

5.2.5.1 Leakage Detection Methods

The existence of leakage from the reactor coolant system to the containment, regardless of the source of leakage, is detected by one or more of the following conditions:

A. Two radiation sensitive instruments provide the capability for detection of leakage from the reactor coolant system. The containment air particulate monitor (R-11) is quite sensitive to low leak rates. The rate of leakage to which the instrument is sensitive is 0.018 gpm within 20 minutes, assuming the presence of noble gas progeny. The containment radiogas monitor (R-12) is much less sensitive but can be used as a backup to the air particulate monitor. The sensitivity range of the instrument is approximately 7 gpm within 1 hour. Operability of both monitors is addressed in the Technical Specifications.

B. An increase in containment sump A level (LT-2039 and LT-2044) and sump pump actuation monitoring are means of detecting increases in unidentified leakage and can measure approximately a 2.0 gpm leak in 1 hour. Operability of these monitors is addressed in the Technical Specifications.

C. An increase in the amount of coolant makeup water which is required to maintain normal level in the pressurizer is apparent from monitoring the volume control tank level.

D. A leakage detection system is installed which determines leakage losses from all water and steam systems within the containment including that from the reactor coolant system. This system collects and measures moisture condensed from the containment atmosphere by the cooling coils of the containment recirculation fan cooler (CRFC) units. It relies on the principle that all leakages up to sizes permissible with continued plant operation will be evaporated into the containment atmosphere. This system provides a dependable and accurate means of measuring integrated total leakage, including leaks from the cooling coils themselves which are part of the containment boundary. This system can detect leakage from approximately 1 gpm to 30 gpm within 1 hour.

E. Other alternative instruments used in leak detection are the humidity detectors. These provide a backup means of measuring overall leakage from all water and steam systems within the containment but furnish a less sensitive measure. The humidity monitoring method provides backup to the radiation monitoring methods. The sensitivity range of these instruments is from approximately 2 gpm to 10 gpm.

F. Additional indication of leakage can be obtained from the containment atmosphere temperature (TE-6031, 6035, 6036, 6037, 6038, and 6045) and pressure (PI-944) monitors.

Table 5.2-5 lists the leakage-detection systems available to monitor reactor coolant pressure boundary leakage to the containment. Table 5.2-6 lists the leakage-detection systems used for intersystem leakage.

5.2.5.2 Leakage Limitations

Reactor coolant system components are manufactured to exacting specifications which exceed normal code requirements (as outlined in Section 5.2.1.2). In addition, because of the welded construction of the reactor coolant system and the extensive nondestructive testing to which it is subjected (as outlined in Section 5.2.3), it is considered that leakage through metal surfaces or welded joints is very unlikely.

However, some leakage from the reactor coolant system is permitted by the reactor coolant pump seals. Also, all sealed joints are potential sources of leakage even though the most appropriate sealing device is selected in each case. Thus, because of the large number of joints and the difficulty of ensuring complete freedom from leakage in each case, a small integrated leakage is considered acceptable.

The Electric Power Research Institute (EPRI) established a program in 1984 that identified two improvements in valve stem packing to reduce leakage. These included replacement of woven asbestos packing with die-formed flexible graphite and the addition of live (spring) loading of packing gland followers. Ginna Station modified the valve stem packing of several valves to include these improvements.

Leakage from the reactor coolant system is collected in the containment or by other closed systems. These closed systems are the steam and feedwater system, the waste disposal system, and the component cooling water (CCW) system. Assuming the existence of the maximum allowable activity in the reactor coolant (see the Technical Specifications), the rate of 1 gpm unidentified leakage, also given in the Technical Specifications, is a conservative limit on what is allowable before the guidelines of 10 CFR 20 would be exceeded. This is shown as follows. If the reactor coolant activity is $100/\bar{E}$ $\mu Ci/cm^3$-MeV ($\bar{E}$ = average beta + gamma energy per disintegration in MeV) and 1 gpm of primary system leakage is assumed to be discharged through the air ejector, the yearly whole-body dose resulting from this activity at the site boundary, using an annual average X/Q = 2.63×10^{-6} sec/m^3, is 0.024 R/yr as compared with the 10 CFR 20 guideline of 0.5 R/yr.

With the limiting reactor coolant activity and assuming initiation of a 1-gpm leak from the reactor coolant system to the component cooling system, the radiation monitor in the component cooling system would annunciate in the control room and initiate closure of the vent line from the surge tank in the component cooling system, within less than 1 minute. In the case of failure of the closure of the vent line and resulting continuous discharge to the atmosphere via the component cooling surge tank vent, the resultant dose at the site boundary would be
0.024 R/yr.

Leakage directly into the containment indicates the possibility of a breach in the coolant envelope. The limitation of 1 gpm for a source of leakage not identified is sufficiently above the minimum detectable leakage rate to provide a reliable indication of leakage. The 1-gpm limit is well below the capacity of one coolant charging pump (60 gpm).

When the source of leakage has been identified, the situation can be evaluated to determine if operation can safely continue. Under these conditions, an allowable leakage rate of 10 gpm

has been established which is also well within the capacity of one charging pump and makeup would be available even under the loss of offsite power condition.

5.2.5.3 Locating Leaks

Methods of leak location that can be used during plant shutdown include visual observation for escaping steam or water or for the presence of boric acid crystals near the leak. The boric acid crystals are transported outside the reactor coolant system in the leaking fluid and then left behind by the evaporation process.

Periodic reactor coolant system leakage surveillance is conducted pursuant to plant procedures.

5.2.5.4 Leakage Detection System Descriptions

5.2.5.4.1 Containment Air Particulate and Radiogas Monitor

5.2.5.4.1.1 Air Particulate Monitor

The containment air particulate monitor (R-11) is the most sensitive instrument of those available for detection of reactor coolant leakage into the containment.

This instrument is capable of detecting particulate radioactivity in concentrations as low as 5 x 10^{-10} µCi/cm^3 of containment air.

5.2.5.4.1.2 Sensitivity Assumptions

The sensitivity of the air particulate monitor to primary system leakage is determined by making the following initial assumptions:

a. Containment volume - 970,000 ft^3 = 2.7 x 10^{10} cm^3.

b. Maximum air recirculation rate - 166,800 ft^3/min.

c. Average minimum noble gas (Xe-138/Kr-88) activity in reactor coolant system is 0.05 µCi/cm^3 in a 4:1 ratio of Xe-138/Kr-88.

d. Detector sensitivity threshold 5 x 10^{-10} µCi/cm^3 of sampled air, for average beta energies of Cs-138/Rb-88.

Using the mass balance equation

$A - Qc = V \times (dc/dt)$ (Equation 5.2-1)

where:	A =	Leak rate (µCi/min)
	Q =	Recirculation flow (cm^3/min)
	c =	Concentration in containment (µCi/cm^3)
	V =	Containment volume (cm^3)
	t =	Time (min)

Rearranging Equation 5.2-1 gives

$dc/(A-Qc) = 1/V\ dt$ (Equation 5.2-2)

Now, for a constant leak rate, i.e., A = constant,

$d(A-Qc) = -Qdc$

$dc = d(A-Qc)/-Q$ (Equation 5.2-3)

Rearranging Equation 5.2-2 gives

$d(A-Qc)/(A-Qc) = -(Q/V)dt$ (Equation 5.2-4)

Integrate Equation 5.2-4 gives

$A - Qc = K\ e^{-Qt/V}$ (Equation 5.2-5)

where $c = C_0$ and $K = A - QC_0$ at $t = O$

thus

$A - Qc = (A - Q\ C_0)\ e^{-Qt/V}$

or

$c = (A/Q) - [(A/Q) - C_0]\ e^{-Qt/V}$ (Equation 5.2-6)

Equation 5.2-6 is solved assuming various leak rates of reactor coolant with a noble gas activity of $0.05\mu Ci/cm^3$ which is the minimum average Kr-88/Xe-138 activity in the reactor coolant system. The results are plotted in Figure 5.2-3.

The sensitivity indicated by Figure 5.2-3 does not take into account the following advantages or disadvantages.

Advantages

i. The air particulate monitor filter paper can be fixed; the resulting sensitivity would afford earlier detection for a given leak rate.

ii. The air recirculation rate can be lower (here we have assumed the maximum), thus giving a more rapid increase in containment air activity.

iii. Other particulate activity released in an RCS leak (eg. NA-24, Co-58, Mo-99, C-11) would increase sensitivity to leak detection.

Disadvantages

i. The effect of partition factor in regions where leakage occurs.

ii. The absence of volatile radioactive particulate (absence of iodine isotope).

5.2.5.4.1.3 Leakage Detection Threshold

The sensitivity of the air particulate monitor is greatest when baseline leakage is low, as has been demonstrated by the experience of Indian Point Unit 1, Yankee Rowe, and Dresden Unit 1. Where containment air particulate activity is below the threshold of detection (5.0×10^{-10} $\mu Ci/cm^3$), the sensitivity of the monitor can be improved by fixing the filter paper in the monitor. In this case, there will be an accumulation of activity at the rate of flow of the sample. For example, if a sample flow rate of R_S cm^3/min is assumed, the accumulation of activity A_D at the detector will be governed by the following relationship:

$$A_D = R_S \int_0^t C(t)dt$$

(Equation 5.2-7)

where A_D is in µCi.

C(t) is given by a modified form of Equation 5.2-6 where V and Q are equal to the volume and recirculation terms applicable.

Hence,

$$A_D = R_S \left[K_1 t + K_2 \cdot e^{\frac{-Q}{Vt}} \right]_0^t$$

(Equation 5.2-8)

The evaluation of the above equation for a given leakage would depend upon the characteristics and response time for a given detector.

Assuming a low background of containment air particulate radioactivity, a reactor coolant noble gas with particle progeny activity of 0.5 $\mu Ci/cm^3$ (a value consistent with little or no fuel cladding leakage), and complete dispersion of the leaking radioactive gas into the containment air, the air particulate monitor is capable of detecting leaks as small as approximately 0.018 gpm (70 cm^3/min) within 20 min after they occur. If only 10% of the particulate activity is actually dispersed in the air, leakage rates of the order of 0.18 gpm (700 cm^3/min) are well within the detectable range.

For cases where baseline reactor coolant leakage falls within the detectable limits of the air particulate monitor, the instrument can be adjusted to alarm on leakage increases from two to five times the baseline value.

5.2.5.4.1.1 Radiogas Monitor

The containment radiogas monitor (R-12) is inherently less sensitive (threshold at 10^{-6} μCi/cm^3) than the containment air particulate monitor. With typical RCS activity, R-12 is able to identify a 7 gpm leak within 1 hour from the liquid space of the RCS. A leak from the gas space or during periods of high make-up or failed fuel will increase the sensitivity to 2-4gpm. Because of the lower sensitivity, this instrument is a useful back up to the particulate monitor.

5.2.5.4.2 Humidity Detector

The humidity detection instrumentation offers another means of detection of leakage into the containment. This instrumentation has not nearly the sensitivity of the air particulate monitor but has the advantage of being sensitive to vapor originating from all sources, including the reactor coolant and steam and feedwater systems. Plots of containment air dewpoint variations above a baseline maximum established by the cooling water temperature to the air coolers should be sensitive to incremental leakage equivalent to 2.0 to 10 gpm.

The sensitivity of this method depends on cooling water temperature, containment air temperature variation, and containment air recirculation rate. The containment humidity information is displayed on the plant computer.

5.2.5.4.3 Condensate Measuring System

The principle that the condensate collected by the cooling coils matches, under equilibrium conditions, the leakage of water and steam from systems within the containment applies because conditions within the containment promote complete evaporation of leaking water from hot systems. The air and internal structure temperatures are normally held at 125°F or less, the air is dry (i.e., not saturated with water vapor), and the cooling coils provide the only significant surfaces at or below the dewpoint temperature.

The containment cooling coils are designed to remove the sensible heat generated within the containment. The resulting large coil surface area means that the exit air from the coils has a dewpoint temperature which is very nearly equal to the cooling water temperature at the air exit.

Measurement of the condensate drained from the cooling coils is made to determine collection rate and thus leak rate. About one-half hour after the occurrence of a leak, the equilibrium condition is established in which the amount of the leakage change is matched by a change in the cooling coil condensation rate.

The condensate from each of the four containment cooling coils drains to a condensate collector (drain pan) that is equipped with a standpipe that is approximately 200 in. long. The condensate collector level instrumentation provides a signal proportional to the water level in the standpipe indicating that the collector is from 0 to 100% full with an uncertainty of less than +3%. Readouts of collector water level and a hi-hi level alarm are provided in the control room. The hi-hi level alarm is actuated when the standpipe is 80% +3% full for three of the collectors and 66% +3% full for the fourth collector at which point the collector is dumped to the containment sump.

Condensate flows from approximately 1 gpm to 30 gpm can be measured by the condensate collection system. Flows less than 1 gpm can be measured by periodic observation of the level changes in the condensate collection system.

5.2.5.4.4 Liquid Inventory in Process Systems and Containment Sumps

Leaks can also be detected by unscheduled increases in the amount of reactor coolant makeup water, which is required to maintain the normal level in the pressurizer. Based on the frequency of the inventory balance, and the volume control tank level instrumentation, it is estimated that the charging system inventory method of leak detection can detect a 0.25-gpm leak.

Gross leakage will cause a rise in the containment sumps water levels. Sump A water level rise will be alarmed in the control room upon auto-start of either of the sump A pumps. Water level in containment sump B is indicated in the control room by a series of five lights actuated by redundant signal contacts evenly spaced along the height of the sump.

5.2.5.5 Leakage Detection System Evaluation

Detection of leakage from the reactor coolant pressure boundary was reviewed as part of the NRC Systematic Evaluation Program (SEP Topic V-5). The results of the review are documented in *References 10* and *11*. The review was based on the requirements of 10 CFR 50, Appendix A, General Design Criterion 30, as implemented by Regulatory Guide 1.45 and SRP Section 5.2.5, which specify the types and sensitivity of the systems, as well as their seismic, indication, and testability criteria necessary to detect leakage of primary reactor coolant to the containment or to other interconnected systems.

The NRC concluded the following:

A. Ginna Station has all three systems required by Regulatory Guide 1.45. Two of the three systems meet the sensitivity requirements. The third system (sump A level monitoring) can measure approximately a 2-gpm leak in 1 hour. In addition to the three leakage detection systems, Ginna also incorporates six other diverse systems. Taking all these systems into consideration, a 1-gpm leak from the reactor coolant pressure boundary to the containment can be detected within 1 hr, as required by the Regulatory Guide.

B. Ginna has, as one of the diverse systems, the sump B level monitoring system, which is Seismic Category I and can measure a 10.5-gpm leak within 1 hour. Therefore, the plant adequately meets the leak detection needs following a seismic event, including the safe shut-down earthquake.

C. Provisions are made to monitor reactor coolant inleakage to interconnected systems (component cooling water (CCW) system and secondary system).

D. The Ginna Technical Specifications meet the intent of the Standard Technical Specifications concerning the operability of the leakage detection systems to monitor leakage to the primary containment. There is a difference in the number of required systems, which is not a significant safety factor because of the various diverse leakage detection systems available to the plant operators.

REFERENCES FOR SECTION 5.2

1. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Safety Topics III-6, Seismic Design Consideration and III-11, Component Integrity, dated January 29, 1982.

2. Letter from L. D. White, Jr., RG&E, to A. Schwencer, NRC, Subject: Overpressure Protection, dated February 24, 1977.

3. Letter from L. D. White, Jr., RG&E, to A. Schwencer, NRC, Subject: Overpressure Protection, dated March 31, 1977.

4. Letter from L. D. White, Jr., RG&E, to A. Schwencer, NRC, Subject: Overpressure Protection, dated July 29, 1977.

5. Westinghouse Electric Corporation, R. E. Ginna Low Temperature Overpressure Protection System (LTOP) Setpoint Phase II Evaluation Final Report, October 1990 (Proprietary) and February 1991 (Non-Proprietary) (Attachment C to letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Rochester Gas and Electric Corporation, R. E. Ginna Nuclear Power Plant, Docket 50-244, dated February 15, 1991).

6. Generic Letter 88-05, Boric Acid Corrosion of Carbon Steel Reactor Pressure Boundary Components in PWR Plants, dated March 17, 1988.

7. Letter from Allen Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Prevention of Boric Acid Corrosion at Ginna Nuclear Power Plant, dated August 10, 1990.

8. NRC Bulletin 88-09, Thimble Tube Thinning in Westinghouse Reactors, dated July 26, 1988.

9. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: NRC Bulletin 88-09, dated April 8, 1993.

10. Letter From D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic V-5, Reactor Coolant Pressure Boundary Leakage Detection, dated February 8, 1982.

11. U.S. Nuclear Regulatory Commission, Integrated Plant Safety Assessment, Systematic Evaluation Program, R. E. Ginna Nuclear Power Plant, NUREG 0821, December 1982.

12. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Generic Letter (GL) 95-03, Response to NRC Request for Additional Information, dated March 2, 1996.

13. Letter from R. C. Mecredy, RG&E, to G. Vissing, NRC, Subject: Response to Generic Letter 95-03, dated July 25, 1996.

14. Generic Letter 95-03, Circumferential Cracking of Steam Generator Tubes, dated April 28, 1995.

15. B&W Nuclear Technologies, LTOP Report for Ginna, 86-1234820-03, approved 9/19/97.

16. B&W Nuclear Technologies, RELAP5/MOD2-B&W - An Advanced Computer Program for Light Water Reactor LOCA and Non-LOCA Transient Analysis, Code Topical Report BAW-10164P, Revision 2, dated August 1992.

17. Deleted.

18. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to NRC Generic Letter 97-05, Steam Generator Tube Inspection Techniques, dated March 23, 1998.

19. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to Request for Additional Information Related to Generic Letter 97-05, dated September 16, 1998.

20. Generic Letter 97-05, Steam Generator Tube Inspection Techniques, dated December 17, 1997.

Table 5.2-1
REACTOR COOLANT SYSTEM CODE REQUIREMENTS

Component	**Codes**
Reactor Vessel	ASME III[a]Class A[b]
Rod drive mechanism housing	ASME III[a]Class A[b]
Replacement steam generators	
Tube side	ASME III[c]Class 1
Shell side[d]	ASME III[c]Class 2
Reactor coolant pump volute	ASME III[a]Class A
Pressurizer	ASME III[a]Class A
Pressurizer relief tank	ASME III[a]Class C
Pressurizer safety valves	ASME III[a]
Reactor coolant piping	USAS B31.1[e](1955)
Reactor coolant valves	ASA B16.5 (1961)
System valves, fittings, and piping	USAS B31.1[e](1955) ASA B16.5 (1961)

a. ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels (1965).

b. The replacement reactor vessel closure head and replacement equivalent control rod drive mechanism housings supplied by PCR 2001-0042, were supplied as ASME Section III Class 1 appurtenances to the 1995 edition with 1996 addenda of the ASME Boiler and Pressure Vessel Code in accordance with the Ginna Station Section XI Repair and Replacement Program.

c. ASME Boiler and Pressure Vessel Code, Section III, Division 1, 1986.

d. The shell side of the steam generator conforms to the requirements for Class 1 vessels and is so stamped as permitted under the rules of Section III.

e. USAS B31.1 Code for Pressure Piping.

Table 5.2-2
MATERIALS OF CONSTRUCTION OF THE REACTOR COOLANT SYSTEM COMPONENTS

Component	Section	Materials
Replacement steam generator	Pressure plate (manway covers)	SA-533 Tp B Cl 1
	Cladding, stainless weld	SFA 5.9 ER 308L/309L
	Cladding for tubesheets and seatbar	SFA 5.14 ER NiCr-3
	Nozzle dam retention rings	SB-166 N06690
	Tubes	SB-163 Alloy 690
	Channel head tubesheet and nozzles	SA-508 Cl 3
	Primary nozzle safe-ends	SA-336 316N/316LN
	Divider plate	SB-168 N06690
Pressurizer	Shell	SA-302, grade B
	Heads	SA-216 WCC
	External plate	SA-302, grade B
	Cladding, stainless	Type 304 equivalent
	Internal plate	SA-240 type 304
	Internal piping	SA-376 type 316
Piping	Pipes	A-376 type 316
	Fittings	A-351, CF8M
	Nozzles	A-182, F316
Pumps	Shaft	Type 304
	Impeller	A-251, CF8
	Casing	A-351, CF8M

Table 5.2-3
REACTOR COOLANT SYSTEM QUALITY ASSURANCE PROGRAM

Component	RT	UT	PT	MT	ET
Replacement steam generator					
Tubesheet					
Forging		X		X	
Cladding		X	X		
Channel head					
Forgings		X		X	
Cladding		X	X		
Secondary shell and head					
Plates and forgings		X		X	
Tubes		X			X
Nozzles (Forgings)		X		X (or PT)	
Weldments					
Secondary shell, longitudinal	X	X		X	
Secondary shell, circumferential	X	X		X	
Cladding		X	X		
Nozzle to shell	X	X		X	
Support brackets				X	
Tube-to-tube sheets			X		
Instrument connections (primary and secondary)			X		
Temporary attachments after removal				X (or PT)	
After hydrostatic test (all welds)				X	
Nozzle safe ends (if forgings)	X	X	X (or MT)		
Nozzle safe ends (if weld deposit)		X			
Pressurizer					
Heads					
Castings	X			X	
Cladding			X		
Shell					

Component	RT	UT	PT	MT	ET
Plates		X			
Cladding			X		
Heaters					
Tubings		X	X		
Centering of Element	X				
Nozzle		X	X		
Piping					
Fittings (castings)	X		X		
Fittings (forgings)		X	X		
Pipe		X	X		
Weldments					
Longitudinal	X		X		
Circumferential	X		X		
Nozzle to run pipe	X		X		
Instrument connections			X		
Pumps					
Castings	X		X		
Forgings		X	X		
Weldments					
Circumferential	X		X		
Instrument connections			X		
Reactor vessel					
Forgings					
Flanges		X		X	
Studs		X		X	
Head adapters	X		X		
Plates		X		X	
Weldments					
Main steam	X			X	
Control rod drive head adapter connection (W85)	X	X	X		X

Component	**RT**	**UT**	**PT**	**MT**	**ET**
Instrumentation tube			X		
Main nozzles	X			X	
Cladding			X		
Nozzle safe ends	X		X	X	

Notes:
RT Radiographic
UT Ultrasonic
PT Dye penetrant
MT Magnetic particle
ET Eddy current

Table 5.2-4

Table DELETED

Table DELETED

Table 5.2-5
REACTOR COOLANT PRESSURE BOUNDARY TO CONTAINMENT LEAKAGE DETECTION SYSTEMS

System	Leak Rate Sensitivity	Time Required to Achieve Sensitivity	Control Room Indication for Alarms and Indicators	Testable During Normal Operation (MODES1 and 2)
Sump A level (LT-2039 and LT-2044) monitoring (inventory)	2 gpm	1 hour	Yes	Yes
Sump A pump actuations monitoring (time meters)	2 gpm	1 hour	Yes	Yes
Airborne particulate radioactivity (R-11) monitoring	1 gpm[a]	NA	Yes	Yes
Airborne gaseous radioactivity (R-12) monitoring	7 gpm	1 hr	Yes	Yes
Condensate flow rate from air coolers	1-30 gpm	1 hr	Yes	Yes
Containment atmosphere pressure (PI-944) monitoring	NA	1 hr	Yes	Yes
Containment atmosphere humidity monitoring	2-10 gpm	NA	No	Yes
Containment atmosphere temperature (TE-6031, 6035, 6036, 6037, 6038, and 6045) monitoring	NA	NA	No	Yes
Chemical and volume control system	0.25 gpm	1 hr	Yes	Yes

NOTE:—NA = Not available

a. 0.018 gpm within 20 min assuming the presence of noble gas with particle progeny.

Table 5.2-6
REACTOR COOLANT PRESSURE BOUNDARY INTERSYSTEM LEAKAGE DETECTION SYSTEMS

Systems Which Interface With Reactor Coolant Pressure Boundary	Methods to Measure Reactor Coolant Pressure Boundary Inleakage	Leak Rate Sensitivity	Time Required to Achieve Sensitivity	Control Room Indication for Alarms and Indicators	Testable During Normal operation (MODES 1 and 2)
Secondary system	Condensate air ejector radiation monitor	0.02 gpm[a]	1 minute	Yes	Yes
Secondary system	Blowdown monitor	0.0025 gpm[b]	1 hour	Yes	Yes
Component cooling water (CCW) system	Surge tank level	NA	NA	Yes	Yes
Component cooling water (CCW) system	Radiation monitor	0.16 gpm	NA	Yes	Yes

a. Primary-to-secondary leakage of 1 gpd (leakage ≥ 0.0007 gpm) can be detected by R-47 with non-defected fuel.

b. Total leakage of 0.5 gal necessary for indication.

Table 5.2-7
SEQUENCE OF EVENTS - MASS ADDITION CASE

EVENT	TIME, SECONDS
Charging pumps started	0.0
Charging pumps reach full flow	1.0
Peak pressure of 587.4 psia reached in the bottom of the reactor vessel	7.45
Peak pressure of 525.5 psia reached at suction point for the residual heat removal system	7.45

Table 5.2-8
HEAT ADDITION AT 60°F - SEQUENCE OF EVENTS

EVENT	TIME, SECONDS
Reactor coolant pump started in loop that contains the pressurizer	0.0
Reactor coolant pump reaches full flow	17.4
Power operated relief valve opening signal for the first time	46.0
Peak pressure reached in the reactor vessel	46.0
Peak pressure reached in the residual heat removal system	46.0

Table 5.2-9
HEAT ADDITION AT 320°F - SEQUENCE OF EVENTS

EVENT	TIME, SECONDS
Reactor coolant pump started in loop that contains the pressurizer	0.0
Power operated relief valve opening signal for the first time	8.81
Peak pressure reached at the residual heat removal pump outlet	10.5
Reactor coolant pump reaches full flow	17.4
Peak pressure reached in the reactor vessel	21.3

5.3 REACTOR VESSEL

5.3.1 REACTOR VESSEL MATERIALS

5.3.1.1 Reactor Vessel Description

The Ginna reactor vessel was designed and fabricated by Babcock and Wilcox Company in accordance with Westinghouse specifications and the requirements of ASME Code, Section III, 1965 Edition. The governing specifications are listed in Table 5.3-1.

A replacement reactor vessel closure head was installed at Ginna Station during the Fall 2003 refueling outage. The replacement closure head was procured by PCR 2001-0042 through Babcock & Wilcox (B&W) of Canada, Ltd. in accordance with technical specification BWG-TS-2915 in accordance with ASME Section III 1995 Edition, with 1996 Addenda, Class 1 requirements.

The replacement closure head eliminated the use of the existing Alloy 600 control rod drive mechanism (CRDM) housings and weld material and replaced them with Alloy 690 TT (thermally treated) CRDM housings and Alloy 52 weld material.

The reactor vessel is cylindrical in shape with a hemispherical bottom and a flanged and gasketed removable upper head. Coolant enters the reactor vessel through two inlet nozzles in a plane just below the vessel flange and above the core. The coolant flows downward through the annular space between the vessel wall and the core barrel into a plenum at the bottom of the vessel where it reverses direction. Approximately 95% of the total coolant flow is effective for heat removal from the core. The remainder is considered as bypass flow as it is not fully effective for removing heat generated in the core. This bypass flow includes the flow through the rod cluster control guide thimbles, the flow between the core baffle and barrel, the leakage across the outlet nozzles, the flow deflected into the head of the vessel for cooling the upper flange, and the excess flow in the flow cells surrounding the rod cluster control guide thimbles. The bypass coolant and core coolant unite and mix in the upper plenum, and the mixed coolant stream then flows out of the vessel through two exit nozzles located on the same plane as the inlet nozzles. Figure 5.3-1, Sheets 1 and 2, is a schematic of the reactor vessel.

A one-piece thermal shield, concentric with the reactor core, is located between the core barrel and the reactor vessel. The shield, which is cooled by the coolant on its downward pass, protects the vessel by attenuating much of the gamma radiation and some of the fast neutrons which escape from the core. This shield minimizes thermal stresses in the vessel which result from heat generated by the absorption of gamma energy. The shield is further described in Section 3.9.5.1.1.

Thirty-six core instrumentation nozzles are located on the lower head.

The reactor closure head and the reactor vessel flange are joined by forty-eight 6-in. diameter studs. Two metallic O-rings seal the reactor vessel when the reactor closure head is bolted in place. A leakoff connection is provided between the two O-rings to monitor leakage across

the inner O-ring. In addition, a leakoff connection is also provided beyond the outer O-ring seal.

The vessel is insulated with metallic, reflective type insulation supported from the nozzles. Insulation panels are provided for the reactor closure head and are supported on the MODE 6 (Refueling) seal ledge and vent shroud support rings.

The reactor vessel internals are designed to direct the coolant flow, support the reactor core, and guide the control rods in the withdrawn position. The reactor vessel contains the core support assembly, upper plenum assembly, fuel assemblies, control rod cluster assemblies, surveillance specimens, and incore instrumentation.

The reactor internals are described in Sections 3.9.5 and 4.2.1 and the general arrangement of the reactor vessel and internals is shown in Figures 3.9-9 and 3.9-10.

Reactor vessel design data is listed in Table 5.3-2.

The reactor vessel is the only component of the reactor coolant system which is exposed to a significant level of neutron irradiation and it is therefore the only component which is subject to material radiation damage effects. The nil ductility transition temperature (NDTT) shift of the vessel material and welds, due to radiation damage effects during service, is monitored by a radiation damage surveillance program, as described in Section 5.3.3.

5.3.1.2 Material Specifications

The materials of construction of the reactor vessel are given in Table 5.3-3. A detailed listing of the reactor vessel core region forgings and welds is given in Table 5.3-4, along with the heat treatment history. The chemistry of all the materials is given in Table 5.3-5 and the mechanical properties are given in Table 5.3-6. The location of the reactor vessel beltline material is shown in Figure 5.3-2.

The cylindrical section of the reactor vessel is comprised of three cylindrical forgings (SA-508, class 2). The top and bottom dome sections are made from plate material (SA-533, grade A). The shell course, flanges, and nozzles are made from forgings (SA-508, class 2).

The forgings were processed by the mandrel forging technique. Prior to mandrel forging, the rough forging was upset and the center section removed. The forged section weld locations use the same inservice test techniques as those used for plate vessel welds.

The fracture toughness properties of forgings are comparable to plates in the unirradiated condition and the irradiated condition. Mechanical property tests for shell course forgings were taken at a one-fourth thickness location and at a minimum distance of one thickness from the quenched edge. Test locations complied with ASME Code, Section III, requirements.

The reactor vessel materials opposite the core were purchased to a specified Charpy V-notch impact energy of 30 ft-lb or greater at a corresponding NDTT of 40°F or less. The materials were subsequently tested (drop weight) to verify conformity to specified NDTT requirements. In addition, the plate sections were 100% volumetrically inspected by an ultrasonic test using both longitudinal and shear wave methods. The remaining material in the reactor vessel

meets the appropriate design code requirements and specific component functional requirements.

The reactor vessel material is heat-treated specifically to obtain good notch-ductility which ensures a low NDTT, and thereby gives assurance that the finished vessel can be initially hydrostatically tested and operated as near to room temperature as possible without restrictions. The stress limits established for the reactor vessel are dependent upon the temperature at which the stresses are applied. As a result of fast neutron irradiation in the region of the core, the material properties will change, including an increase in the NDTT.

There are two welds in the beltline region: the nozzle shell to intermediate shell (SA-1101) and the intermediate shell to lower shell (SA-847). Both are circumferential welds made by the submerged arc process. Based on radiation exposure and chemical composition, weld SA-847 is the limiting vessel material.

5.3.1.3 Testing and Surveillance

Westinghouse required, as part of its reactor vessel specification, that certain special tests which were not specified by the applicable codes be performed. These tests are listed below:

Ultrasonic Testing

Westinghouse required that a 100% volumetric ultrasonic test of reactor vessel plate for shear wave be performed. The 100% volumetric ultrasonic test is a severe requirement, but it ensures that plate used for the reactor vessel is of the highest quality.

Radiation Surveillance Program

In the surveillance program, the evaluation of the radiation damage is based on pre-irradiation and post-irradiation testing of Charpy V-notch, tensile and wedge opening loading test specimens. These programs are directed toward evaluation of the effect of radiation on the fracture toughness of reactor vessel steels based on the transition temperature approach and the fracture mechanics approach, and are in accordance with ASTM E185, Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors. The surveillance program for the RG&E reactor vessel is described in Section 5.3.3.

5.3.2 PRESSURE-TEMPERATURE LIMITS

5.3.2.1 Thermal and Pressure Loadings

Reactor vessel design is based on the transition temperature method of evaluating the possibility of brittle fracture of the vessel material resulting from operations such as leak testing and plant heatup and cooldown. To establish the service life of the reactor coolant system components as required by the ASME Code Section III for Class A vessels, the unit operating conditions which involve the cyclic application of loads and thermal conditions have been established for the 40-year design life. The number of thermal and loading cycles used for design purposes are listed in Table 5.1-4.

The stress level of material in the reactor vessel, or in other reactor coolant system components, is a combination of stresses caused by internal pressures and by thermal

gradients. The latter are significant as they may result from a rate of change of reactor coolant temperature and change the location of the limiting stress between heatup and cooldown. During cooldown, the thermal stress varies from tensile at the inner wall to compressive at the outer wall. The internal pressure superimposes a tensile stress on this thermal stress pattern, increasing the stress at the inside wall and relieving the stress at the outside wall. Therefore, the location of the limiting stress is always at the inside wall surface; however, for heatup the thermal stress is reversed so the location of the limiting stress is a function of the heatup rate. Operating restrictions are imposed to limit the combined stresses to 20% of minimum yield stress when at the design transition temperature. The design transition temperature is defined as the initial NDTT plus the increase in NDTT due to irradiation experienced plus 60°F. This stress limit (20% of yield stress) is reduced linearly to a value of 10% of yield at a temperature of 200°F below design transition temperature. Curves which define the operating limits are incorporated in the Pressure and Temperature Limits Report (PTLR).

5.3.2.2 Pressure-Temperature Limits

Pressure-temperature limits for reactor vessel operation provide a means of ensuring vessel integrity throughout its operating life. Operation in accordance with the curves ensures that, in the normal operating range, the vessel will operate in the upper-shelf region of its material toughness. This also provides assurance that the fracture toughness of vessel materials during heatup and cooldown transients will be adequate to prevent rapid crack propagation (brittle fracture).

Pressure-temperature limits for inservice testing, heatup and cooldown, and core operation are required to be in compliance with the rules of Appendix G to 10 CFR Part 50, Fracture Toughness Requirements. When first published in 1971, Appendix G used a transition temperature approach to establish safe operating limits. Appendix G was revised in 1973 to require a fracture mechanics approach, which usually gives more conservative operating limits.

The fracture mechanics approach relies on a fracture mechanics characterization of the material and its stress environment. Using this characterization, the stress in any portion of the vessel, in conjunction with any assumed flaw, can be compared with the stressed-flaw tolerance of the material, a material parameter such as K_{IC} (the plane strain fracture toughness of a material). Using this parameter, the stress in the vessel can be limited such that, in the presence of an assumed flaw size so large as to ensure detection, no rapid crack propagation can occur. Above NDTT, the fracture toughness of the materials used in the nuclear reactor vessels increases greatly. Thus, the crack tolerance of the material at the normal operating temperatures is high. Under this system of fracture control, prevention of rapid fracture is ensured by the control of stresses and flaw sizes. For nuclear vessel materials of normal shelf fracture toughness (according to Appendix G, 10 CFR 50, a Charpy upper-shelf energy of 50 ft-lb is required), very large cracks would be required to cause the onset of rapid crack propagation at operating temperature and pressure. In regions of high local stresses, such as nozzle corners, ductile tearing could commence at smaller cracks or lower pressure but, as the tear extended into a region of lower nominal stress such as the vessel wall, rapid fracture would again require very large cracks.

5.3.2.3 Pressure-Temperature Limit Calculation

The specific methods to calculate the pressure-temperature operating limits are contained in Appendix G to ASME Code, Section III. For regions remote from discontinuities (the beltline region), the stress intensity factors calculated in the development of these operating limits are based on a postulated sharp surface flaw penetrating to a depth of one-fourth of the vessel wall thickness and having a length one-and-one-half times the section thickness. Since the maximum size flaw that might escape detection in a preservice or inservice inspection is much smaller than this assumed flaw size, the combination of inspections and conservative pressure-temperature limits provides a high degree of assurance for vessel integrity throughout service life. For nozzles, flanges, and shell regions near discontinuities, a smaller defect size may be used. The smaller defect size must be justified and non-destructive examination methods must be sufficiently reliable and sensitive to detect these smaller defects. The procedures to calculate the stress intensity factors for these regions provide margins of safety comparable to those required for the beltline region. Appendix G provides methods to calculate stress intensities for membrane tension stress, bending stress, and stresses resulting from thermal gradients, and lists the safety factors to be applied to these stress intensities.

As a result of the extended power uprate to 1775 MWt, the impact of increased neutron fluence on the existing Ginna P/T limits curves was evaluated *(Reference 26)*. The evaluation determined that for the existing Ginna methodology for determining P/T limits, the integrated neutron fluence after uprate did not exceed the fluence projections used to develop the pre-uprate P-T limit curves for both 28 and 32 EFPY *(Reference 27)*. Consequently, the pre-uprate P-T limits used in the Ginna PTLR remained valid for up to 32 EFPY of reactor operation. For plant operation up to 53 EFPY new P-T limit curves for the Ginna PTLR were developed consistent with the methodology specified in Ginna Technical Specifications *(Reference 29)*.

The impact of the increased end of life neutron fluence due to uprate on the Upper Shelf Energy (USE) for reactor vessel pressure boundary materials was also evaluated *(Reference 26)*. The USE evaluation was re-performed following material analysis of Capsule N *(Reference 30)*. All beltline materials are expected to have a USE greater than 50 ft-lb through the end of plant life in 2029 except for the intermediate-to-lower shell girth weld and the intermediate-to-nozzle shell girth weld. As required by 10CFR50 Appendix G, an equivalent margins analysis (EMA) reflecting uprated conditions for these two weld locations was performed which demonstrated that sufficient USE margin existed at the uprated power level.

5.3.2.4 Irradiation Effect on Pressure-Temperature Limit

Irradiation degrades material toughness causing RT_{NDT} to increase. Since the pressure-temperature limits are based on a temperature above RT_{NDT}, these limits must be revised periodically to reflect the changes in toughness. Since the postulated flaw penetrates to one-fourth the wall thickness, the increase in RT_{NDT} is based on the fluence at the one-fourth thickness location. Increases in RT_{NDT} are usually obtained from the results of the vessel material surveillance program. If these results are for some reason not considered applicable or valid, the staff uses Regulatory Guide 1.99, Revision 2, to obtain conservative radiation damage values.

5.3.2.5 Heatup and Cooldown Rates

The reactor coolant system temperature and pressure and heatup and cooldown rates (with the exception of the pressurizer) are limited in accordance with the Pressure and Temperature Limits Report (PTLR). The actions to follow if the limits are exceeded are included in the Technical Specifications. The heatup and cooldown rates shall not exceed 60°F per hour and 100°F per hour, respectively. For the pressurizer, the heatup and cooldown rates do not exceed 100°F per hour and 200°F per hour, respectively. The pressurizer spray is not to be used if the temperature difference between the pressurizer and the spray fluid is greater than 320°F.

The normal system heating rate is 50°F per hour. Sufficient electrical heaters are installed in the pressurizer to permit a heatup rate of 55°F/hr, starting with a minimum water level.

The administrative limit for plant cooldown is 90°F/hr. The fastest cooldown rates which result from the hypothetical case of a main steam line break are discussed in Section 15.1.5.

A maximum temperature difference of 200°F between the pressurizer and reactor coolant system is specified to maintain thermal stresses within the surge line below design limits.

Temperature requirements for pressurization of the pressurizer and steam generators correspond with the design transition temperature measured for the material of each component.

The rates of temperature change are applied as total change in temperature in any 1-hour period.

5.3.3 REACTOR VESSEL INTEGRITY

5.3.3.1 Safety Factors

The reactor vessel has a 132-in. I.D., which is within standard size limits for which there is a good deal of operating experience. A stress evaluation of the reactor vessel was carried out in accordance with the rules of Section III of the ASME Code. The evaluation demonstrated that stress levels were within the stress limits of the code. Table 5.3-7 presents a summary of the results of the stress evaluation.

A summary of fatigue usage factors for components of the reactor vessel is given in Table 5.3-8.

The cycles specified for the fatigue analysis are the results of an evaluation of the expected plant operation coupled with experience from operational nuclear power plants. These cycles include five heatup and cooldown cycles per year, a conservative selection since the vessel would not complete more than one cycle per year during MODES 1 and 2.

The vessel design pressure is 2485 psig, while the normal operating pressure is 2235 psig. The resulting operating membrane stress is therefore amply below the code-allowable membrane stress to account for operating pressure transients.

The vessel closure contains forty-eight 6-in. studs. The stud material is ASTM A-540 and code case 1335-2 which has a minimum yield strength of 104,400 psi at design temperature.

The membrane stress in the studs when they are at the steady-state operational condition is approximately 37,500 psi. This means that 18 of the 48 studs have the capability of withstanding the hydrostatic end load on vessel head without the membrane stress exceeding yield strength of the stud material at design temperature.

The method to perform analyses to guard against fast fracture in the reactor vessel is included in Appendix G to Section III of the ASME Code. The method utilizes fracture mechanics concepts and is based on the reference nil ductility temperature, RT_{NDT}.

RT_{NDT} is defined as the greater of the drop weight NDTT (per ASTM E-208) or the temperature 60°F less than the 50 ft-lb temperature (or 35-mil lateral expansion temperature if this is greater), as determined from Charpy specimens oriented normal to the working direction of the material. The RT_{NDT} of a given material is used to index that material to a reference stress intensity factor curve, K_{IR} curve, which appears in Appendix G of the ASME Code.
The K_{IR} curve is a lower bound of dynamic, crack arrest, and static fracture toughness results obtained from several heats of pressure vessel steel. When a given material is indexed to the K_{IR} curve, allowable stress intensity factors can be obtained for this material as a function of temperature. Allowable operating limits can then be determined utilizing these allowable stress intensity factors.

The RT_{NDT} and, in turn, the operating limits of the reactor are adjusted to account for the effects of radiation on the reactor vessel material properties. The radiation embrittlement or changes in mechanical properties of the pressure vessel steel are monitored by the material surveillance program as described in Section 5.3.3.2. The increase in the Charpy V-notch 50 ft-lb temperature (delta RT_{NDT}) due to irradiation is added to the original RT_{NDT} to adjust the RT_{NDT} for radiation embrittlement. This adjusted RT_{NDT} (RT_{NDT} initial + delta RT_{NDT}) is used to index the material to the K_{IR} curve and, in turn, to set operating limits for the plant which take into account the effects of irradiation on the reactor vessel materials.

As part of the plant operator training program, supervisory and operating personnel are instructed in reactor vessel design, fabrication, and testing, as well as precautions necessary for pressure testing and MODES 1 and 2. The need for recordkeeping is stressed, such records being helpful for future summation of time at power level and temperature which tends to influence the irradiated properties of the material in the core region. These instructions are incorporated into the operating manuals.

5.3.3.2 Material Surveillance Program

The material surveillance program for Ginna was previously described in WCAP 7254 (*Reference 1*). The program was designed to meet the requirements of 10 CFR 50, Appendix H, and ASTM E-185-73. Capsules withdrawn after July 26, 1983, will be tested and the results reported in accordance with the 1982 revision of ASTM E-185 as required by 10 CFR 50, Appendix H. It consists of six surveillance capsules (V, R, T, P, S, and N) positioned in the reactor vessel between the thermal shield and the reactor vessel wall as shown in Figure 5.3-3. The vertical center of each capsule is opposite the vertical center of the core. Each capsule contains tensile, Charpy V-notch, and wedge opening loading specimens from the

forgings (heats 125P666 and 125S255) and weld metal, and Charpy V-notch specimens from heat-affected zone material and from an A-302, Grade B correlation material furnished by U.S. Steel Corporation. Data on the correlation material gives an indication of radiation damage in a commercial reactor vessel compared to a test reactor vessel, and also gives an indication of the accuracy of the neutron fluence calculations.

The material surveillance program for Ginna is described in BAW 1543 (*Reference 2*). BAW 1543 describes the Master Integrated Reactor Vessel Material Surveillance Program for Babcock & Wilcox-fabricated PWR reactor vessels containing seam welds fabricated by the automatic submerged arc process using copper-plated magnesium-molybdenum-nickel steel filler metal and Linde 80 flux. BAW 1543 describes the approach that the Babcock & Wilcox vessel owners will use in addressing the "Linde 80" welds. In addition to the six supplementary capsules that were previously added to the program, eight irradiation capsules are included, which further expand the fracture toughness data base for this class of materials and include life extension and annealing considerations. The Master Integrated Reactor Vessel Material Surveillance Program, therefore, includes a total of 17 plant specific reactor vessel surveillance programs and 14 supplementary material irradiation capsules. These reactor vessels include eight Babcock & Wilcox-designed 177 fuel assembly plants and nine Westinghouse designed plants with Babcock & Wilcox-fabricated reactor vessels. The information obtained from all of these sources is coordinated and shared to maximize the usefulness of the data.

All surveillance specimens were machined from the one-fourth thickness location of the forgings. The specimens represent material that was taken at least one forging thickness away from the quenched end of the forging. All Charpy V-notch and tensile specimens were oriented with the longitudinal axis of each specimen parallel to the hoop direction (strong direction) of the forgings. The wedge opening loading specimens were machined with the simulated crack of each specimen perpendicular to the surfaces and the hoop direction of the forgings.

The surveillance capsules contain dosimeter wires of copper, nickel, and aluminum-cobalt. They also contain cadmium-shielded dosimeters of Neptunium-237 and Uranium-238. The dosimeters permit evaluation of the neutron flux seen by the various specimens. Surveillance capsules V, R, T, S, and N have been removed and tested in accordance with Technical Specifications and test results documented in *References 3 through 6* and *28* respectively. Test results are analyzed, the shift in transition temperature is compared to the predicted shift, and pressure-temperature limit curves (Section 5.3.2) are revised accordingly. Surveillance capsule P will be removed in the future. See Section 5.3.3.3.

5.3.3.3 Surveillance Program Analysis

Capsule V was removed and tested in 1971, capsule R in 1974, capsule T in 1980, and capsule S in 1993. The insertion and withdrawal schedules for capsules P and N have been prepared in accordance with ASTM E-185-82 and the criteria for integrated surveillance programs of 10 CFR 50, Appendix H, paragraph II.C, and reside in the Master Integrated Reactor Vessel Material Surveillance Program (*Reference 2*). The NRC staff has determined that the material surveillance program at Ginna satisfies Appendix H to 10 CFR 50 (*Reference 8*). *Reference 9* documented acceptability of BAW 1543. All capsules in the

Ginna reactor vessel surveillance program contain SA-1036 weld material, which is a surrogate for SA-847, a beltline material in Ginna and Point Beach Unit 1. SA-1135 weld material is also a surrogate for SA-847.

As a result of the extended power uprate to 1775 MWt the integrated neutron fluence on the vessel at the end of the 60 year plant life increased. The effect of the increased integrated neutron fluence on the Ginna reactor vessel surveillance withdrawal schedule was evaluated as part of the extended power uprate *(Reference 26)*. The only changes to the capsule withdrawal schedule due to uprate are related to i) the capsule fluence values, ii) lead factors and
iii) timing of future capsule withdrawals. The total number of capsules that are required to be removed over the life of the plant was unaffected by implementation of the extended power uprate.

Surveillance capsule N was withdrawn at the refueling outage following achievement of a neutron fluence shortly after the equivalent of 60 calendar years of operation. Capsule N was removed during the spring 2008 refueling outage and the test report submitted to the NRC by *Reference 28*.

Capsule P, the last surveillance capsule, is to be removed shortly after it accumulates a fluence equivalent to 80 years of operation, however, it is planned that dosimetry monitors will be reinstalled, such that the neutron flux could continue to be monitored throughout the period of extended operation (until year 2029).

5.3.3.3.1 Results Summary

The analysis of the reactor vessel materials contained in surveillance capsule N, the fifth capsule to be removed from the reactor vessel, was reported to the NRC by *Reference 28*. The analysis led to the following conclusions:

A. The capsule received an average fast neutron fluence (E > 1.0 MeV) of 5.80 x 10^{19} n/cm^2 after 30.5 effective full power years of plant operation.

B. Irradiation of the reactor vessel lower forging 125P666 Charpy specimens, oriented with the longitudinal axis of the specimen parallel to the major rolling direction (longitudinal direction), to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) resulted in a 30 ft-lb transition temperature increase of 91.1°F and a 50 ft-lb transition temperature increase of 93.3°F. This results in an irradiated 30 ft-lb transition temperature of 44.9°F and an irradiated 50 ft-lb transition temperature of 78.4°F for the longitudinally oriented specimens.

C. Irradiation of the reactor vessel intermediate shell forging 125S255 Charpy specimens, oriented with the longitudinal axis of the specimen parallel to the major rolling direction (longitudinal orientation), to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) resulted in a 30 ft-lb transition temperature increase of 76.4°F and a 50 ft-lb transition temperature increase of 100.0°F. This results in an irradiated 30 ft-lb transition temperature of 47.5°F and an irradiated 50 ft-lb transition temperature of 102.8°F for the longitudinally oriented specimens.

D. Irradiation of the weld metal Charpy specimens to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) resulted in a 30 ft-lb transition temperature increase of 216.9°F and a 50 ft-lb transition

temperature increase of 261.0°F. This results in an irradiated 30 ft-lb transition temperature of 182.2°F and an irradiated 50 ft-lb transition temperature of 276.0°F.

E. Irradiation of the weld heat affected zone metal Charpy specimens to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) resulted in a 30 ft-lb transition temperature increase of 107.7°F and a 50 ft-lb transition temperature increase of 74.5°F. This results in an irradiated 30 ft-lb transition temperature of 43.0°F and an irradiated 50 ft-lb transition temperature of 58.4°F.

F. The average upper shelf energy of intermediate shell forging 125P666 (longitudinally orientation) resulted in an energy decrease of 32.3 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV). This results in an irradiated average upper shelf energy of 142.3 ft-lb for longitudinally oriented specimens.

G. The average upper shelf energy of intermediate shell forging 125S255 (longitudinally orientation) resulted in an energy decrease of 5.7 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV). This results in an irradiated average upper shelf energy of 134.3 ft-lb for longitudinally oriented specimens.

H. The average upper shelf energy of the weld metal Charpy specimens resulted in an energy decrease of 27.1 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV). This results in an irradiated average upper shelf energy of 51.9 ft-lb for the weld metal specimens.

I. The calculated end-of-life (53 effective full power years) maximum neutron fluence (E > 1.0 MeV) for the reactor vessel is as follows:

Vessel inner radius [a] = 5.56 x 10^{19} n/cm^2

Vessel 1/4 thickness = 3.76 x 10^{19} n/cm^2

Vessel 3/4 thickness = 1.73 x 10^{18} n/cm^2

J. The average upper shelf energy of the weld heat affected zone metal decrease of 1.7 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV). This results in an irradiated upper shelf energy of 88.3 ft-lb for the weld heat affected zone metal.

K. A comparison of the surveillance Capsule N test results with the Regulatory Guide 1.99, Revision 2, predictions led to the following conclusions:

- The measured 30 ft-lb shift in transition temperature values of the Intermediate Shell Forging 125S255 and Lower Shell Forging 125P666 specimens contained in capsule N are greater than the Regulatory Guide 1.99, Revision 2 predictions.
- The measured 30 ft-lb shift in transition temperature value of the Surveillance Weld Heat # 61782 specimens contained in capsule N is less than the Regulatory Guide 1.99, Revision 2 prediction.
- The measured percent decrease in upper shelf energy for all forging and weld surveillance materials in Capsule N are less than the Regulatory Guide 1.99, Revision 2 predictions.

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a. Clad / base metal interface.

The summary of all five surveillance capsule results appears in Table 5.3-9.

Generic Letter 88-11, "NRC Position on Radiation Embrittlement of Reactor Vessel Materials and Its Impact on Plant Operations," required each licensee to reevaluate the effect of neutron radiation on reactor vessel material using the methods described in Regulatory Guide 1.99, Revision 2. This reevaluation was performed by Westinghouse and is documented in *Reference 10*. Based on the Westinghouse reevaluation, the heatup and cooldown limit curves in effect at the time were considered to be appropriate for use up to 21 effective full power years of operation. The most recent curves are in the Pressure and Temperature Limits Report (PTLR).

5.3.3.3.2 Charpy V-Notch Impact Test Results

Irradiation of the reactor vessel lower shell forging 125P666 Charpy specimens oriented with the longitudinal axis of the specimen parallel to the major rolling direction of the forging (longitudinal orientation) to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F resulted in a 30 ft-lb transition temperature increase of 91.1°F and a 50 ft-lb transition temperature increase of 93.3°F. This results in an irradiated 30 ft-lb transition temperature of 44.9°F and an irradiated 50 ft-lb transition temperature of 78.4°F (longitudinal orientation).

The average upper shelf energy of the lower shell forging 125P666 Charpy specimens (longitudinal orientation) resulted in an energy decrease of 32.3 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F. This results in an irradiated average upper shelf energy of 142.3 ft-lb.

Irradiation of the reactor vessel intermediate shell forging 125S255 Charpy specimens oriented with the longitudinal axis of the specimen parallel to the major rolling direction of the forging (longitudinal orientation) to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F resulted in a 30 ft-lb transition temperature increase of 76.4°F and a 50 ft-lb transition temperature increase of 100.0°F. This results in an irradiated 30 ft-lb transition temperature of 47.5°F and an irradiated 50 ft-lb transition temperature of 102.8°F (longitudinal direction).

The average upper shelf energy of the intermediate shell forging 125S255 Charpy specimens (longitudinal direction) resulted in an energy decrease of 5.7 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F. This results in an irradiated average upper shelf energy of 134.3 ft-lb.

Irradiation of the surveillance weld metal Charpy specimens to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F resulted in a 30 ft-lb transition temperature shift of 216.9°F and a 50 ft-lb transition temperature increase of 261.0°F. This results in an irradiated 30 ft-lb transition temperature of 182.2°F and an irradiated 50 ft-lb transition temperature of 276.0°F.

The average upper shelf energy of the surveillance weld metal resulted in an energy decrease of 27.1 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F. This results in an irradiated average upper shelf energy of 51.9 ft-lb.

Irradiation of the reactor vessel weld heat affected zone metal Charpy specimens to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F resulted in a 30 ft-lb transition temperature increase of 107.7°F and a 50 ft-lb transition temperature increase of 74.5°F. This results in an irradiated 30 ft-lb transition temperature of 43.0°F and an irradiated 50 ft-lb transition temperature of 58.4°F.

The average upper shelf energy of the weld heat affected zone metal resulted in an energy decrease of 1.7 ft-lb after irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F. This results in an irradiated average upper shelf energy of 88.3 ft-lb.

A comparison of the 30 ft-lb transition temperature increases and upper shelf energy decreases for the various R. E. Ginna surveillance materials with predicted values using the methods of Regulatory Guide 1.99, Revision 2, is presented in Table 5.3-10. This comparison indicates that the capsule N surveillance materials are in good agreement with the Regulatory Guide 1.99, Revision 2, predictions.

5.3.3.3.3 Tension Test Results

The results of the tension tests performed on the lower shell forging 125P666 (longitudinal orientation) indicated that irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F caused a 13.1 ksi increase in the 0.2% offset yield strength and a 8.3 ksi increase in the ultimate tensile strength when compared to unirradiated data.

The results of the tension tests performed on the intermediate shell forging 125S255 (longitudinal orientation) indicated that irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F caused a 17.7 ksi increase in the 0.2% offset yield strength and a 11.3 ksi increase in the ultimate tensile strength when compared to unirradiated data.

The results of the tension tests performed on the surveillance weld metal indicated that irradiation to 5.80 x 10^{19} n/cm^2 (E > 1.0 MeV) at 550°F to 574°F caused a 24.5 ksi increase in the ultimate tensile strength when compared to unirradiated data.

5.3.3.3.4 Radiation Analysis and Neutron Dosimetry

The radiation analysis and neutron dosimetry methods employed in the Surveillance Program Analysis are described in detail in *Reference 30*.

5.3.3.4 Analysis of Effects of Loss of Coolant and Safety Injection on the Reactor Vessel

The analysis of the effects of injecting safety injection water into the reactor coolant system following a postulated loss-of-coolant accident was performed by Westinghouse for the initial licensing with the following results.

5.3.3.4.1 Reactor Vessel

For the reactor vessel, three modes of failure were considered, including the ductile mode, brittle mode, and fatigue mode.

A. Ductile Mode. The failure criterion used for this evaluation was that there shall be no gross yielding across the vessel wall using the material yield stress specified in Section III of the ASME Boiler and Pressure Vessel Code. The combined pressure and thermal stresses during injection through the vessel thickness as a function of time were calculated and compared to the material yield stress at the times during the safety injection transient.

   The results of the analyses showed that local yielding may occur in approximately the inner 12% of the base metal and in the cladding.

B. Brittle Mode. The possibility of a brittle fracture of the irradiated core region was considered from both a transition temperature approach and a fracture mechanics approach. The failure criterion used for the transition temperature evaluation was that a local flaw cannot propagate beyond any given point where the applied stress would remain below the critical propagation stress at the applicable temperature at that point.

   The results of the transition temperature analysis showed that the stress-temperature condition in the outer 65% of the base metal wall thickness remains in the crack arrest region at all times during the safety injection transient. Therefore, if a defect were present in the most detrimental location and orientation (i.e., a crack on the inside surface and circumferentially directed), it could not propagate any further than approximately 35% of the wall thickness, even considering the worst case assumptions used in the analysis.

   The results of the fracture mechanics analysis, considering the effects of water temperature, heat transfer coefficients, and fracture toughness of the material as a function of time, temperature, and irradiation were considered. Both a local crack effect and a continuous crack effect were considered with the latter requiring the use of a rigorous finite element axisymmetric code.

C. Fatigue Mode. The failure criterion used for the failure analysis was the one presented in Section III of the ASME Boiler and Pressure Vessel Code. In this method the piece was assumed to fail once the combined usage factor at the most critical location for all transients applied to the vessel exceeds the code allowable usage factor of one.

   The results of the analysis showed that the combined usage factor never exceeded 0.2, even after assuming that the safety injection transient occurred at the end of plant life.

In order to promote a fatigue failure during the safety injection transient at the end of plant life, it has been estimated that a wall temperature of approximately 1100°F is needed at the most critical area of the vessel (instrumentation tube welds in the bottom head).

The design basis of the safety injection system ensures that the maximum cladding temperature does not exceed the Zircaloy-4 or ZIRLO™ melt temperature. This is achieved by prompt recovery of the core through flooding, with the passive accumulator and the injection systems. Under these conditions, a vessel temperature of 1100°F is not considered a credible possibility and the evaluation of the vessel under such elevated temperatures is for a hypothetical case.

For the ductile failure mode, such hypothetical rise in the wall temperature would increase the depth of local yielding in the vessel wall.

The results of these analyses show that the integrity of the reactor vessel is never violated.

5.3.3.4.2 Safety Injection Nozzles

The safety injection nozzles have been designed to withstand ten postulated safety injection transients without failure. This design and associated analytical evaluation was made in accordance with the requirements of Section III of the ASME Boiler and Pressure Vessel Code.

The maximum calculated pressure plus thermal stress in the safety injection nozzle during the safety injection transient was calculated to be approximately 55,400 psi. This value compares favorably with the code allowable stress of 80,000 psi.

These ten safety injection transients were considered along with all the other design transients for the vessel in the fatigue analysis of the nozzles. This analysis showed the usage factor for the safety injection nozzles was 0.219 which is well below the code allowable value of 1.0.

The safety injection nozzles are not in the highly irradiated region of the vessel and thus they are considered ductile during the safety injection transient.

5.3.3.4.3 Fuel Assembly Grid Springs

The effect of the safety injection water on the fuel assembly grid springs was evaluated. Due to the fact that the springs have a large surface area to volume ratio, being in the form of thin strips, and are expected to follow the coolant temperature transient with very little lag, no thermal shock is expected and the core cooling is not compromised.

5.3.3.4.4 Core Barrel and Thermal Shield

Evaluations of the core barrel and thermal shield have also shown that core cooling is not jeopardized under the postulated accident conditions.

5.3.3.4.5 Subsequent Analyses of Reactor Vessel

Subsequent analyses on the reactor vessel integrity were submitted to the NRC by *Reference 11* (WCAP 10019). This report, submitted in response to NUREG 0737, Item II.K.2.13, provides classical mechanics analyses of design-basis accidents, which demonstrate that there are no immediate reactor vessel integrity concerns. The basis for the thermal stress and fracture analyses of *Reference 11* was also used in the evaluation of the reactor vessel integrity, performed after the Ginna steam generator tube rupture incident (*Reference 12*).

In 1992, the NRC issued Revision 1 of Generic Letter 92-01 (*Reference 13*) to obtain information needed to assess compliance with requirements and commitments regarding reactor vessel integrity in view of certain concerns raised in the NRC staff's review of reactor vessel integrity for the Yankee Rowe Nuclear Power Station. In 1995, the NRC issued Revision 1, Supplement 1 of Generic Letter 92-01 (*Reference 15*). RG&E's responses to Revision 1 and Revision 1, Supplement 1 of Generic Letter 92-01 are contained in *References 14*, *16*, and *17*. The NRC in *Reference 18* stated that since RG&E had provided the requested information and indicated that previous submittals remained valid, the NRC considers the reactor pressure vessel integrity data for Ginna to be complete and closed out Generic Letter

92-01, Revision 1, Supplement 1. Since the closure, additional reactor vessel integrity correspondence (*References 19 through 21*) has been sent to or from the NRC.

5.3.3.5 Pressurized Thermal Shock

The issue of pressurized thermal shock arises because in pressurized water reactors transients and accidents can occur that result in severe overcooling of the reactor vessel, concurrent with or followed by repressurization. The issue is a concern after the reactor vessel has lost its toughness properties and is embrittled by neutron irradiation. The rate of decrease of the fracture resistance of the reactor vessel material is dependent on the metallurgical composition of the vessel walls and welds.

In accordance with the requirements of 10 CFR 50.61, Fracture Toughness Requirements for Protection Against Pressurized Thermal Shock Events, Ginna Station submitted projected RT_{PTS} values for the reactor vessel beltline materials from the present to the expiration date of the operating license to the NRC (*Reference 22*). The projected values were below the screening criteria for the expiration date and beyond 32 effective full power years. The NRC by *Reference 23*, which included the safety evaluation report, reported that Ginna Station met the requirements of the pressurized thermal shock rule (10 CFR 50.61).

On March 22, 1996, the NRC issued a Safety Evaluation Report (*Reference 24*), that concluded that the Ginna reactor vessel is projected to be below the PTS screening criteria at the expiration of its license. The Safety Evaluation Report contained the following conclusions:

1. The RG&E method for determining the credibility of the Ginna surveillance data did not conform to the guidance in Regulatory Guide 1.99, Rev. 2. However, the NRC evaluation of the data indicates that the Ginna surveillance data complies with the credibility criteria of Regulatory Guide 1.99, Rev. 2.
2. Since the Ginna surveillance data complies with the credibility criteria of Regulatory Guide 1.99, Rev. 2, the surveillance data should be used to determine the chemistry factor for the limiting Ginna vessel weld.
3. RG&E's and the NRC's calculated values of RT_{PTS} at the expiration of the Ginna license are within 3°F (265°F and 268°F) and are well below the 300°F screening criterion specified in 10 CFR 50.61 for circumferential welds. Since this conclusion is dependent upon the available chemistry data and surveillance data, it is subject to change when new data becomes available.

As a result of the extended power uprate to 1775 MWt, the impact of increased neutron fluence on the pre-uprate Pressurized Thermal Shock (PTS) analyses was evaluated *(Reference 26)*. PTS was re-evaluated following the pulling of Capsule N for extended plant operation out to 53 EFPY *(Reference 29)*. The evaluation assessed the impact of increased fluence on all of the reactor vessel beltline materials using the rules from 10CFR50.61. The limiting materials for PTS is the intermediate-to-lower shell girth weld. For this material the increased fluence from the uprate caused the end of plant life reference temperature to increase slightly from 270.6°F to 273°F, and increased slightly following Capsule N analysis to 275°F out to 53 EFPY, which is below the 300°F allowable temperature for

circumferential weld materials. Consequently, at uprated plant conditions, the reactor vessel beltline materials continue to comply with the PTS screening criteria requirements of 10CFR50.61.

REFERENCES FOR SECTION 5.3

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2. Babcock & Wilcox, Master Integrated Reactor Vessel Material Surveillance Program, BAW 1543, Revision 4, February 1993.

3. Westinghouse Electric Corporation, Analysis of Capsule V from the Rochester Gas and Electric, R. E. Ginna Unit No. 1 Reactor Vessel Radiation Surveillance Program, FP-RA-1, April 1973. (Submitted by RG&E to the AEC by letter from G. E. Green to D. K. Skovholt, dated April 25, 1973.)

4. Westinghouse Electric Corporation, Analysis of Capsule R from the Rochester Gas and Electric, R. E. Ginna Unit No. 1 Reactor Vessel Radiation Surveillance Program, WCAP 8421, November 1974. (Submitted by RG&E to the NRC as an enclosure to Application for Amendment to the Operating License for the R. E. Ginna Nuclear Power Plant, dated March 6, 1975.)

5. Westinghouse Electric Corporation, Analysis of Capsule T from the Rochester Gas and Electric Corporation of R. E. Ginna Nuclear Plant Reactor Vessel Radiation Surveillance Program, WCAP 10086, April 1982. (Submitted by RG&E to the NRC as an enclosure to the Application for Amendment to the Operating License for the R. E. Ginna Nuclear Power Plant, dated December 8, 1982.)

6. Westinghouse Electric Corporation, Analysis of Capsule S from the Rochester Gas and Electric Corporation R. E. Ginna Reactor Vessel Radiation Surveillance Program, WCAP 13902, December 1993. (Submitted by RG&E to the NRC in a letter dated March 29, 1994.)

7. Letter from R. W. Kober, RG&E, to W. A. Paulson, NRC, Subject: Reactor Vessel Surveillance Capsule T, dated August 8, 1984.

8. Letter from A. R. Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Issuance of Amendment 44 to Facility Operating License (Section 3.1.2), dated August 8, 1991.

9. Letter from J. N. Hannon, NRC, to J. H. Taylor, B&W Owners Group, Subject: B&W Report BAW 1543, Rev. 3, Master Integrated Reactor Vessel Material Surveillance Program (TAC No. 75131), dated June 11, 1991.

10. Westinghouse Electric Corporation, Rochester Gas and Electric Reactor Vessel Life Attainment Plan, March 1990.

11. T. Mayer, Summary Report on Reactor Vessel Integrity for Westinghouse Operating Plants, WCAP 10019, December 1981.

12. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: Incident Evaluation Steam Generator Tube Rupture Incident, dated April 12, 1982.

13. NRC Generic Letter 92-01, Revision 1, Subject: Reactor Vessel Structural Integrity, 10 CFR 50.54(f), dated March 6, 1992.

14. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Reactor Vessel Structural Integrity, 10 CFR 50.54(f), Response to Generic Letter 92-01, Revision 1, dated July 2, 1992.

15. Generic Letter 92-01, Revision 1, Supplement 1, Subject: Reactor Vessel Structural Integrity, dated May 19, 1995.

16. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Response to NRC Generic Letter 92-01, Revision 1, Supplement 1, Reactor Vessel Structural Integrity, dated August 11, 1995.

17. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Six Month Response to NRC Generic Letter 92-01, Revision 1, Supplement 1, Reactor Vessel Structural Integrity, dated November 20, 1995.

18. Letter from G. S. Vissing, NRC, to R. C. Mecredy, RG&E, Subject: Closeout for RG&E Response to Generic Letter 92-01, Revision 1, Supplement 1 (TAC No. M92679), dated August 1, 1996.

19. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Use of Ratio Procedure in Determining RT_{PTS} Values for the R. E. Ginna Reactor Pressure Vessel, dated December 30, 1996.

20. Letter from G. S. Vissing, NRC, to R. C. Mecredy, RG&E, Subject: Request for Additional Information Regarding Reactor Pressure Vessel Integrity (TAC No. MA0546), dated April 3, 1998.

21. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to Request for Additional Information (RAI) Related to Reactor Pressure Vessel Integrity (TAC No. MA0546), dated August 25, 1998.

22. Letter from R. W. Kober, RG&E, to G. E. Lear, NRC, Subject: Pressurize Thermal Shock Rule - Six Month Submittal, dated January 13, 1986.

23. Letter from G. E. Lear, NRC to R. W. Kober, RG&E, Subject: Projected Values of Material Properties for Fracture Toughness Requirements for Protection Against Pressurized Thermal Shock Events, R. E. Ginna Nuclear Power Plant, dated November 17, 1986.

24. Letter from A. R. Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Pressurized Thermal Shock Evaluation (TAC No. M93827), dated March 22, 1996.

25. Dominion Engineering Inc. R-4814-00-01 Revision 1.0, Oct. 2006, "Reactor Vessel Tensioning Optimization Stress Report, Ginna Nuclear Power Plant."

26. Letter from M. Korsnick, CEG, to US NRC Document Control Desk, Subject: R.E. Ginna Nuclear Power Plant, License Amendment Request Regarding Extended Power

Uprate, (Attachment 5 — Licensing Report), dated July 7, 2005 (CMIS record ID "1001353").

27. WCAP-14684, R.E. Ginna Heatup and Cooldown Limit Curves for Normal Operation, June 1996.

28. Letter from T. Harding, Constellation Energy to U. S. NRC Document Control Desk, Subject: Reactor Vessel Surveillance Capsule "N" Results, Westinghouse Report WCAP-17036-NP, "Analysis of Capsule N from the R. E. Ginna Reactor Vessel Radiation Surveillance Program," Revision 0, dated May 29, 2009 (WPLNRC-1002144).

29. WCAP-17214-NP, R. E. Ginna Heatup and Cooldown Limit Curves for Normal Operation and Pressurized Thermal Shock Evaluation, Revision 0, dated July 26, 2010.

30. WCAP-17036-NP, Analysis of Capsule N from the R. E. Ginna Reactor Vessel Radiation Surveillance Program, Revision 1, dated September 14, 2010.

Table 5.3-1
REACTOR VESSEL SPECIFICATIONS

No.	**Specification**
1.	The American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Rules for Construction of Nuclear Vessels, 1965, and applicable code cases for Class A vessels. Code Cases: Upper Shell Course - 1332-1 Shell is fabricated of SA-336 manganese-molybdenum steel. Lower Head Ring - 1332-1 Ring is fabricated of SA-336 manganese-molybdenum steel.
2.	The American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section IX, Welding Qualifications, 1965.
3.	ASA B31.1, Code for Pressure Piping, Section VI, Chapter 3, 1955.
4.	Westinghouse Atomic Power Division Equipment Specification 676206 except as amended by Westinghouse Electric Corporation, Atomic Power Division Contract No. 54-Q-49758-BP, dated November 15, 1965.
5.	The Babcock & Wilcox Company, Quality Control Department Specifications covering the topics of welding, nondestructive testing, heat treating, cleaning, and testing.
6.	Replacement reactor vessel closure head provided by PCR 2001-0042 was designed in accordance with ASME Boiler and Pressure Vessel Code Section III, 1995 edition with 1996 addenda.

Table 5.3-2
REACTOR VESSEL DESIGN DATA

Design/operating pressure, psig	2485/2235
Hydrostatic test pressure, psig	3110
Design temperature, °F	650
Overall height of vessel and closure head, ft-in.	39-1.3
Water volume (with core and internals in place), ft^3	2473
Minimum thickness of insulation, in.	3.0
Number of reactor closure head studs	48
I.D. of flange, in.	121.81
Inlet Nozzle I.D., in.	27.47
I.D. at shell, in.	132.0
Outlet nozzle, I.D., in.	28.97
Core flooding water, nozzle, in.	3.5
Minimum clad thickness, in.	0.156
Minimum lower head thickness, in.	4.125
Minimum vessel beltline thickness, in.	6.5
Closure head thickness, in.	5.375

Table 5.3-3
REACTOR VESSEL MATERIALS

Section	Materials
Dome plate (bottom)	SA-533, grade A
Cylindrical forgings	SA-508, Class 2
Shell course, flanges, and nozzle forgings	SA-508, Class 2
Cladding (stainless weld rod)	Type 304 equipment
Thermal shield and internals	A-240, type 304
Replacement reactor vessel closure head (PCR 2001-0042)	SA-508, grade3, class 1 forging with cladding, 1st layer: 309L, subsequent layers: 308L

Table 5.3-4
IDENTIFICATION OF BELTLINE MATERIALS

WELDS

Weld Location	Weld Process	Weld Control Number	Weld Wire Type	Flux Type	Postweld Heat Treatment
Nozzle shell to intermediate shell	Submerged arc	SA-1101	Mn-Mo-Ni	Linde 80	1100-1125°F-48 hr-FC
Intermediate shell to lower shell	Submerged arc	SA-847	Mn-Mo-Ni	Linde 80	1100-1125°F-48 hr-FC
Surveillance weld	Submerged arc	SA-1036	Mn-Mo-Ni	Linde 80	1100°F-11-1/4 hr-FC

FORGINGS

				Heat Treatment		
Component	Forging Number	Material Specs	Supplier	Austenitize	Temper	Stress Relief
Nozzle shell	123P118VA1	A336	Bethlehem	1550°F-11 hr-WQ	1220°F-22 hr-AC	1125°F-30 hr-FC
Intermediate shell	125S255VA1	A508 CL2	Bethlehem	1550°F-15-1/2 hr-WQ	1210°F-18 hr-AC	1125°F-30 hr-FC
Lower shell	125P666VA1	A508 CL2	Bethlehem	1550°F-9 hr-WQ	1220°F-12 hr-AC	1125°F-30 hr-FC
Surveillance	125S255VA1	A508 CL2	Bethlehem	1550°F-15-1/2 hr-WQ	1210°F-18 hr-AC	1100°F-11-1/4 hr-FC
Forgings	125P666VA1	A508 CL2	Bethlehem	1550°F-9 hr-AC	1220°F-12 hr-AC	1100°F-11 hr-FC
Inlet nozzle	ZT 2254-2	A508 CL2	Midvale Hepenstall Co	Not available		
Forgings	ZT 2289-2	A508 CL2	Midvale Hepenstall Co.	Not available		

AC air cooled
WQ water quenched
FC furnace cooled

Table 5.3-5
BELTLINE MATERIAL CHEMICAL COMPOSITION (WEIGHT PERCENT)

Forging Number	C	P	S	Mn	Si	Mo	Ni	Cr	Cu	V
123P118VA1	0.19	0.010	0.009	0.65	0.23	0.60	0.69	0.42	---	---
125S255VA1	0.18	0.010	0.007	0.66	0.23	0.58	0.69	0.33	0.07	0.02
125P666VA1	0.19	0.010	0.011	0.67	0.20	0.57	0.69	0.37	0.05	0.02
ZT-2254-2	0.19	0.012	0.014	0.59	0.21	0.58	0.71	0.37	0.09	---
ZT-2289-2	0.20	0.011	0.014	0.66	0.20	0.60	0.69	0.30	0.09	---
Weld Control Number										
SA-1101	0.07	0.021	0.014	1.28	0.52	0.37	0.60	0.16	0.26	---
SA-847	0.080	0.012	0.012	1.34	0.45	0.38	0.54	0.08	0.25	---
Surveillance weld (SA-1036)	0.075	0.012	0.016	1.31	0.59	0.36	0.56	0.59	0.23	---

Table 5.3-6a
MECHANICAL PROPERTIES OF BELTLINE MATERIALS - FORGINGS

Parameter					
Forging Number	123P118VA1	125S255VA1	125P666VA1	125S255VA1 Surveillance test results	125P666VA1 Surveillance test results
T_{NDT} °F	30	20	40	20	40
RT_{NDT} °F[a]	30	20	40	20	40
Upper Shelf Energy (ft-lb[a])	117	106	114	91	120
Yield strength ksi	66.87	67.25	63.50	78.22	62.72
Ultimate tensile strength ksi	88.00	88.25	85.00	97.19	83.65
Elongation (%)	25.50	26.25	26.25	23.30	26.35
RA (%)	73.50	70.10	71.05	66.85	70.75

a. Estimated based on NRC Standard Review Plan Section 5.3.2 and MTEB 5-2.

Table 5.3-6b
MECHANICAL PROPERTIES OF BELTLINE MATERIALS

Parameter			
Weld Control Number	SA-1484	SA-1101	Surveillance weld
Weld Wire Type	Mn-Mo-Ni	Mn-Mo-Ni	---
Flux Type	Linde 80	Linde 80	---
T_{NDT} °F[a]	0	0	0
Energy at 10°F ft-lb	45, 45, 46	58, 60, 36	54, 66.5, 71[b]
$R_{T\ NDT}$ °F	0[a]	0[a]	-19.5[c]
Shelf Energy (ft-lb)	---	---	79.0
Yield strength ksi	68.63	67.00	73.52
Ultimate tensile strength ksi	84.26	81.88	87.35
Elongation (%)	28.5	29.5	22.8
RA (%)	---	0	62.0

a. Estimated based on NRC Standard Review Plan Section 5.3.2 and MTEB 5-2.

b. Energy at 60°F.

c. Mean value from data in BAW-1803, Revision 1 and BAW-1920P

Table 5.3-7
SUMMARY OF PRIMARY-PLUS-SECONDARY STRESS INTENSITY FOR COMPONENTS OF THE REACTOR VESSEL

Location	Maximum Range of Stress Intensity (ksi)	
	Stress Intensity (ksi)	Allowable Stress $3S_m$ at Operating Temperature (ksi)
Closure Head at Flange	52.9[ab]	80.1
Vessel at Flange	52.3[a]	80.1
Closure Studs	97.2[a]	104.1[ac]
CRDM Nozzle CRDM Nozzle J-weld	79.0/35.6[d] 63.8	69.9[d] 69.9
Vent Nozzle Vent Nozzle J-weld	31.9 41.9	41.1 69.9
Outlet Nozzle Safe End Nozzle Support Pad	 39.9 49.2 n/a[e]	 49.2 80.1 n/a[e]
Inlet Nozzle Safe End Nozzle Support Pad	 35.8 38.8 n/a[e]	 49.2 80.1 n/a[e]
Safety Injection Nozzle	55.4	80.1
Vessel Wall Transition	32.2	80.1
Bottom Head to Shell Juncture	28.6	80.1<
Bottom Head Instrumentation Nozzle	37.1	69.9
Core Support Pad	52.5	69.9
External Support Bracket	41.2	80.1

a. Values reported from the Reactor Vessel Tensioning Optimization Stress Report, (Reference 25).

b. "Closure Head at Flange" historic location reported is at the junction of closure dome plate section to the forged flange ring section. The original closure head design included a weld at this location. The replacement Reactor Vessel Closure Head for the Ginna Reactor installed under PCR 2001-0042, is a one piece forging and the weld at the dome to flange location was eliminated. The value reported herein, is taken from the Reactor Vessel Tensioning Optimization Stress Report at the closure head flange mating surface and is reported as bounding for all stress levels in the head model used for the tensioning optimization stress report.

c. Value reported is at 2.7 Sm.

d. For the CRDM tube, the allowable range of stress is exceeded (79.0) but this is permissible since the range excluding thermal bending (35.6) is below $3S_m$.

e. The nozzle at the support pad is considered a peak stress location and consequently, only fatigue is considered at that location.

Table 5.3-8
SUMMARY OF CUMULATIVE FATIGUE USAGE FACTORS FOR COMPONENTS OF THE REACTOR VESSEL

Location	Cumulative Fatigue Usage Factor
Closure Head at Flange Vessel at Flange Closure Studs	0.386[a] 0.264[a] 0.972[a]
CRDM Nozzle Nozzle J-Weld	0.580 0.742
Vent Nozzle Nozzle J-Weld	0.009 0.494
Outlet Nozzle Safe End Nozzle Forging Support Pad	 n/a[b] 0.044 0.386
Inlet Nozzle Safe End Nozzle Forging Support Pad	 n/a[b] 0.033 0.061
Safety Injection Nozzles	0.219
Vessel Wall Transition	0.003
Bottom Head to Shell Juncture	0.002
Bottom Head Instrumentation Nozzle	0.228
Core Support Guides	0.132
External Support Brackets	0.979

a. Revised cumulative usage factors are provided from Reactor Vessel Tensioning Optimization work, (Reference 25).

b. Cumulative fatigue usage factors were not reported for the safe ends of the outlet and inlet nozzles because the nozzle-to-shell junction, not the safe end, was found to be the worst fatigue location.

Table 5.3-9
SUMMARY OF SURVEILLANCE CAPSULE RESULTS

Material	Copper (wt. %)	30 ft-lb Temperature Shift After Fluence				
		[a] 5.87×10^{18} n/cm^2	[b] 1.02×10^{19} n/cm^2	[c] 1.69×10^{19} n/cm^2	[d] 3.64×10^{19} n/cm^2	[e] 5.8 x 1019 n/cm2
Weld SA-1036	0.23	146.7 °F	156.2 °F	149.7 °F	212.2 °F	216.9°F
Forging 125P666VA1	0.05	34.7 °F	57.5 °F	33.6 °F	45.8 °F	91.1°F
Forging 125S255VA1	0.07	0 °F	20.1 °F	0 °F	76.8 °F	76.4°F

a. Analysis of Capsule V, *Reference 30*.

b. Analysis of Capsule R, *Reference 30*

c. Analysis of Capsule T, *Reference 30*.

d. Analysis of Capsule S, *Reference 30*

e. Analysis of Capsule N, *Reference 30*

Table 5.3-10
COMPARISON OF SURVEILLANCE MATERIAL 30 FT-LB TRANSITION TEMPERATURE SHIFTS AND UPPER SHELF ENERGY DECREASES WITH REGULATORY GUIDE 1.99, REVISION 2, PREDICTIONS

			30 ft-lb Transition Temperature Shift		Upper Shelf Energy Decrease	
Material	**Capsule**	**Fluence ($x\ 10^{19}\ n/cm^2$)**	**Predicted[a] (°F)**	**Measured (°F)[b]**	**Predicted[a] (%)**	**Measured (%)**
Intermediate Shell Forging 125S255 (Longitudinal)	V	0.587	37.4	0.0[c]	14.5	3.7
	R	1.02	44.2	20.1	16.5	-1.6
	T	1.69	50.4	0.0[c]	19	-8.8
	S	3.64	58.8	76.8	23	0.7
	N	5.8	62.9	76.4	26	4.1
Lower Shell Forging 125P666 (Longitudinal)	V	0.587	26.4	34.7	13	10.1
	R	1.02	31.2	57.5	15	15.4
	T	1.69	35.5	33.6	17	18.4
	S	3.64	41.4	45.8	21	18.4
	N	5.8	44.3	91.1	23	18.5
Weld Metal (Heat # 61782)	V	0.587	135.2	146.7	32.5	30.1
	R	1.02	159.7	156.2	37	38.1
	T	1.69	181.8	149.7	40.5	33.3
	S	3.64	212.1	212.2	49	33.9
	N	5.8	227.2	216.9	54	34.3
HAZ Material	V	0.587	---	30.7	---	-50.0
	R	1.02	---	58.6	---	8.0
	T	1.69	---	41.0	---	-30.8
	S	3.64	---	38.9	---	-15.0
	N	5.8	---	107.7	---	1.9

a. Based on Regulatory Guide 1.99, Revision 2, methodology using mean wt % values of copper and nickel.

b. Calculated by CVGraph Version 5.3 using measured Charpy data

c. ΔRT_{NDT} value was determined to be negative, but physically a reduction should not occur, therefore a conservative valueof zero is not used.

5.4 COMPONENT AND SUBSYSTEM DESIGN

5.4.1 REACTOR COOLANT PUMPS

5.4.1.1 General Description

5.4.1.1.1 Centrifugal Pump

Each reactor coolant loop contains a vertical single-stage centrifugal type pump, which employs a controlled leakage seal assembly. A view of a controlled leakage pump is shown in Figure 5.4-1 and the principal design parameters for the pumps are listed in Table 5.4-1. The reactor coolant pump estimated performance and net positive suction head characteristics are shown in Figure 5.4-2. The performance characteristics are common to all of the higher specific speed centrifugal pumps and the "knee" at about 45% design flow introduces no operational restrictions, since the pumps operate at full speed.

The reactor coolant pump performance characteristics were updated as a result of the conversion to an 18 month fuel cycle to incorporate the currently installed internals and impellers. The updated characteristics are illustrated on Figures 5.4-2a through 5.4-2d for both hot and cold conditions. The tabular data upon which the curves are based is shown as Tables 5.4-3 and 5.4-4, as submitted by *Reference 56*.

Reactor coolant is pumped by the impeller attached to the bottom of the rotor shaft. The coolant is drawn up through the impeller, discharged through passages in the diffuser and out through a discharge nozzle in the side of the casing. The motor-impeller can be removed from the casing for maintenance or inspection without removing the casing from the piping. All parts of the pumps in contact with the reactor coolant are austenitic stainless steel or equivalent corrosion resistant materials.

5.4.1.1.2 Controlled Leakage Shaft Seal

The pump employs a controlled leakage seal assembly to restrict leakage along the pump shaft, as well as a secondary seal that directs the controlled leakage out of the pump, and a third seal that minimizes the leakage of water and vapor from the pump into the containment atmosphere.

A portion of the high-pressure water flow from the charging pumps is injected into the reactor coolant pump between the impeller and the controlled leakage seal. The shaft seal arrangement is shown in Figure 5.4-3. Part of the flow enters the reactor coolant system through a labyrinth seal in the lower pump shaft to serve as a buffer to keep reactor coolant from entering the upper portion of the pump. The remainder of the injection water flows along the drive shaft, through the controlled leakage seal, and finally out of the pump. A very small amount that leaks through the secondary seal is also collected and removed from the pump.

The original seal material for the reactor coolant pump controlled leakage (number one) seal assembly was aluminum oxide. These seals were replaced during pump maintenance during the 1991 and 1997 refueling outages. The new seal assemblies contain silica nitrate. The new material was an improvement over aluminum oxide since silica nitrate seals are capable of surviving minor rubbing of the seal faces with no degradation in seal performance. New

high temperature o-rings have also been installed on both reactor coolant pump seal assemblies.

Component cooling water (CCW) is supplied to the motor bearing oil coolers and the thermal barrier cooling coil. Motor bearing lube-oil level indication is provided in the control room.

Reactor coolant pump seal operation requires one of two water sources for operation. The normal supply to the seals is cooled and filtered seal injection water from the charging system. If seal injection were lost, the reactor coolant system water would pass up through the labyrinth seal and thermal barrier to the number one seal. This water is unfiltered and is cooled by the thermal barrier by component cooling water. Since it is unlikely that both seal injection and component cooling water would be terminated, if termination did occur, in most cases, the reactor would be shutdown and the reactor coolant pumps tripped.

Essential services for reactor coolant pump operation are available during a containment isolation signal unless a safety injection signal occurs with a loss of offsite power. Seal injection from the chemical and volume control system is terminated by a charging pump trip upon receipt of a safety injection signal. However, component cooling water (CCW) services to the reactor coolant pump remain in operation independent of the safety injection and/or containment isolation signals, unless offsite power is lost. A loss of offsite power coincident with a safety injection signal will trip the component cooling water (CCW) pumps, thereby terminating component cooling water (CCW) flow to the reactor coolant pumps. Since the reactor coolant pumps operate from offsite power, the reactor coolant pumps will also be tripped and will not be available while offsite power is lost.

An extensive test program was conducted for several years to develop the controlled leakage shaft seal for pressurized water reactor applications. Long-term tests were conducted on less than full scale prototype seals as well as on full size seals. At the time of initial operation of Ginna Station, operating experience with large size, controlled leakage shaft seal pumps was available from plants such as San Onofre Unit 1 and Connecticut Yankee.

5.4.1.1.3 Pump Motor

The squirrel cage induction motor driving the pump is air cooled and has oil-lubricated thrust and radial bearings. A water-lubricated bearing provides radial support for the pump shaft.

5.4.1.1.4 Vibration Measurement

Each pump is equipped with two vibration pickups (seismic displacement) mounted at the bottom of the motor casings and two shaft (non-contact) pickups mounted below the coupling on the seal housing, located 90 degrees apart, to determine pump shaft vibration. Vibration levels are checked periodically or whenever an abnormal condition is expected. A keyphasor proximity transducer is provided to supply vibration information for diagnostic testing. The data includes dynamic and static vibration data, which is digitized and transmitted to the general purpose computer where it can be reduced and displayed in the form of plots, alarm lists, reports, and logs.

5.4.1.1.5 Lube Oil Leakage Collection System

A system of drip pans, splash guards, and enclosures for collecting reactor coolant pump lube-oil leakage is installed on each reactor coolant pump to reduce the potential for fire caused by lube-oil contacting and igniting on hot reactor coolant system components. Drain piping from seven collection points on each reactor coolant pump directs leakage to an oil collection system storage tank.

5.4.1.2 Pump Flywheel Integrity

5.4.1.2.1 Pump Overspeed

Precautionary measures, taken to preclude missile formation from primary coolant pump components, ensure that the pumps will not produce missiles under any anticipated accident condition.

The primary coolant pumps run at 1189 rpm and may operate briefly at overspeeds up to 109% (1295 rpm) during loss of external load. At 1189 rpm, the bore stress due to rotation is 14,000 psi. For conservatism, however, 125% of operating speed was selected as the design overspeed for the primary coolant pumps. The maximum pump overspeed on loss of external load is 118% based on turbine overspeed with failure of the turbine steam control valve.

For the overspeed condition, which would not persist for more than 30 sec, pump operating temperatures would remain at about the design value. Furthermore, the probability of attaining a post-loss-of-coolant accident overspeed sufficient to cause loss of flywheel integrity is very remote. This probability would be the product of the conditional probabilities of (1) the break of a large primary coolant pipe, (2) the failure of associated pipe restraints such that the break could become a double-ended guillotine break (calculations show a significantly smaller overspeed for a realistic break), and (3) the loss of electric power to the pump motor such that there is no electric braking effect and the pump is permitted to accelerate freely.
Also, the pump would have to remain free-spinning; seizure of the shaft or motor components could prevent overspeed.

Each component of the primary pumps has been analyzed for missile generation. Any fragments would be contained by the heavy stator. The same conclusion applies to the impeller because the small fragments that might be ejected would be contained by the heavy casing. At an overspeed of 1486 rpm, the maximum tangential stress reaches 21,500 psi, which is less than 50% of the minimum yield strength (50,000 psi) at the operating temperature.

5.4.1.2.2 Pump Flywheel Design and Fabrication

The primary coolant pump flywheels are shown in Figure 5.4-4. As for the pump motors, the most adverse operating condition of the flywheels is visualized to be the loss-of-load situation. The following conservative design operating conditions preclude missile production by the pump flywheels. The wheels are fabricated from rolled, vacuum-degassed, ASTM A-533 steel plates. Flywheel blanks are flame-cut from the plate, with allowance for exclusion of flame affected metal. A minimum of three Charpy V-notch tests are made from each plate parallel and normal to the rolling direction, to determine that each blank satisfies design requirements. A nil ductility transition temperature (NDTT) less than +10°F is specified.

The flywheel material has a minimum yield strength of 50,000 psi and tensile strength of 80,000 psi. The finished flywheels are subjected to 100% volumetric ultrasonic inspection. The finished machined bores are also subjected to magnetic particle or liquid penetrant examination. The pump flywheels are mounted on a shaft of radius 4.2 in. and consist of two large steel disks bolted together. The disks are 75 in. and 65 in. in diameter.

These design-fabrication techniques yield flywheels with primary stress at operating speed (shown in Figure 5.4-5) less than 50% of the minimum specified material yield strength at room temperature (100°F to 150°F). Bursting speed of the flywheels has been calculated by Ginna, on the basis of Griffith-Irwin's results, (*Reference 1*) to be 3900 rpm. The NRC staff has independently determined the bursting speed for the flywheel to be 3400 rpm (*Reference 2*). Regulatory Guide 1.14 requires that the margin against ductile failure relative to the minimum specified yield strength be 3 and 1.5 at normal operating speed and design overspeed, respectively. For the Ginna Station flywheels, the margin is 3.7 at normal operating speed and 2.33 at a design overspeed of 125%. Therefore, they have a wide margin of safety against ductile failure and the requirements of Regulatory Guide 1.14 are satisfied.

5.4.1.2.3 Flywheel Design Evaluation

A fracture mechanics evaluation was made on the reactor coolant pump flywheel. This evaluation considered the following assumptions:

A. Maximum tangential stress at an assumed design overspeed of 125% compared to a maximum expected overspeed of 109%.

B. A crack through the thickness of the flywheel at the bore.

C. 400 cycles of startup operation in 40 years.

Using critical stress intensity factors and crack growth data attained on flywheel material, the critical crack size for failure was greater than 17 in. radially and the crack growth data was 0.030 in. to 0.060 in. per 1000 cycles.

The NRC staff performed an independent fracture mechanics evaluation to determine the speed at which unstable crack propagation would occur for a 4-in. crack emanating from the keyway. The results of the evaluation showed that a 4-in. crack would remain stable at speeds up to 3000 rpm. Therefore, a very large crack, on the order of 10 in., would remain stable at an overspeed of 1486 rpm (*Reference 2*).

5.4.1.2.4 Pump Seismic Design

The original design specifications for the reactor coolant pumps include as a design condition the stresses generated by a maximum hypothetical earthquake ground acceleration of 0.2g. Within the scope of SEP Item III-6, the reactor coolant pump was analyzed with respect to stresses induced by 0.8g horizontal and 0.54g vertical loadings. It was shown that the stresses are lower than the ASME Code allowable and that the reactor coolant pumps have acceptable seismic resistance.

5.4.1.2.5 Inservice Inspection Program

The inservice inspection program for the installed reactor coolant pump (RCP) flywheels consists of either an ultrasonic (UT) examination over the volume from the inner bore of the flywheel to the circle of one-half the outer radius or conduct an ultrasonic (UT) and a surface (MT and/or PT) examination of exposed surfaces defined by the volume of the disassembled flywheels once every 20 years *(Reference 66)*. The Ginna Station reactor coolant pump flywheels meet the requirements of Regulatory Guide 1.14. Additional information on the overall inservice inspection program is contained in the Inservice Inspection (ISI) Program document.

5.4.1.2.6 Conclusion

Following a hypothetical bearing seizure, the flywheel is not expected to twist off. Therefore, it has been concluded that the reactor coolant pumps are not sources of missiles and the engineered safety features are not in jeopardy.

5.4.2 STEAM GENERATORS

Each loop contains a vertical shell and U-tube steam generator. A steam generator of this type is shown in Figure 5.4-6. Principal design parameters are listed in Table 5.4-2. The steam generators are designed and manufactured in accordance with Section III of the ASME Boiler and Pressure Vessel Code (see Section 3.2).

Reactor coolant enters the inlet side of the channel head at the bottom of the steam generator through the inlet nozzle, flows through the U-tubes to an outlet channel, and leaves the generator through another bottom nozzle. The inlet and outlet channels are separated by a partition. Manways are provided to permit access to the U-tubes and moisture separating equipment.

Feedwater to the steam generator enters just above the top of the U-tubes through a feedwater ring. The water flows downward through an annulus between the tube wrapper and the shell and then upward through the tube bundle where part of it is converted to steam. The steam water mixture from the tube bundle passes through centrifugal type separators which impart a centrifugal motion to the mixture and separate the water particles from the steam. The water leaves the separator through the separator return cylinder and combines with the feedwater for another pass through the tube bundle. The steam rises through additional separators which limit the moisture content of the steam to 0.1% or less at design load conditions.

Steam generator performance gradually deteriorates over time. A pressure transmitter is installed in each steam generator to monitor pressure in the upper portion of the generator. The pressure signals are monitored by the plant process computer system and used to trend and analyze degradation of steam generator performance.

A loose-parts monitoring system was installed for each steam generator in 1982. The system is designed to indicate the presence of potentially damaging foreign objects in either the primary channel head or the secondary side of the tubesheet. Information specific to the secondary side of the steam generator is contained in Section 10.3.2.

5.4.2.1 Replacement Steam Generator Materials

The steam generator pressure boundary is constructed of ferritic steel, either carbon steel or low alloy. The heat transfer tubes are SB-163 Alloy 690 as permitted by ASME Code Case N-20-3. The interior surfaces of the channel heads and nozzles are clad with austenitic stainless steel, and the side of the tubesheet in contact with the reactor coolant is clad with Alloy 600 weld metal. The primary head is a single forging of SA-508 Cl 3 material with integrally forged manways and inlet/outlet nozzles. There are stainless steel safe ends of SA-336-316N/ 316LN welded to each of these nozzles. The tubesheet is also a SA-508 Cl 3 forging. The divider plate is machined from SB-168 Alloy 690 plate and welded around its entire periphery to either the primary head or the tubesheet. The tubes are welded to the cladding on the tubesheet face after which each tube is hydraulically expanded through the full tubesheet thickness. The tubes are supported by lattice bars and U-bend flat bars made of SA-240 Type 410S stainless steel.

5.4.2.2 Steam Generator Inservice Inspection

Inservice inspection of steam generators is conducted in accordance with the Inservice Inspection (ISI) Program document. A program of periodic steam generator inspections, designed to meet the Ginna Technical Specifications and EPRI PWR Steam Generator Program documents, is conducted to provide assurance of acceptable steam generator performance. The inservice inspection program for the reactor coolant pressure boundary is discussed in Section 5.2.4. As part of the response (*Reference 64*) to NRC Generic Letter 97-06 (*Reference 65*), RG&E committed to develop a secondary side inspection program to ensure that degradation of steam generator internals does not adversely affect tube integrity, as defined within the Steam Generator Program.

5.4.2.3 Replacement Steam Generator Design Evaluation

Structural and seismic evaluation of replacement steam generators primary and secondary side pressure boundaries demonstrate that these components satisfy ASME III, Division 1, Class I design requirements for service levels A, B, C, and D (normal, upset, emergency and faulted conditions, respectively). Steam generator internal components are not governed by the ASME Boiler & Vessel Code. However, ASME III Subsection NB for Class 1 components is used as a guide for structural analysis of RSG internal components. RSG internal components are required to withstand all specified loadings to maintain heat transfer capability during and following a design basis earthquake. In addition, tubes must be shown not to deform as a result of a design basis earthquake. This helps to ensure that safe shutdown capability is maintained. The RSG structural evaluation is documented in a Code Stress Report.

The structural analysis demonstrates that for an instantaneous full rupture of the steam line downstream of the steam outlet nozzle occurring during normal full power operation, the tube integrity is maintained. The structural evaluation of the tubing for level D is in accordance with the ASME Boiler and Pressure Vessel Code Section III requirements. Tube Integrity must also be maintained for a small steam line break to Level C criteria.

A flow-induced vibration (FIV) analysis is performed to confirm that the tube bundle is adequately supported to avoid significant levels of tube vibration. An FIV Analysis Report

and a Wear Analysis Report verify that the vibration of the RSG internals does not result in excessive wear or fatigue throughout the tube bundle and U-bend regions.

The three pertinent cross-flow FIV mechanisms in the RSG are vortex shedding resonance, random turbulence excitation, and fluid elastic instability. The FIV analysis verifies that excessive tube vibration from these sources is avoided. Particular areas of emphasis are the tube bundle entrance and the U-bend region.

5.4.2.4 High Cycle Fatigue Failure of Original Steam Generator Tubes

This section was applicable to the original Westinghouse Model 44 steam generators which were replaced in 1996 and does not apply to the BWI replacement steam generators. This section is retained for historical purposes.

NRC Bulletin 88-02 (*Reference 11*) requested that holders of operating licenses or construction permits for Westinghouse reactors with carbon steel support plates implement actions to minimize the potential for a steam generator tube rupture event caused by a rapidly propagating fatigue crack, such as occurred at North Anna Unit 1 on July 15, 1987. North Anna experienced a circumferential tube rupture at the top of the top tube support plate, which was attributed to limited displacement, fluid elastic instability. The unstable condition was caused by tube denting at the support plate. The Ginna steam generator tubes were examined and analyzed to determine their susceptibility to high cycle fatigue failure (*References 50* and *51*). Those tubes identified as being potentially susceptible to high cycle fatigue or susceptible to the consequences of fatigue failure were stabilized unless they had been previously plugged. This action was found acceptable by the NRC (*Reference 12*). Subsequent to the initial evaluation, Westinghouse performed a re-evaluation based on updated information and, as a result, additional Ginna Station steam generator tubes were plugged and stabilized during the 1992 MODE 6 (Refueling) outage (*References 13* and *52*).

The NRC concluded that the actions taken in response to Bulletin 88-02 were acceptable as long as administrative controls were developed that ensured updated stress ratio and fatigue usage calculations were performed in the event of any significant changes to the steam generator operating parameters (*Reference 53*). As a result, during startup core physics testing coming out of a MODE 6 (Refueling) outage, steam generator pressure was verified to be greater than 675 psig and if steam generator pressure fell below 675 psig, power escalation was terminated. In addition, during plant coastdown operations, steam generator pressure was maintained above 660 psig (675 psia).

5.4.3 REACTOR COOLANT PIPING

5.4.3.1 General

<u>5.4.3.1.1 General Description</u>

The general arrangement of the reactor coolant system piping is described in Section 5.1.1. Piping design data are presented in Table 5.1-2.

The austenitic stainless steel reactor coolant piping and fittings which make up the loops are 29 in. I.D. in the hot legs, 27-1/2 in. I.D. in the cold legs, and 31 in. I.D. between the steam

generators and the reactor coolant pump inlet. The pressurizer relief line, which connects the pressurizer safety and pressurizer power operated relief valves (PORVs) to the discharge nozzle flange on the pressurizer relief tank, is constructed of carbon steel.

Smaller piping, including the pressurizer surge and spray lines, drains, and connections to other systems, is austenitic stainless steel. All joints and connections are welded except for stainless steel flange connections to the carbon steel pressurizer relief tank and the connections at the relief and safety valves.

Thermal sleeves are installed at the following locations where high thermal stresses could otherwise develop due to rapid changes in fluid temperature during MODES 1 and 2 transients:

- Return line from the residual heat removal loop.
- Both ends of the pressurizer surge line.
- Pressurizer spray line connection to the pressurizer.
- Charging lines and alternate charging line connections.

All valve surfaces in contact with reactor coolant are austenitic stainless steel or equivalent corrosion resistant materials. Connections to stainless steel piping are welded.

All relief valves used in systems handling radioactive fluids are of the closed bonnet design and are constructed of stainless steel.

5.4.3.1.2 Pressure Isolation of Low-Pressure Systems

Three systems have a direct interface with the reactor coolant system pressure boundary but have a design pressure rating below that of the reactor coolant system. These systems are the chemical and volume control system (Section 9.3.4), the safety injection system (Section 6.3), and the residual heat removal system (Section 5.4.5). The isolability of the low-pressure systems from the reactor coolant system is discussed in the respective sections.

In response to Generic Letter 87-06, *Reference 16* lists all pressure isolation valves (PIV) that separate the high pressure reactor coolant system from attached lower pressure systems and the periodic tests or other measures performed to ensure the integrity of the isolation valves as an independent barrier at the reactor coolant pressure boundary.

5.4.3.2 Reactor Coolant System Vents

5.4.3.2.1 General

The requirements for reactor coolant system high point vents are stated in 10 CFR 50.44, Standards for Combustible Gas Control Systems in Light Water Cooled Power Reactors. They are further described in Standard Review Plan Section 5.4.12, Reactor Coolant System High Point Vents, and in Item II.B.1 of NUREG 0737, Clarification of TMI Action Plan Requirements. In response to these and previous requirements, RG&E has submitted information in *References 17 through 21* in support of the vent system at Ginna Station.

The function of the high point vent system is to vent noncondensible gases from the high points of the reactor coolant system to ensure that core cooling during natural circulation will not be inhibited. The Ginna Station reactor vessel head vent system provides venting capability from the reactor vessel head while the pressurizer can be vented through the existing pressurizer power operated relief valves (PORVs). The noncondensible gases, steam, and/or liquids vented from the reactor vessel head are piped and discharged directly to the refueling cavity and the discharges from the pressurizer are piped to the pressurizer relief tank. The reactor vessel head vent system is designed to vent a volume of gas at least equal to one half of the reactor coolant system volume in 1 hour. Flow restriction orifices in the reactor vessel head vent system paths, however, limit the flow from a pipe rupture or from inadvertent actuation of the vent system to less than the capability of the reactor coolant makeup system.
Compliance with 10 CFR 50.46, Acceptance Criteria for Emergency Core Cooling Systems (ECCS) for Light Water Nuclear Power Reactors, is not affected by the addition of the reactor vessel head vent system.

5.4.3.2.2 Reactor Head Vent System Description

The reactor vessel head vent system consists of two redundant vent paths from the reactor vessel head to the refueling cavity, each containing a manually operated valve followed by two solenoid-operated valves in series that are remotely controlled from the main control room. The two paths are connected to a single 3/4-in. reactor head vent pipe downstream of a manually operated valve. A degree of redundancy has been provided by powering each reactor vessel head vent system vent path from a separate emergency bus to ensure that reactor coolant system venting capability from the reactor vessel head is maintained. Reactor vessel head vent system valve seat leakage is detected, together with other unidentified reactor coolant system leakage, by way of containment radiation (R-11 and R-12) and containment sump A level monitoring (LT-2039, LT-2044, and sump pump actuation) in accordance with the Technical Specifications. The manual valves in each vent path provide a means of isolating that path in the event of leakage of the normally closed solenoid valves. The pressurizer power operated relief valves (PORVs), used to vent the pressurizer, function as a part of the automatic reactor coolant system pressure control system but they can also be manually controlled from the main control room. The pressurizer power operated relief valves (PORVs) and block valves receive power from emergency buses and have positive valve position indication in the main control room. The portion of each reactor vessel head vent system path up to and including the second normally closed valve forms a part of the reactor coolant pressure boundary and thus must meet reactor coolant pressure boundary requirements. Therefore, the piping out to the flow restriction orifices is ASME Code, Section III, Class 1, and the system beyond the orifices to the second vent valves is ASME Code, Section III, Class 2, in compliance with 10 CFR 50.55a and Regulatory Guide 1.26. The entire reactor vessel head vent system is designated Seismic Category I in compliance with Regulatory Guide 1.29. The reactor vessel head vent system is designed for pressures and temperatures corresponding to the reactor coolant system design pressure and temperature.

In addition, the vent system materials are compatible with the reactor coolant chemistry and were fabricated and tested in accordance with ASME Code, Section III, subsections NB, NC, and NF, and plant specifications. The reactor vessel head vent system and the pressurizer power operated relief valve (PORV) vent system are separated and protected from missiles

and the dynamic effects of postulated piping ruptures. The design of the portions of the reactor vessel head vent system up to and including the second normally closed valve conforms to all reactor coolant pressure boundary requirements, including 10 CFR 50.55a and the applicable portions of General Design Criteria 1, 2, 4, 14, 30, and 31. The essential operation of other safety-related systems will not be impaired by postulated failures of reactor vessel head vent system components.

The reactor vessel head vent system design has been reviewed to ensure an acceptably low probability for inadvertent or irreversible actuation of the vent system. Each vent path has two solenoid-operated globe valves in series, and each valve has a separate key-locked control switch that is locked closed during normal reactor operation. The valves are powered by 125-V dc emergency power supplies and fail to the closed position in the event of loss of power. No single active component failure or human error should result in inadvertent opening or failure to close after intentional opening of the reactor vessel head vent system.

The locations where the reactor vessel head vent system normally discharges to the containment atmosphere in the vicinity of the refueling cavity are in areas that ensure good mixing with the containment atmosphere to prevent the accumulation or pocketing of high concentrations of hydrogen in compliance with 10 CFR 50.44, Standards for Combustible Gas Control System in Light Water Cooled Power Reactors. Additionally, these locations are such that the operation of safety-related systems would not be adversely affected by the discharge of the anticipated mixtures of steam, liquids, and noncondensible gases.

The reactor vessel head vent system valves are exercised periodically and proper valve position is visually verified. Operability testing of the pressurizer power operated relief valves (PORVs) and block valves is specified in the Ginna Pump and Valve Inservice Testing Program and is in accordance with the Code for Operation and Maintenance of Nuclear Power Plants.

Reactor vessel head vent system parameters are given in Table 5.4-5.

5.4.4 MAIN STEAM LINE ISOLATION SYSTEM

Each steam line has a fast-closing Main Steam Isolation Valve (MSIV) and a Main Steam non-return check valve. These four valves prevent blowdown of more than one steam generator for any break location even if one valve fails to close (Section 15.1.5. See also Sections 10.3.2.6 and 10.3.2.7).

The main steam isolation valves are 30-in. pipe size, 24-in. seat diameter, ANSI 600-lb rating, Atwood and Morrill Company, Inc., swing-disk check valves. The open position of the disk is at full horizontal held open against the steam flow by an air cylinder. The valves have stainless steel disks and disk arms. The stiffness of the disk arms is designed to reduce valve strains developed during closure following a postulated downstream pipe break. The disks and disk arms are also designed to uniformly transfer the kinetic energy from the disk to the valve body during impact. The valve disks and disk arms are stainless steel in order to better withstand the local strains in the contact region. The overall design of the valve will reduce the likelihood of damage due to spurious closure and will prevent excessive degradation of the valves during normal service.

The main steam isolation valves serve to limit an excessive reactor coolant system cooldown rate and resultant reactivity insertion following a main steam line break incident. Their ability to close upon signal is verified at periodic intervals. A closure time of 5 sec was selected as being consistent with expected response time for instrumentation as described in the steam line break incident analysis.

The purpose of the fast acting valves is to prevent continuous blowdown from more than one steam generator following any steam line rupture even with failure of any single check or isolation valve. Flow from a second steam generator for up to 7 sec (including 2 sec for instrument response time) following a steam line break has a negligible effect on the peak core power eventually attained from continuous blowdown of one steam generator. The main effect of flow from the second steam generator is to reduce the pressure faster during the initial portion of the transient, thereby causing safety injection flow to occur earlier. Flow from the second steam generator has little effect on the reactivity insertion rate, which occurs after the reactor pressure has fallen to the safety injection pump shutoff head, since by this time the isolation valve has closed.

It should be noted that 5 sec is the maximum allowable closure time for the valves with no flow passing through them. Tests with no flow have shown that closure time to be less than 5 sec. With the flow, which will exist through a valve following a steam line rupture, the closure time will be considerably faster than 5 sec since flow will tend to force the valve to its closed position.

The steam line break accident analysis is presented in detail in Section 15.1.5. The main steam isolation valves are tested in accordance with the Technical Specifications.

5.4.5 RESIDUAL HEAT REMOVAL (RHR) SYSTEM

5.4.5.1 Design Bases

The residual heat removal loop is designed to remove residual and sensible heat from the core and reduce the temperature of the reactor coolant system during the second phase of plant cooldown. During the first phase of cooldown, the temperature of the reactor coolant system is reduced by transferring heat from the reactor coolant system to the steam and power conversion system.

All active loop components which are relied upon to perform their function are redundant, except as described in Section 5.4.5.3.4.

The loop design provides means to detect radioactivity migration to the ultimate heat sink environment and includes provisions, which initiate adequate action for continued core cooling when required, in the event radioactivity limits are exceeded.

The loop design precludes any significant reduction in the overall design reactor shutdown margin when cooling water is introduced into the core for decay heat removal during the emergency core cooling recirculation mode of operation.

The loop design includes provisions to enable periodic hydrostatic testing to applicable code

test pressure. Loop components, whose design pressure and temperature are less than the reactor coolant system design limits, are provided with redundant isolation means and overpressure protective devices.

5.4.5.2 System Design

The residual heat removal loop consists of two heat exchangers, two pumps, piping, and the associated valves and instrumentation (Drawing 33013-1247). After the steam generators have been used to reduce the reactor coolant temperature to 350°F, decay heat cooling is initiated by aligning the residual heat removal pumps to take suction from the reactor coolant system loop A hot leg and discharge through the residual heat removal heat exchangers to the loop B cold leg. With both pumps and heat exchangers in operation, residual heat removal flow is adjusted to maintain a cooldown rate of less than 80°F/hr. If only one pump and heat exchanger are available, cooldown is accomplished at a lower rate.

The heat from the residual heat removal heat exchangers is transferred to the component cooling water (CCW) system (Section 9.2.2), and from the component cooling water (CCW) system to the service water (SW) system (Section 9.2.1). The minimum pump head on the residual heat removal pumps is 150 psig, the component cooling water system operating pressure is 80 psig, and the service water (SW) system operating pressure is 75 psig; therefore, in the event of a residual heat removal heat exchanger tube leak, the flow of impurities should be away from the primary system.

During plant shutdown, the cooldown rate of the reactor coolant is controlled by regulating the flow through the tube side of the residual heat removal heat exchangers. A bypass line and control valve around the residual heat exchangers are used to maintain a constant flow through the residual heat removal loop. To minimize the potential for flow-induced vibration in the residual heat removal heat exchangers, as of 1994 component cooling water (CCW) flow has been limited to approximately 1800 gpm through the shell side of each exchanger. See Section 9.2.2.4.1.6.

The pumps and heat exchangers are each half-capacity for the normal heat removal function; however, they are full capacity for their alternative function, which is low-head safety injection during loss-of-coolant accident conditions.

The residual heat removal pumps are driven by drip-proof type motors with either Class B PMR (protective moisture resistant) insulation or Class H insulation to be capable of operation in high humidity conditions. They are also equipped with Seismic Category I splash barriers to protect the motors in the event of a pipe line break in the area, which could possibly spray and wet the motors. Two access doors have been installed on each motor splash barrier near the radial and thrust bearing vibration transducer buttons to improve access for motor vibration measurements to be taken with hand-held transducers.

The residual heat removal pumps are powered from separate safeguards buses. Emergency power for these buses is available from either of two separate emergency diesel generators (Section 8.3). Two reactor coolant drain pumps, also powered from separate safeguards buses, can be used to back up the residual heat removal pumps for core cooling. The loss of

residual heat removal pumps and loss of reactor coolant drain pumps are addressed by emergency procedures.

Double, remotely operated valving is provided to isolate the residual heat removal loop from the reactor coolant system (Section 5.4.5.3). During reactor operation all equipment of the low-head injection and residual heat removal loop is idle, and the associated isolation valves are closed. During an accident condition fission products are recirculated through the exterior piping system. To obtain the total radiation dose to the public due to leakage for this system, the potential leaks have been evaluated and discussed in Sections 6.2 and 15.6.4.

5.4.5.2.1 Codes and Classifications

All piping and components were designed to the applicable codes and standards listed in Table 3.2-1. Austenitic stainless steel piping is used in the residual heat removal loop, which contains reactor coolant, and in the spent fuel pool (SFP) cooling system, which contains water without corrosion inhibitor.

Pressure retaining components (or compartments of components) through which reactor coolant circulates at pressures and temperatures significantly less than the reactor operating conditions at rated power comply with the following codes:

A. System pressure vessels - ASME Boiler and Pressure Vessel Code, Section III (1965), Class C, including paragraph N-2113.
B. System valves, fittings, and piping - USAS B31.1 (1965), including nuclear code cases.

A comparison of the requirements of the original design codes and standards to the current requirements is presented in Section 3.2.2.

5.4.5.2.2 Components

Residual heat removal system component design data is given in Table 5.4-6.

5.4.5.2.2.1 Heat Exchangers

The two residual heat removal heat exchangers are of the shell and U-tube type with the tubes welded to the tubesheet. Reactor coolant circulates through the tubes, while component cooling water (CCW) circulates through the shell side. The tubes and other surfaces in contact with reactor coolant are austenitic stainless steel and the shell is carbon steel.

5.4.5.2.2.2 Pumps

The two residual heat removal pumps are horizontal, centrifugal units with special seals to prevent reactor coolant leakage to the atmosphere. All pump parts in contact with reactor coolant are austenitic stainless steel or equivalent corrosion resistant material.

5.4.5.2.2.3 Valves

The valves used in the residual heat removal loop are constructed of austenitic stainless steel or equivalent corrosion resistant material.

Manual stop valves are provided to isolate equipment for maintenance. Throttle valves are provided for remote and manual control of residual heat exchanger tube side flow, and for automatic control of bypass flow. Check valves prevent reverse flow through the residual heat removal pumps.

Overpressure in the residual heat removal loop is relieved through a check valve to the low pressure letdown stream in the chemical and volume control system.

Manually operated valves have backseats to facilitate repacking and to limit the stem leakage when the valves are fully opened to the back-seated position.

5.4.5.2.2.4 Piping

All residual heat removal loop piping is austenitic stainless steel. The piping is welded except for flanged connections at the pumps, flow orifices, and flow control valves 624, 625, and 626.

5.4.5.3 Performance Evaluation

Basic functional requirements for the residual heat removal system are contained in NRC Branch Technical Position RSB 5-1, Design Requirements of the Residual Heat Removal System. Although the Position was issued after the design of Ginna Station, the following paragraphs provide a comparison of the Ginna design to these guidelines.

<u>5.4.5.3.1 Isolation Requirement</u>

5.4.5.3.1.1 Isolation Valve Description

The residual heat removal suction and discharge valves connecting this system to the primary coolant system are shown in Drawing 33013-1247. The reactor coolant system suction supply to the residual heat removal pumps is from the hot leg of loop A through motor-operated valves MOV-700 and MOV-701 in series. The residual heat removal pump discharge return to the loop B cold leg of the reactor coolant system is through two in-series motor-operated valves, MOV-720 and MOV-721. There are no check valves in series with MOV-720 and MOV-721.

Permissive interlocks required to open the four residual heat removal system isolation valves are listed below.

1. MOV-700
    a. Reactor coolant system pressure must be less than 410 psig.
    b. Residual heat removal suction valves MOV-850A and MOV-850B from the containment sump must be closed.
2. MOV-701
    a. Residual heat removal suction valves MOV-850A and MOV-850B from the containment sump must be closed.
    b. The valve is operated by a key switch.

3. MOV-720

   No interlocks exist but the valve is operated by a key switch.

4. MOV-721

   Reactor coolant system pressure must be less than 410 psig.

No interlocks are associated with valve closure. There are no automatic functions that close the valves and no alarms generated by the valves (*Reference 22*). The valves fail "as is" upon loss of power supply and have remote position indication in the control room.

The residual heat removal system discharge line is not used for an Emergency Core Cooling System (ECCS) function that would require MOV-720 or MOV-721 to open; however, a branch of the residual heat removal discharge line provides low-pressure safety injection to the reactor vessel via parallel lines with one normally closed motor-operated valve (MOV-852A or B) and one check valve (CV-853A or B) in each line. The check valves are periodically tested. The motor-operated valve position indication is provided in the control room and these valves receive an open signal coincident with the safety injection signal.

5.4.5.3.1.2 Deviations From Branch Technical Position RSB 5-1

Based on the above description, the residual heat removal system deviates from the following provisions of Branch Technical Position RSB 5-1:

a. The outboard residual heat removal discharge and suction isolation valves (MOVs 701 and 720) do not have independent diverse interlocks to prevent opening the valves until reactor coolant system pressure is below 410 psig. The outboard valves are manually controlled with key-locked switches. By procedure, MOV-701 and MOV-720 are not opened until reactor coolant system pressure is less than 410 psig.
b. The power-operated valves (MOVs 852A and B) in the low-pressure safety injection lines open on a safety injection signal before reactor coolant system pressure drops below residual heat removal design pressure.
c. The residual heat removal isolation valves have no interlock feature to close them when reactor coolant system pressure increases above the design residual heat removal pressure.

RG&E has concluded that the deviation regarding the independent, diverse interlocks to prevent opening of the outboard residual heat removal isolation valves (MOVs 701 and 720) until pressure is below 410 psig is acceptable. The outboard residual heat removal isolation valves will open against a differential pressure of greater than 500 psid. However, the inboard isolation valves (MOVs 700 and 721) are provided with a pressure interlock. By administrative procedure, the outboard residual heat removal valves (MOVs 701 and 720) are key-locked closed, with power removed. In addition, a relief valve (RV-203) with a capacity of 70,000 lb/hr, set at 600 psig, is available. Power would have to be restored, the key-locked switch enabled, and MOV 701 or 720 opened in violation of procedures and, in addition, interlocked valve MOV 700 or 721 would have to fail to allow significant leakage for a potential residual heat removal system overpressurization to occur. MOVs 700 and 721 are in the Ginna Pump and Valve Inservice Test Program and are leak-tested on a refueling basis.

Therefore it is concluded that the probability of an intersystem loss-of-coolant accident is acceptably low.

The deviation regarding the low-pressure safety injection isolation valves (MOVs 852A and B) is considered acceptable (*Reference 23*), since the check valve testing provides sufficient assurance that these valves (CVs 853A and B) will perform their isolation function until reactor coolant system pressure decreases below residual heat removal system pressure.

The deviation regarding lack of automatic closure for the residual heat removal isolation valves on increasing pressure is acceptable (*Reference 23*) based on the administrative controls which are provided for the operation of these valves, coupled with the residual heat removal system high pressure alarm at 550 psig and the reactor coolant system interlock pressure alarm at 410 psig. These alarms provide adequate assurance that the operator action required by procedure will be taken to shut the isolation valves when reactor coolant system pressure is increasing towards the residual heat removal design pressure.

5.4.5.3.2 Residual Heat Removal Overpressure Protection

5.4.5.3.2.1 Design Basis

The residual heat removal relief valve has a nominal setpoint of 600 psig and a capacity of 70,000 lb/hr. The residual heat removal system is provided with a 550 psig high-pressure alarm and a reactor coolant system interlock pressure alarm at 410 psig. The residual heat removal system is connected to the loop A hot leg on the suction side and the loop B cold leg on the discharge side. The design pressure and temperature of the residual heat removal system are 600 psig and 400°F. The design basis with regard to overpressure protection for the Ginna Station residual heat removal system is to prevent opening of the residual heat removal isolation valves when reactor coolant system pressure exceeds 450 psig and to provide relief capacity sufficient to accommodate thermal expansion of water in the residual heat removal system and/or leakage past the system isolation valves.

5.4.5.3.2.2 Analysis

An analysis of incidents which might lead to overpressurizing the residual heat removal system was performed (*Reference 24*). Three events were considered in the analysis:

a. With reactor coolant system in solid condition and residual heat removal and charging pumps operating, the letdown line from the reactor coolant system is isolated.
b. During cooldown using two residual heat removal trains, one residual heat removal train suffers a failure at a time when the core heat generation rate exceeds the heat removal capability of one train.
c. Pressurizer heaters are energized with residual heat removal in operation and reactor coolant system solid.

The results of these analyses showed that the residual heat removal system is provided adequate relief capacity when appropriate procedural steps are in place.

There is no safety relief valve at the suction side of the residual heat removal system to protect the residual heat removal system from potential overpressurization; thus the Low-Temperature Overpressure Protection (LTOP) system (Section 5.2.2) also protects the residual heat removal system from overpressurization when the residual heat removal system is connected to the reactor coolant system. Westinghouse performed an evaluation of the design basis transients for mass input and heat input (*Reference 24*), which was subsequently updated in support of the steam generator replacement project (*Reference 54*). The design basis transient for the mass input case is the charging-letdown mismatch with three positive displacement charging pumps in operation. The design-basis transient for the heat input case is the start of a reactor coolant pump with the steam generator secondary-side water and primary-side tube water 50°F higher than the rest of the reactor coolant system. It was determined that the allowable peak reactor coolant system pressure is more limiting for the residual heat removal system protection than that for the protection against the 10 CFR 50 Appendix G reactor pressure vessel limits.

The Technical Specifications (LCO 3.4.12) require that no safety injection pump be capable of injecting into the reactor coolant system whenever overpressure protection is provided by the pressurizer power operated relief valves (PORVs). The PORV setpoints contained in the Pressure and Temperature Limits Report (PTLR) provide overpressure protection for both the residual heat removal system and the reactor vessel 10 CFR 50 Appendix G limits for both the mass and heat input events. Also, the Technical Specifications allow that no more than one safety injection pump be capable of injecting when the overpressure protection is provided by a reactor coolant system vent equal to or greater than 1.1 in.^{2} Mass addition from the inadvertent operation of a safety injection pump will not result in residual heat removal system pressure exceeding allowable limits when overpressure protection is being provided by a reactor coolant system vent equal to or greater than 1.1 in.^{2}

The Technical Specifications requirements discussed above were originally approved by *Reference 15*, and later by *Reference 55*. The analysis was subsequently updated in support of the steam generator replacement project (*Reference 54*) and approved by the NRC in *Reference 57*.

The ability of the Ginna Low Temperature Over-Pressure Protection (LTOP) System to provide over-pressure protection for both the Reactor Coolant System (RCS) and the RHR system following a plant uprate to 1775 MWt was reviewed as part of the uprate project. As discussed in Section 2.8.4.3 of *Reference 69*, the bounding LTOP mass addition and heat addition analyses for RCS and RHR over-pressure protection are not affected by loss of decay heat cooling and therefore are not affected by the power uprate. Consequently, the existing over-pressure protection of the RHR System provided by LTOP is acceptable for uprate.

5.4.5.3.2.3 Effect of Stuck Open Relief Valve

Fluid discharged through the 2-in. residual heat removal relief valve (RV-203) is directed to the pressure relief tank inside the reactor containment. The pressure relief tank has a rupture disk which is designed to rupture at 100 psig and allow the contents of the tank to overflow to the containment sump, where it would be available for recirculation. Should flow from a stuck open residual heat removal relief valve cause the rupture disk to rupture, the

consequences to safety-related equipment would be less severe than the consequences of post-loss-of-coolant accident containment flooding which has been previously analyzed and found acceptable (*Reference 23*).

If RV-203 were to stick open in a post-loss-of-coolant-accident event, residual heat removal flow to the reactor coolant system for both low-head recirculation and low-head safety injection modes would be affected. This is because a flow path would exist from the residual heat removal system to RV-203 via valves HCV-133 and V-703 in either of these residual heat removal operating modes. HCV-133 fails shut following loss of instrument air on containment isolation following a loss-of-coolant accident, but a flow path would still exist to RV-203 via the 0.75-in. locked open manual valve 703. The effect of this flow diversion would not reduce the capability of the Emergency Core Cooling System (ECCS) below that needed to mitigate the consequences of a postulated loss-of-coolant accident. This is because the design flow rate through RV-203 (70,000 lb/hr, which is a conservative number in this case since HCV-133 is shut) is much less than the flow rate of a residual heat removal pump in the low-pressure safety injection mode (776,000 lb/hr). Each residual heat removal pump has the capacity to provide 100% of the required low-pressure safety injection flow. Therefore, the leakage through RV-203 would not be as severe an event as the loss of a residual heat removal pump which has been postulated as a single failure in the Emergency Core Cooling System (ECCS) analysis.

5.4.5.3.3 Residual Heat Removal Pump Protection

The features designed into the Ginna Station residual heat removal system to prevent damage to the system centrifugal pumps are provisions for pump cooling, a pump mini-flow recirculation flow path, and system design to prevent loss of net positive suction head.

The component cooling water (CCW) system provides cooling for the residual heat removal pumps to prevent damage from overheating. The residual heat removal pumps are provided with a recirculation line to recycle a portion of the pump discharge fluid to the pump suction. This prevents overheating during pump operation when the residual heat removal system is not delivering flow to the reactor coolant system. Net positive suction head calculations were performed for the residual heat removal pumps, and the residual heat removal system operation was evaluated for normal plant shutdown cooling, low-pressure safety injection, and post-loss-of-coolant accident recirculation. Although recirculation operation developed the most limiting net positive suction head requirements, the calculations indicated that an acceptable net positive suction head margin is available. See Section 6.3.3.9.

NRC Bulletin 88-04 expressed concern about the possibility of residual heat removal pump damage during parallel pump operation feeding a common discharge header under low flow conditions. Slight differences in their performance characteristics could result in the stronger pump forcing the weaker pump's discharge check valve closed, thereby creating a zero-flow or deadhead condition and damaging the pumps by overheating.

The Ginna residual heat removal pumps are each provided with a recirculation path to prevent pump damage from overheating. Each pump is provided with a 3-in. recirculation line with manual isolation valves on either end. The lines tap off from the pump discharge line between the heat exchanger and check valve and return to the residual heat removal pump

suction line just downstream of the outlet check valve for the refueling water storage tank (RWST). The check valves (697A, 697B) isolate the pump recirculation paths from each other. Each 3-in. recirculation line contains a 200-gpm orifice plate. Each residual heat removal pump thus has a minimum flow recirculation line that is independent of the opposite train and that provides sufficient recirculation flow to prevent damage when the pump discharge path is isolated. Each recirculation line is equipped with relief valves, located downstream of each 200 gpm orifice plate. The relief valves function during pump recirculation to ensure that pump suction pressure does not prevent the pump suction isolation valves (MOV 850A and MOV 850B) from opening, due to thermal expansion of the recirculating fluid. In MODES 4, 5, and 6, when the system takes suction from the hot leg, the relief valves are manually isolated. Pressure, temperature, and flow instrumentation is provided for each recirculation train. Therefore, it has been determined that the safety concerns raised in NRC Bulletin 88-04 have been resolved (*Reference 25*).

The residual heat removal pumps are provided continuously with component cooling water (CCW) flow for the pump thrust and radial bearing housings using a water jacket that surrounds the oil bath. Component cooling water (CCW) is also provided continuously through a water jacket within the residual heat removal pump head that encloses the mechanical seal. In addition, the mechanical seal includes a pumping ring that pumps process fluid from the seal area through an external heat exchanger (cooled by component cooling water), and back to the seal area. During the injection phase post-accident, the water source for the pump is the refueling water storage tank (RWST). Component cooling water (CCW) is assumed not to be available during the injection phase. Since the temperature of the water source (RWST) is less than 104°F during this period, the residual heat removal pump remains fully operable without component cooling water (CCW). During the recirculation phase, component cooling water (CCW) is made available to the pump. Because the temperature of the water source (from containment sump B) is expected to be much higher during the recirculation phase, component cooling water (CCW) is needed for cooling the mechanical seal. Cooling for the bearing housing water jacket is expected to be available, since the same cooling lines provide this CCW flow. This cooling flow would improve reliability, but is not required. Component cooling water (CCW) is required to be operable by Technical Specifications while the plant is operating in MODES 1, 2, 3, and 4, and operable (as a “necessary support system”) during MODES 5 and 6 when the residual heat removal pump is operating. During the transfer to the sump recirculation phase post-accident, cooling water would be immediately delivered to the residual heat removal pumps upon start of the component cooling water (CCW) pumps.

During normal plant cooldown or heatup, when the residual heat removal system is in operation and the pumped fluid is taken directly from the reactor coolant system, component cooling water (CCW) is necessary for the residual heat removal pump mechanical seal and bearing housing water jackets. However, during normal plant shutdown cooling operation, once the water temperature is stable and less than 120°F, component cooling water (CCW) is no longer considered necessary to maintain residual heat removal pump operability, but is desired from the standpoint of reliability.

5.4.5.3.4 Single-Failure Considerations

The single residual heat removal cooling suction line from the reactor coolant system and single discharge line to the reactor coolant system render the residual heat removal susceptible

to single failure of the in-line suction valves (700, 701) in the closed position and passive failures of either suction or discharge lines. (Valves 700 and 701, which are inside containment, can be manually operated to overcome a motor operator or power supply failure.) Although these failures would render the residual heat removal mode of decay heat removal inoperable, the alternate means of decay heat removal using the steam generators is still available as a backup. For the case of a failure of valves 700 or 701 or a pipe break downstream of these valves, an alternative flow path for core cooling is available via the residual heat removal cooling discharge line and the high-pressure safety injection pumps. Other means of core decay heat removal have a low heat removal capability but could be used to supplement steam generator heat removal until the decay heat rate was low enough. These methods are heat removal via the chemical and volume control system nonregenerative and excess letdown heat exchangers (requires component cooling water (CCW)) and cooldown flow from the pressurizer to the containment via the pressurizer safety valves with coolant injections from the safety injection or chemical and volume control systems. If a pipe break upstream of valves 700 and 701 should occur (i.e. a loss-of-coolant accident), the core could be adequately cooled by means of the residual heat removal sump recirculation mode.

The residual heat removal system contains a bypass line which is normally isolated during operation at power. During cooldown, the bypass line functions to control the total flow through the residual heat removal loops. A redundant bypass line is unnecessary in the system design, since the line can be manually isolated and the decay heat removal rate manually controlled in the event of a failure.

5.4.5.3.5 Leakage Provisions

The two residual heat removal pumps are located below the basement floor of the auxiliary building in a room provided with two environmentally qualified 50-gpm sump pumps, which discharge to the waste holdup tank. Environmentally qualified level switches control operation of the auxiliary building sump pumps and provide high level alarms on the plant process computer system. A single sump pump is capable of handling a leak rate from a residual heat removal pump seal failure, conservatively assumed to be 50 gpm. It is assumed that this passive failure could be isolated within 30 minutes. Consequently the waste holdup tank is required to operate at a level that will provide a holdup capability of 1500 gal for this postulated event during postaccident recirculation. Each auxiliary building sump pump starts automatically upon receiving a high-water level signal from one of two level instruments in the room.

From the standpoint of system reliability and availability in the unlikely event of failure of both auxiliary building sump pumps and assuming a conservative leak rate of 50-gpm, sufficient time is available (approximately 2 hr) to isolate the leaking residual heat removal pump before the water level in the pump room would flood the residual heat removal pump motors. The residual heat removal pumps are on separate pipe lines in the room in which they are located. Each pipe line contains a motor-operated valve, which could be closed remotely to isolate the leakage should the seal failure occur during postaccident recirculation. The residual heat removal pumps are driven by drip-proof type motors capable of operation in high humidity conditions and are provided with splash barriers. (See Section 5.4.5.2.)

5.4.5.3.6 Boron Concentration

One or more reactor coolant pumps or the residual heat removal system is in operation when a reduction is made in the boron concentration of the reactor coolant. At least one reactor coolant pump must be in operation for a planned transition from one reactor operating mode to another involving an increase in the boron concentration of the reactor coolant, except for emergency boration. When the boron concentration of the reactor coolant system is to be changed, the process must be uniform to prevent sudden reactivity changes in the reactor. Mixing of the reactor coolant is sufficient to maintain a uniform boron concentration if at least one reactor coolant pump or one residual heat removal pump is running (except as noted above) while the change is taking place. One residual heat removal pump will circulate the reactor coolant system volume in approximately 0.5 hour.

5.4.5.4 Residual Heat Removal at Reduced Coolant Inventory

5.4.5.4.1 Generic Letter 88-17 Requirements

Generic Letter 88-17 identified actions to be taken to preclude loss of decay heat removal during nonpower operations. These actions included operator training and the development of procedures and hardware modifications as necessary to prevent the loss of decay heat removal during reduced reactor coolant inventory operations, to mitigate accidents before they progress to core damage, and to control radioactive material if a core damage accident should occur. Procedures and administrative controls were required to ensure containment closure prior to the time that a core uncovery could result from a loss of decay heat removal coupled with an inability to supply alternative cooling or addition of water to the reactor coolant system inventory. Procedures were required that cover reduced inventory operations and ensure that all hot legs are not blocked by nozzle dams unless a vent path is provided that is large enough to prevent pressurization and loss of water from the reactor vessel. Instrumentation was required to provide continuous core exit temperature and reactor water level indication. Sufficient equipment was required to be maintained in an operable or available status so as to mitigate loss of the residual heat removal cooling or loss of reactor coolant system inventory should they occur.

Westinghouse provided thermal hydraulic evaluations of the loss of the residual heat removal system in the reduced inventory condition in *Reference 26*. *Reference 26* analyzed five configurations the plant could be in while the reactor coolant system (RCS) is in the reduced inventory mode. Ginna Station has committed in *Reference 58* not to enter two of those configurations: configuration #4 (cold side opening exists and nozzle dam not installed) and configuration #5 (cold side opening exists and nozzle dam installed).

Ginna Station had committed to maintain configuration #3 (hot side vent path exists when the RCS is being drained) for entry into the reduced inventory mode where the RCS will be opened for maintenance activities.

The RCS can be filled by two methods. The first is a conventional fill and vent with several starts of the reactor coolant pumps (RCPs) to dynamically vent the steam generators. This method uses the hot side vent path as the reduced inventory configuration. Operating

procedures have alternate steps to perform a conventional RCS fill and vent to bring the RCS to a solid plant condition.

Ginna specific analysis were performed in *Reference 67* and *68*. The results of these analyses were used by Ginna to form the basis for the required operator actions, which are implemented in procedures and administrative controls, and for the equipment required to be available for providing core cooling in the event residual heat removal cooling is lost.

The second fill method, RCS vacuum vent and fill, utilizes configuration #2 (intact RCS with water in the secondary side in the narrow range of a steam generator) for the final fill of the reactor coolant system. RCS vacuum vent equipment and temporary hoses are setup prior to the final fill of the RCS. When all nozzle dams have been removed, all primary manways have been installed on both steam generators, and at least one steam generator secondary has been filled to the narrow indication range, the plant is made ready to transition to configuration #2 (intact RCS).

The configuration is established when one or more of the power operated relief valves (PORVs) are opened and the pressurizer hot side vent is closed. At this time the vacuum vent and fill of the RCS can begin. Operating procedures maintain all the reduced inventory controls during the vacuum venting process while the RCS is being filled to a solid condition.

5.4.5.4.2 Containment Closure

Generic Letter 88-17 allows Westinghouse plants to take up to 2 hr to close containment when operating in the reduced inventory condition with openings totaling greater than 1 in^2 in the cold legs if a vent path exists that is sufficiently large that core uncovery cannot occur due to pressurization resulting from boiling in the core. Ginna procedures provide for control of containment penetrations and the capability to establish containment closure within 2 hr while in the reduced inventory condition during the period following reactor shutdown when the decay heat rate is high enough to cause core uncovery. However, since RCS pressure is not large enough to prevent gravity fill from the RWST, the core will not uncover and the Generic Letter 88-17 2 hour containment closure criteria is applicable. As an improvement to achieve containment closure within 2 hr, containment penetration number 2 was modified to provide access into the containment for the steam generator inspection and maintenance cabling, which had been previously routed through the equipment hatch during the annual inspection and outage (see Section 6.2.4.4.6). Thus, the hatch can be closed and containment isolated within the 2-hr time limit. The 2-hr time limit is not applicable at the end of a planned MODE 6 (Refueling) outage when operating in the reduced inventory condition because the time to reach saturation and core uncovery are extended. *Reference 27* provides plant-specific curves covering the reactor coolant system response to a loss of residual heat removal cooling with the reactor coolant system partially filled for all anticipated plant configurations. Ginna procedures provide for establishment of a large hot-side reactor coolant system vent path by removing the pressurizer manway before RCS inventory is reduced for mid-loop operation.

The use of configuration #2 (intact RCS) (see Section 5.4.5.4.1) during RCS vacuum vent and fill does not change the containment closure allowance time of 2 hours. Generic Letter 88-17

specifies that a closure time of 2.5 hours is acceptable provided there are no openings in the cold legs, reactor coolant pumps, and crossover legs (RCS intact). This configuration is entered following refueling when the time after shutdown is extended and decay heat is reduced.

5.4.5.4.3 Instrumentation for Reduced Inventory Operation

Ginna has instrumentation that is designed to aid operators in trending parameters important to maintaining residual heat removal operation and to detect abnormalities prior to a condition that could lead to a loss of residual heat removal cooling. The concern was that when using the residual heat removal system for shutdown cooling with a reduced reactor coolant system inventory, residual heat removal pump net positive suction head (NPSH) could be lost. The Ginna residual heat removal system has been provided with instrumentation to continuously monitor residual heat removal system performance whenever the system is being used for cooling the reactor coolant system and the coolant inventory is reduced. The instrumentation measures pump suction pressure, pump motor current, pump suction temperature, and pump discharge flow. The pump suction pressure, temperature, and flow signals are provided to the plant process computer system, which calculates pump NPSH from these inputs. The residual heat removal pump motor current and suction pressure also permit trending of current and pressure fluctuations associated with vortexing at the junction of the residual heat removal suction pipe and the reactor coolant loop. The plant process computer system can display and trend pump suction pressure and temperature, discharge flow, motor current, and margin to loss of NPSH for each residual heat removal pump. The plant process computer system provides an audible alarm on reaching the set low limit of margin for loss of NPSH. The plant process computer system also has a rate-of-change alarm on pump motor current. Loop level instrumentation is provided that accurately measures reactor coolant system loop level during reduced inventory conditions. The range is 0 to 100 in. Zero in. corresponds to a level 4 in. above the bottom of the hot leg and 100 in. is approximately 16 in. above the reactor vessel flange. The level sensing line for reactor coolant loop A is tied into the reactor coolant loop A hot leg via the residual heat removal suction lines. The sensing line for the reactor coolant loop B hot leg is tapped directly off the hot leg. The loop level instrumentation directly senses the head of water existing in the reactor coolant system and converts it to proportional electrical signals for transmission to the display and processing systems. The loop level instrumentation is designed for use when the plant is shut down and the reactor coolant system depressurized.

Local sightglass indication of loop level for the B loop is available in the containment basement. The sightglass (polycarbonate tube) with graduated level indication markings ranging from 0 to 144 in. of water is installed and used only during MODE 6 (Refueling) outages and is removed and stored prior to commencing power operations. The 0 in. marking corresponds to a level 4 in. above the bottom of the hot leg. Ten in. equals the mid-loop condition (centerline of the reactor coolant system hot leg). The sightglass is tied into the B loop level instrumentation tap. Permanently installed stainless steel tubing, valves (2), and supports accommodate the removable sightglass.

5.4.5.4.4 Available Equipment to Mitigate Loss of Residual Heat Removal Cooling

Generic Letter 88-17 recommends that at least two available or operable means of adding inventory to the reactor coolant system be provided in addition to the residual heat removal system during reduced inventory operations. These means should include at least one high-pressure injection pump. Ginna will have three methods available during reduced inventory operations. The preferred method is by gravity feed from the refueling water storage tank (RWST) directly to the loop A hot leg through valves MOV-856, MOV-701, and MOV-700 (see Drawing 33013-1247). Procedures provide for a large hot-side vent and allowable time constraints prior to entering a reduced inventory condition where the reactor coolant system will be opened for maintenance activities. The gravity feed method will be effective as long as a sufficient vent path exists and time constraints are adhered to, as defined in *Reference 27*. Gravity feed will raise the water level well above the top of the hot leg and allow restart of the residual heat removal pump. Charging pumps will be available as the second method of inventory addition. After uprate two charging pumps are required for the time period from 48 to 70 hours after shutdown to provide sufficient reactor coolant system (RCS) water addition to match the steam boil off rate. After a shutdown time of 70 hours only one charging pump is required to match the steam boil off rate. The flow path will be from the refueling water storage tank (RWST) to the loop B cold leg (normal charging path). For situations where a loop B cold-side opening exists, charging will be shifted to the loop A alternative charging pump line prior to opening the loop B cold side. (See Drawing 33013-1265, Sheets 1 and 2.) The adequacy of the charging pump method to the intact cold leg has been demonstrated in *Reference 26* by equating charging pump flow to core boil off rate.

The third method of recovery will be an available safety injection pump taking suction from the refueling water storage tank (RWST) and delivering to the loop A or B cold legs.

The fourth method of recovery will be an available safety injection pump taking suction from the refueling water storage tank (RWST) and delivering to the loop A hot leg if safety injection pump B is used or to the loop B hot leg if safety injection pump A is used. This method is also used if at any time core boiling is imminent or occurring as determined by core exit thermocouple indication or steam escaping from any reactor coolant system vents. (See Drawing 33013-1262, Sheets 1 and 2).

Ginna procedures require that the preferred flow paths and equipment be available prior to draindown with power to the appropriate components.

5.4.5.4.5 Reduced Inventory Procedures

Ginna procedures provide for the following during reduced reactor coolant inventory operations:

- Require a large vent path (i.e., pressurizer manway) sufficient to limit pressurization and subsequent loss of inventory, which could subsequently lead to core uncovery if unmitigated, whenever the reactor coolant system is to be opened for maintenance activities.
- Two core exit thermocouples powered from separate trains remain connected during reduced inventory operations.

- Control the removal and installation of steam generator manways and nozzle dams so that the hot leg manways and nozzle dams are removed first and installed last in the sequencing of steam generator maintenance.
- Provide control of containment penetrations and the capability to control containment closure.
- Provide capability to establish containment closure condition within the 2-hr limit.
- Require residual heat removal flow to be reduced and maintained at 800 gpm or less when operating at a level between 6 in. above loop centerline to loop centerline.
- Reduce residual heat removal flow to approximately 500 gpm or less for operation below loop centerline (necessary to perform resistance temperature detector maintenance).
- Reduced inventory condition will not be entered until reactor coolant system cold-leg water temperature has been reduced to less than 140°F and until at least 48 hours after shutdown.
- Require preferred flow paths and equipment be available with power to the appropriate components prior to draindown for means of adding inventory to the reactor coolant system in the event of loss of residual heat removal cooling.

Administrative controls implemented based on reduced inventory considerations:

- Prohibiting cold-side openings with the reactor coolant system unvented.
- Stationing an individual inside containment when water level is below the top of the hot leg to vent the residual heat removal system if necessary.
- Use of a volumetric measurement of reactor coolant system inventory during draindown to ensure that the appropriate volume of water has been drained prior to steam generator manway removal.
- Minimizing the time while operating at reduced inventory consistent with accomplishing required tasks during this condition and in consideration of overall plant safety.
- The hot leg vent is not required after the nozzle dams are removed and manways are installed if the intact reactor coolant system (RCS) configuration, with at least one steam generator filled with water to the narrow range taps (e.g. tubes covered with water) has been established. The power operated relief valves (PORVs) can then be opened and the pressurizer manway installed. This transition is only performed during the RCS vacuum vent and fill process.

<u>5.4.5.4.6 Analyses</u>

Plant-specific analyses were conducted to provide the evaluations for expected nuclear steam supply system behavior for all phases of non-power operations after the plant uprate to 1775 MWt. Results of the analyses include the following:

- Determined the plant-specific curves for time to reach saturation as a function of time after shut down for reactor coolant system initial temperatures of 100°F and 140°F.
- Calculated the boil-off rate following loss of residual heat removal for the above. These results were used to determine required makeup flow to prevent core uncovery.

- Calculated the reactor coolant system pressurization following loss of residual heat removal with nozzle dams installed and as a function of time after shut down. This analysis was used to justify the use of the pressurizer manway vent path and demonstrate gravity fill would be available.
- Calculated the time to core uncovery as a function of time after shutdown assuming the pressurizer manway opening for the following scenarios:

    1. All nozzle dams installed.
    2. No nozzle dams installed.
    3. Cold-leg opening nozzle dams not installed.
    4. Effects of surge line flooding on reactor coolant system pressurization.

5.4.5.5 Tests and Inspections

The residual heat removal pumps flow instrument channels are calibrated periodically.

Regulatory Guide 1.68, Initial Test Programs for Water-Cooled Reactor Power Plants, was not in existence when the Ginna Station preoperational and initial startup testing was accomplished. However, tests have been performed to confirm that cooldown under natural circulation can be achieved. The core flow rates achieved under natural circulation were more than adequate for decay heat removal. The calculated core flow at approximately 2% reactor power was 4.2% of nominal full power flow. At approximately 4% reactor power, calculated core flow was 5.2% of nominal. Flow rates of this magnitude provide adequate mixing of boron added to the reactor coolant system during cooldown.

Rochester Gas and Electric Corporation has implemented a valve test program in response to a generic NRC requirement (*Reference 28*) associated with the issue of the isolability of low-pressure systems from interfacing high-pressure systems. As implemented in the Technical Specifications, check valves 853A, 853B, 867A, 867B, 877A, 877B, 878F, 878G, 878H, and 878J, and motor operated valves 878A and 878C are tested to 0.5 gpm or less per nominal inch of valve size up to 5.0 gpm leakage.

5.4.6 MAIN STEAM AND FEEDWATER PIPING

The main steam piping has an inner diameter of 28 in. Steam flow is measured by monitoring dynamic head in nozzles inside the main steam piping. The nozzles, which have an inner diameter of 16 in., are located inside containment near the steam generators and serve to limit the maximum steam flow for any main steam line break further downstream. Note that in 1996, Replacement Steam Generators (RSGs) with integral main steam nozzle flow restrictors were installed. These restrictors limit maximum steam flow for all main steam line breaks. The main steam system is discussed in Chapter 10.

The main feedwater piping is ASTM A106 grade C seamless pipe with ASTM A234 grade WPB fittings (except as noted below), and was fabricated to the requirements of the ASA Code for Pressure Piping, B31.1-1955. Replacement Steam Generators (RSGs) were installed in 1996. The RSG feedwater nozzles are forged SA-508 Class 3. The RSG feedwater nozzles include a forged Inconel (SB166, UNS NO6690) safe-end transition

between the nozzle and feedwater piping. This safe-end also provides the connection to the RSG welded thermal sleeve for the RSG internal feedwater distribution piping and feed ring. The internal thermal sleeve, distribution piping and feed ring are SA-355, GR P22. The feed ring is equipped with Inconel J-nozzles (SB-167, UNS NO6690).

In 1979, several pressurized water reactors, Ginna Station included, experienced feedwater pipe cracking in the vicinity of the feedwater to steam generator nozzles. At Ginna Station, stress-assisted corrosion and corrosion fatigue cracking were found in the feedwater piping-to-nozzle elbow welds just upstream of the nozzles. In response to IE Bulletin 79-13 (*References 29 through 31*), the welds were repaired and documented by reports (*References 32* and *33*) submitted to the NRC. Also in 1979, the 18-in. elbows at the steam generator nozzles were replaced with elbows of ASTM A234 grade WP-11. To facilitate Steam Generator Replacement, these elbows were again replaced in 1996. The 1996 replacement elbows are SA234, GR WP11. Additionally, the internal feedwater distribution piping for the RGSs employs a gooseneck design between the feedwater nozzle and feed ring. The gooseneck limits the volume of horizontal piping. This minimizes fill time and, therefore, reduces the thermal stratification, temperature distributions, and thermal stresses which contribute to the stress-assisted corrosion and corrosion fatigue cracking experienced previously.

5.4.7 PRESSURIZER

5.4.7.1 System Description

The general arrangement of the pressurizer is shown in Figure 5.4-8 and the design characteristics are listed in Table 5.4-7.

The pressurizer maintains the required reactor coolant pressure during steady-state operation, limits the pressure changes caused by coolant thermal expansion and contraction during normal load transients, and prevents the pressure in the reactor coolant system from exceeding the design pressure. The pressurizer vessel contains replaceable direct immersion heaters, multiple safety and pressurizer power operated relief valves (PORVs) (Section 5.4.10), a spray nozzle, and interconnecting piping, valves, and instrumentation.

There are 78 heaters separated into a control/variable group and a backup group. The heaters are made of nichrome wire with a magnesium oxide insulator. The heater terminals are hermetically sealed and designed to withstand the design pressure and temperature of the pressurizer. The heaters are located in the lower section of the vessel and pressurize the reactor coolant system by keeping the water and steam in the pressurizer at saturation temperature. The heaters are capable of raising the temperature of the pressurizer and contents at approximately 55°F/hr during startup of the reactor. Of the 78 heaters installed, 74 heaters are currently available for use. The 74 heaters have a total capacity of approximately 760 kW. (The original 78 heaters had a total capacity of approximately 800 kW).

In the event of a loss of offsite power, pressurizer heaters can be manually loaded onto emergency power sources.

The pressurizer is designed to accommodate positive and negative surges caused by load transients. The surge line, which is attached to the bottom of the pressurizer, connects the

pressurizer to the hot leg of the B reactor coolant loop. During a positive surge, caused by a decrease in plant load, the spray system, which is fed from the cold leg of a coolant loop, condenses steam in the vessel to prevent the pressurizer pressure from reaching the setpoint of the pressurizer power operated relief valves (PORVs). Power-operated spray valves on the pressurizer limit the pressure during load transients. In addition, the spray valves can be operated manually by a controller in the main control room.

Two separate, automatically controlled spray valves with remote-manual overrides are used to initiate pressurizer spray. A manual throttle valve in parallel with each spray valve permits a small continuous flow through each spray line to reduce thermal stresses and thermal shock when the spray valves open. The throttle valve flow also helps maintain uniform temperature and water chemistry in the pressurizer. Two separate spray valves and spray line connections are provided so that the spray will operate when only one reactor coolant pump is operating.

A flow path from the chemical and volume control system is also provided to the pressurizer spray line. This flow path provides auxiliary spray to the vapor space of the pressurizer during cooldown when the reactor coolant pumps are out of service. Thermal sleeves on the pressurizer spray connection and spray piping are designed to withstand the thermal stresses resulting from the introduction of cold spray water.

During a negative pressure surge, caused by an increase in plant load, flashing of water to steam and generation of steam by automatic actuation of the heaters keep the pressure above the minimum allowable limit. Heaters are also energized on high water level during positive surges to heat the subcooled surge water entering the pressurizer from the reactor coolant loop.

The pressurizer is a vertical, cylindrical vessel with hemispherical top and bottom heads, constructed of carbon steel with internal surfaces clad with austenitic stainless steel. The heaters are sheathed in austenitic stainless steel. The pressurizer is insulated to minimize heat loss from the pressurizer vessel. The insulation consists of reflective panels that are removable to permit visual examination of the pressurizer as required by the Inservice Inspection (ISI) Program document.

The pressurizer vessel surge nozzle is protected from thermal shock by a thermal sleeve. A thermal sleeve also protects the pressurizer spray nozzle connection.

5.4.7.2 Seismic Evaluation

Within the scope of SEP Topic III-1 [Classification of Structures, Components, and Systems (Seismic and Quality)] the seismic resistance of the pressurizer was evaluated. Based on analyses of a heavier, 1800 ft^3, model (but with identical support skirts to the Ginna 800 ft^3 model) and utilizing a finite element model it was concluded that the Ginna pressurizer is adequately supported for the 0.2g safe shutdown earthquake.

5.4.8 PRESSURIZER RELIEF DISCHARGE SYSTEM

5.4.8.1 System Description

The pressurizer safety and pressurizer power operated relief valves (PORVs), described in Section 5.4.10, discharge to the pressurizer relief tank.

Principal design parameters of the pressurizer relief tank are given in Table 5.4-8. A diagram of the tank is shown in Figure 5.4-9.

Steam and water discharged from the pressurizer safety valves and pressurizer power operated relief valves (PORVs) pass to the pressurizer relief tank which is partially filled with water at or near ambient containment conditions. The cool water condenses the discharged steam and the condensate is drained to the waste disposal system. The tank normally contains water in a predominantly nitrogen atmosphere, although provisions have been made to periodically analyze the tank gas for accumulation of hydrogen and oxygen. Nitrogen pressure is normally maintained at 3 psig. The tank is equipped with a spray and drain which are operated to cool the tank following a discharge.

The tank size is based on the requirement to condense and cool a discharge equivalent to 110% of the full power pressurizer steam volume. Assuming an initial tank water temperature of 125°F, the tank is capable of absorbing an amount of heat such that the final water temperature is no greater than 200°F. If the temperature in the tank rises above 120°F during plant operation, the tank is cooled by spraying in cool reactor makeup water and draining out the warm mixture to the reactor coolant drain tank.

The spray rate is designed to cool the tank from 200°F to 120°F in approximately 1 hr following the design discharge of pressurizer steam. The volume of nitrogen gas in the tank is selected to limit the maximum pressure to 50 psig following a design discharge.

The tank is protected against a discharge exceeding the design value by a rupture disk which discharges into the reactor containment. The rupture disk on the relief tank has a relief capacity in excess of the combined capacity of the pressurizer safety valves. The tank design pressure (and the rupture disk setting) is twice the calculated pressure resulting from the maximum safety valve discharge described above, i.e., the tank design pressure is 100 psig. This margin is to prevent deformation of the disk. The tank and rupture disk holder are also designed for full vacuum to prevent tank collapse if the tank contents cool without nitrogen being added.

The impact of plant uprate to 1775 MWt on the design of the pressure relief tank was assessed by determining the amount of steam discharged to the tank from the limiting loss of electrical load transient at the uprate power level. As described in Section 2.5.2.2.2 of *Reference 69*, the amount of steam discharged to the tank from the limiting uprate external load transient is less than the amount of steam assumed to be discharged to the tank for the original tank design basis. Therefore, the original pressure relief tank discharge design basis is still satisfied at the plant uprate power level of 1775 MWt.

The impact of an elevated containment temperature of 5°F (from 120°F to 125°F) on the design of the pressurizer relief tank was assessed in Reference 71 for the limiting loss of electrical load transient at the uprate power level. This assessment demonstrated

that the original pressurizer relief tank discharge design basis is still satisfied.

Pressure relief tank pressure is indicated in the control room on the main control board on a narrow range (0-7.5 psig) and wide range (0-150 psig) meter. This allows the control room operator to monitor pressure relief tank pressure up to the rating of the rupture disk.

The discharge piping from the safety and pressurizer power operated relief valves (PORVs) to the relief tank is sufficiently large to prevent backpressure at the safety valves from exceeding 20% of the setpoint pressure at full flow.

The pressurizer relief tank, by means of its connection to the waste disposal system, provides a means for removing any noncondensible gases from the reactor coolant system which might collect in the pressurizer vessel. The tank is constructed of carbon steel with a corrosion resistant coating on the internal surface. A flanged nozzle is provided on the tank for the pressurizer discharge line connection. The pressurizer discharge line, the nozzle, and the sprayer inside the tank are austenitic stainless steel.

5.4.8.2 System Analysis

In response to NUREG 0737, Section II.D.1, and the NRC plant-specific submittal request for piping evaluation, Westinghouse performed an analysis of the Ginna pressurizer safety and relief valve discharge piping system (see Section 3.9.2.1.4). It was determined that the operability and structural integrity of the system were ensured for all applicable loadings and load combinations including all pertinent safety and relief valve discharge cases.

5.4.9 VALVES

5.4.9.1 Original Valve Design

All the valves originally installed in the nuclear steam supply system had stems with back seats to prevent ejection of valve stems. If it were assumed that the stem threads fail, the upset required for the back seat prevents penetration of the bonnet as shown by analysis, thereby preventing the stem from becoming a missile. The stems of air and motor-operated valves included similar interference.

Valves with nominal diameter larger than 2 in. were designed to prevent bonnet-body connection failure and subsequent bonnet ejection. The means of prevention included (a) using the design practice of ASME Section VIII, which limits the allowable stress of bolting material to less than 20% of its yield strength, (b) using the design practice of ASME Section VIII for flange design, and (c) controlling the load during the bonnet-body connection stud-tightening process.

The pressure containing parts, except the flange and studs, were designed per criteria established by the USAS B16.5. Flanges and studs were designed in accordance with ASME Section VIII. Materials of construction for these parts were procured per ASTM A182, F316, or A351, GR CF8M.

Stud and nut material was ASTM A193-B7 and A194-2H. The bonnet-body studs and nut material were later upgraded with 17-4PH ASTM A-564 TP 630 and ASTM A-194-8M TP316 material, respectively. The proper stud torquing procedures and the use of a torque wrench, with indication of the applied torque, limited the stress of the studs to the allowable

limits established in the ASME Code, i.e., 20,000 psi. This stress level was far below the material yield, i.e., about 105,000 psi. The complete valves were hydrotested per USAS B16.5 (1500-lb USAS valves were hydrotested to 5400 psi). The cast stainless steel bodies and bonnets were radiographed and dye penetrant tested to verify soundness.

Valves with nominal diameter of 2 in. or smaller were forged and had screwed bonnets with canopy seals. The canopy seal was the pressure boundary while the bonnet threads were designed to withstand the hydrostatic end force. The pressure containing parts were designed to the criteria established by the USAS B16.5 specification.

5.4.9.2 Valve Wall Thickness

An engineering review of nuclear valves was conducted during the 1974-1975 time period as required by *Reference 34*. The review was the first phase of a program to demonstrate acceptable wall thickness on certain valves important to nuclear safety.

The engineering review of valves identified 55 valves with greater than a 1-in. nominal pipe size within the Ginna Station reactor coolant pressure boundary. These valves were 1500-lb pressure class valves designed for reactor coolant system design pressure of 2485 psi and design temperature of 650°F. The valves were originally purchased to either ASA B16.5, MSS SP-66, or ASME Section III. The valves varied in size from 2-in. to 10-in. nominal pipe size.

Physical or ultrasonic inspections were conducted to verify adequate wall thickness on all valves described above. The measurement program was based on design and manufacturing requirements in ANSI B16.5 or MSS SP-66. The valves were either found to meet requirements or, in the case of one valve, repaired to meet requirements. Valve wall thickness measurements were made on all spare nuclear valves then in stock. Specifications were prepared requiring measurement and manufacturer's certification of adequate valve wall thickness for all valves to be subsequently purchased for use in Ginna Station Seismic Category I systems.

5.4.9.3 Motor-Operated Valve Program

<u>**Generic Letters 89-10 and 96-05**</u>

The Ginna Station motor-operated valve program was established in response to IE Bulletin 85-03 (*Reference 35*). The program was later expanded to address the recommendations of Generic Letter 89-10 (*Reference 36*) and Generic Letter 96-05 (*Reference 61*) to include all motor-operated valves in safety-related systems that are not blocked from inadvertent operation from either the control room, motor control center, or the valve itself. The following safety-related systems are included in the program:

- High-head safety injection - injection mode.
- Low-head safety injection - injection mode.
- High-head safety injection - recirculation mode.
- Low-head safety injection - recirculation mode.
- Auxiliary feedwater.

- Standby auxiliary feedwater.
- Containment spray.
- Component cooling water (CCW) - safety injection and residual heat removal pump cooling; sump recirculation cooling.
- Service water (SW) - nonessential load isolation.

The motor-operated valves in the above systems are tested at design pressure when practicable; otherwise, alternative methods are used to ensure motor-operated valve operability. The motor-operated valve program is described in the Ginna Station Motor-Operated Valve Qualification Program Plan. The motor-operated valve program is used to establish torque switch and limit switch settings for safety-related ac and dc motor-operated valves and to demonstrate valve operability during normal and abnormal design-basis events. The program also includes periodic and post maintenance and repair testing to verify continued valve operability. This program includes periodic verification of motor-operated valve capability and trending of motor-operated valve problems. The motor-operated valve program and Ginna Station procedures are designed to ensure that the switch settings of the motor-operated valves in the program are selected, set, and maintained correctly to accommodate the maximum differential pressures expected across the valves during both normal and abnormal design-basis events throughout the life of the plant. In response to Generic Letter 96-05 (*Reference 62*), the program was enhanced to include provisions for continually monitoring valve performance for degradation and periodic verification of program effectiveness. In *Reference 59*, RG&E provided closure notification to the NRC and in *Reference 60*, the NRC closed out its review of Generic Letter 89-10. In *Reference 63*, the NRC stated that RG&E has established an acceptable program to verify periodically the design-basis capability of all safety-related motor-operated valves at Ginna Station, and is adequately addressing the actions requested in Generic Letter 96-05.

As discussed in Section 2.2.4.2.2 of *Reference 69*, the impact of a plant uprate to 1775 MWt on the Ginna MOV Program was evaluated. The evaluation determined that although there were minor changes to flows, temperatures and differential pressures for some of the valves within the Ginna MOV Program, the changes did not affect the ability of the Ginna MOVs to comply with the requirements of Generic Letter 89-10 and Generic Letter 95-06.

Generic Letter 95-07

In response to Generic Letter 95-07 (*Reference 48*), Pressure Locking and Thermal Binding of Safety-Related Power-Operated Gate Valves, RG&E considered the safety-related motor-operated gate valves, including all valves within the GL 89-10 program, that could be potentially susceptible to this phenomena, and performed assessments, analyses or identified previous valve modifications to justify continued operability of the valves. The assessments of each valve were based upon the operational configurations and conditions imposed.

A number of valves received analysis that demonstrated that the developed valve thrust is capable of overcoming the imposed loads. These included: valves 860A, 860B, 860C, and 860D, (discharge isolation valves from containment spray pumps); and 871A and 871B (discharge valves from safety injection pump C to reactor coolant system loops A and B).

Valves 852A and 852B, residual heat removal supply valves to the reactor vessel deluge, were modified in 1999 with flexible wedges that have vent holes. Valves 515 and 516, the pressurizer power operated relief (PORV) block valves, were modified in 1989 with upstream discs that have vent holes, and valves 850A and 850B, the residual heat removal suction valves from containment sump B, were modified in 1970 to include bonnet vents to the residual heat removal pump suction side of the valves. Valves 857A, 857B, and 857C, the discharge valves from residual heat removal pumps to safety injection pumps, were modified in 1996 to install a bonnet pressure relieving hole in the designated valve disc relieving pressure to the residual heat removal side of the valves. The balance of the valves identified were justified based upon the operational configuration and conditions imposed. These included: valves 738A and 738B, (component cooling water supply valves to the residual heat removal heat exchanger); 3504A and 3505A, (main steam supply valves to the turbine driven auxiliary feedwater pump); 704A and 704B, (suction isolation valves to the residual heat removal pumps); 1815A and 1815B, (suction isolation valves for safety injection pump C); and 4615 and 4616 (service water isolation valves to auxiliary building loads). RG&E's response to the generic letter is contained in *Reference 49*.

As discussed in Section 2.2.4.2.2 of *Reference 69*, the impact of a plant uprate to 1775 MWt on the Ginna valve pressure locking and thermal bounding analyses was evaluated. The evaluation determined that the plant uprate had no impact on any of the Ginna pressure locking or thermal binding analyses.

5.4.10 SAFETY AND PRESSURIZER POWER OPERATED RELIEF VALVES (PORVS)

5.4.10.1 System Description

The reactor coolant system is protected against overpressure (Section 5.2.2) by control and protective circuits such as the two high-pressure code safety valves and the two pressurizer power operated relief valves (PORVs) connected to the top head of the pressurizer. The valves discharge into the pressurizer relief tank, which condenses and collects the valve effluent. The schematic arrangement of the relief devices is shown in Drawings 33013-1258 and 33013-1260.

The pressurizer power operated relief valves (PORVs) and spring-loaded code safety valves are provided to protect against pressure surges that are beyond the pressure limiting capacity of the pressurizer spray. The pressurizer discharge lines leading to each pressurizer power operated relief valve (PORV) contain a motor-operated block valve to be used if the pressurizer power operated relief valve (PORV) opens inadvertently or fails to close following an overpressurization transient. The block valves are remote manually controlled from the control room. Leakage limits for the block valves are included in the reactor coolant system (RCS) operational limits in the Technical Specifications. Design parameters of the safety, relief, and blocking valves are given in Table 5.4-9.

At least one pressurizer code safety valve is in service whenever the reactor is subcritical and the reactor coolant system is in MODE 4 (Hot Standby), except during hydrostatic tests. Both pressurizer code safety valves are in service during MODE 3 (Hot Shutdown) and prior to criticality.

Each of the two pressurizer code safety valves is designed to relieve 288,000 lb/hr of saturated steam at the valve setpoint. Below 350°F and 350 psig in the reactor coolant system, the residual heat removal system can remove residual heat and thereby control system temperature and pressure. >For the original licensed power of 1520 MWt if no residual heat were removed by any of the means available the amount of steam which could be generated at safety valve relief pressure would be less than half the valves capacity. Since the plant uprate to 1775MWt increased reactor power and the corresponding decay heat by approximately 17%, the amount of steam generated if no residual heat was removed would still be less than the flow capacity of one safety valve. Therefore, one valve provides adequate defense against overpressurization. In addition, the low temperature overpressure protection system (LTOP) is placed in service prior to the RCS system being cooled below the LTOP enable temperature or the residual heat removal system being placed in service. The LTOP system and its operators are described in detail in Section 5.2.2.

A resistance temperature detector located in the discharge pipe of each code safety valve provides indication of valve movement or significant seat leakage. Actuation of a safety valve will cause a rapid rise in discharge temperature, which is sensed by the resistance temperature detector and indicated/alarmed in the control room. Also linear voltage differential transducers on the pressurizer safety valves provide a direct indication of valve position.

The pressurizer power operated relief valves (PORVs) have direct stem position indication in the control room. An alarm is provided in conjunction with the indication.

5.4.10.2 Performance Testing and Evaluation

Under NUREG 0737, Item II.D.1, Performance Testing of BWR and PWR Relief and Safety Valves, all operating plant licensees and applicants were required to conduct testing to qualify the reactor coolant system relief and safety valves under expected operating conditions for design-basis transients and accidents. In addition to the qualification of valves, the functional ability and structural integrity of the as-built discharge piping and supports were also required to be demonstrated on a plant-specific basis.

In response to these requirements, a program for the performance testing of pressurized water reactor safety and pressurizer power operated relief valves (PORVs) was formulated by EPRI. The primary objective of the test program was to provide full-scale test data confirming the functional ability of the reactor coolant system pressurizer power operated relief valves (PORVs) and safety valves for expected operating and accident conditions. The second objective of the program was to obtain sufficient piping thermal hydraulic load data to permit confirmation of models which may be utilized for plant unique analysis of safety and relief valve discharge piping systems.

The valves, piping arrangements, and fluid inlet conditions used in the EPRI tests confirmed the ability of the Ginna Station safety valves, pressurizer power operated relief valves (PORVs), and block valves to open and close under expected conditions. Power-operated relief and block valves were found to fulfill their design functions with neither the valves nor the control circuitry being subjected to a harsh environment.

The operability and structural integrity of the Ginna Station configuration was also verified on a plant-specific basis by Westinghouse for all applicable loadings and load combinations, including pertinent safety valve and relief valve discharge cases. See Section 3.9.2.1.4 for a discussion of the analysis. The NRC Safety Evaluation Report (*Reference 37*) concluded that Ginna Station had provided an acceptable response to the requirements of NUREG 0737, Item II.D.1, provided that plant procedures are adopted for inspecting the relief and safety valves after each lift involving the loop seal or water discharge.

5.4.11 COMPONENT SUPPORTS

5.4.11.1 Design Criteria

5.4.11.1.1 General

The classification of all components, systems, and structures for the purposes of seismic design are given in Section 3.7.1. The definition of the three original seismic Classes is given in Section 3.7.1.1.

All components of the reactor coolant system and associated systems were designed to the standards of the applicable ASME Code or USAS Code. The loading combinations that were originally employed in the design of Seismic Category I components of these systems, i.e., vessels, piping, supports, vessel internals, and other applicable components, are given in Table 3.9-1. This table also indicates the stress limits that were used in the design of the listed equipment for the various loading combinations.

To be able to perform their function, i.e., allow core shutdown and cooling, the reactor vessel internals had to satisfy deformation limits that were more restrictive than the stress limits shown in Table 3.9-1. For this reason the reactor vessel internals were treated separately (see Section 3.9.5).

In general, modifications or additions to piping systems at Ginna Station since initial operation have been seismically qualified using dynamic analyses. Some small piping has been seismically qualified using equivalent analysis or spacing table techniques. Specific cases are discussed in Section 3.9.2.1.

As a result of the SEP preliminary seismic review of Ginna Station, IE Bulletin 79-14, and other NRC seismic requirements, Ginna initiated a seismic upgrade program after the completion of piping support modifications required by IE Bulletin 79-14. The loading combinations and associated stress limits used for the piping systems that are part of the seismic upgrade program are discussed in Section 3.9.2.1.8 and appear in Table 3.9-8.

5.4.11.1.2 Asymmetric Loss-of-Coolant Accident Loading

In January 1978, all licensees of pressurized-water reactor plants were required by the NRC to provide an assessment of the adequacy of the reactor vessel supports and other affected structures to withstand combinations of response to asymmetric loss-of-coolant accident loads and the safe shutdown earthquake. In response, *References 38 through 41* were submitted to the NRC for the Westinghouse Owners Group plants in the form of Topical Reports relating to the "leak-before-break" concept. The NRC evaluation (*Reference 42*) of the above

references concluded that an acceptable basis had been provided so that the asymmetric blowdown loads resulting from double-ended pipe breaks in main coolant loop piping need not be considered as a design basis for the Westinghouse Owners Group plants, provided that leakage detection systems exist to detect postulated flaws utilizing guidance from Regulatory Guide 1.45.

By *Reference 43* Ginna provided information to the NRC concerning the capability of the leakage detection systems installed at Ginna Station to detect a 1.0-gpm leak within 4 hours. By *Reference 44* the NRC reported that the NRC met the criteria specified in *Reference 42* and that the asymmetric blowdown loads resulting from double-ended pipe breaks in main coolant loop piping need not be considered as a design basis for Ginna Station.

In the SER provided by the NRC, *Reference 69* concluded, in Section 2.1.6, that the Ginna analyses were still valid after the plant uprate to 1775MWt.

5.4.11.1.3 Lamellar Tearing

During the mid-1970s the NRC raised a number of questions about the potential for lamellar tearing and low fracture toughness of materials used in steam generator supports and reactor coolant pump supports; Ginna addressed this issue in *References 45* and *46*. It was concluded that adequate fracture toughness exists for the supports at Ginna Station and that lamellar tearing was not an issue for the Ginna Station design and installation.

5.4.11.2 Support Structures

See also Section 3.9.3.2.

5.4.11.2.1 Reactor Vessel Supports

The vessel is supported on six individual pedestals. Each pedestal rests upon plates that are in turn supported upon the circular concrete primary shield wall.

The reactor vessel has six supports comprising four support pads located one on the bottom of each of the primary nozzles and two gusset support pads. One of the reactor inlet nozzles is centered approximately 2 degrees counterclockwise from the 90-degree axis and the other is centered approximately 2 degrees counterclockwise from the 270-degree axis.

Each support bears on a support shoe, which is fastened to the support structure. The support shoe is a structural member that transmits the support loads to the supporting structure. The support shoe is designed to restrain vertical, lateral, and rotational movement of the reactor vessel, but allows for thermal growth by permitting radial sliding at each support, on bearing plates.

The seismic resistance of the reactor vessel supports was evaluated as part of SEP Topic III-6. It was concluded, based on experience for nozzle-supported vessels, that the seismically induced stresses in the nozzles and adjacent shells are very small and that the governing element for reactor vessel support is the concrete shield wall. The shield wall was considered to be adequate to withstand the 0.2g safe shutdown earthquake according to the NRC review (*Reference 47*).

5.4.11.2.2 Steam Generator Supports

Each steam generator is supported on a structural system consisting of four vertical support columns and two (upper and lower) support systems. The vertical columns, which are pin connected to the steam generator support feet, serve as vertical restraint for operating weights, pipe rupture, and seismic considerations while permitting movement in the horizontal plane. The support systems, by using a combination of stops, guides, and snubbers, prevent rotation and excessive movement of the steam generator in any vertical plane. Thermal expansion is permitted in the support systems by a key arrangement. (See Section 3.9.3.2.2.)

5.4.11.2.3 Reactor Coolant Pump Supports

The reactor coolant pump is supported by a structural system consisting of three vertical columns and a system of stops. The vertical columns are bolted to the pump support feet and permit movement in the horizontal plane to accommodate reactor coolant pipe expansion. Horizontal restraint is accomplished by a combination of tie rods and stops which limit horizontal movement for pipe rupture and seismic effects.

5.4.11.2.4 Pressurizer Supports

The pressurizer is supported on a heavy concrete slab spanning between the concrete shield walls for the steam generator compartment. The pressurizer is a bottom skirt support vessel.

5.4.11.2.5 Reactor Coolant Piping Supports

The reactor coolant piping layout is designed on the basis of providing floating supports for the steam generator and reactor coolant pump in order to permit the thermal expansion from the fixed or anchored reactor vessel. A comprehensive thermal analysis was performed to ensure that stresses induced by linear thermal expansion were within code limits.

Two shock suppressors (snubbers) are provided on each steam generator to ensure piping structural integrity during and following a seismic event or other event initiating dynamic loads.

5.4.11.2.6 Inspection and Testing

The inspection and testing of all safety-related hydraulic and mechanical shock suppressors (snubbers) shall be implemented and performed in accordance with the "Snubber Inspection and Testing Program", to ensure the required operability of these snubbers during and following a seismic or other event, initiating dynamic loads. Station procedures include a listing of safety-related hydraulic snubbers that must be operable, limiting conditions of operations relative to these snubbers, and an inspection and testing program for snubbers. The inspection program includes all safety-related snubbers and snubbers installed on non-safety-related systems whose failure or failure of the system on which they are installed could have an adverse effect on a safety-related system (see Section 3.9.3.3.5).

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54. Letter from R. C. Mecredy, RG&E, to A. R. Johnson, NRC, Subject: Application for Amendment to Facility Operating License, Methodology for Low Temperature Overpressure Protection (LTOP) Limits, dated February 9, 1996.

55. Letter from A. R. Johnson, NRC, to R. C. Mecredy, RG&E, Subject: Issuance of Amendment 48 to Facility Operating License No. DPR-18, dated March 6, 1992.

56. Letter from K.C. Hoskins, Westinghouse, to R.W. Eliasz, RG&E, Subject: Reactor Coolant Pump Performance Curves, NTD-NSRLA-OPL-94-301, dated October 10, 1994.

57. Letter from G. S. Vissing, NRC, to R. C. Mecredy, RG&E, Subject: Issuance of Amendment No. 64 to Facility Operating License No. DPR-18, R. E. Ginna Nuclear Power Plant, (TAC No. M94770), dated May 23, 1996.

58. Letter from R. C. Mecredy, RG&E, to C. Stahle, NRC, Subject: Loss of Decay Heat Removal (Generic Letter 88-17), dated January 4, 1989.

59. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Closure of NRC Generic Letter 89-10, Safety-Related Motor-Operated Valve Testing and Surveillance, dated March 3, 1998.

60. Letter from E. M. Kelly, NRC, to R. C. Mecredy, RG&E, Subject: NRC Motor Operated Valve Inspection 50-244/98-06, dated June 12, 1998.

61. U.S. Nuclear Regulatory Commission, "Periodic Verification of Design-Basis Capability of Safety-Related Motor-Operated Valves," Generic Letter 96-05, dated September 18, 1996.

62. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to NRC Generic Letter 96-05, dated March 3, 1998.

63. Letter from G. S. Vissing, NRC, to R. C. Mecredy, RG&E, Subject: Safety Evaluation Regarding the Licensee's Response to Generic Letter 96-05 (TAC No. M97050), dated December 27, 1999.

64. Letter from R. C. Mecredy, RG&E, to G. S. Vissing, NRC, Subject: Response to NRC Generic Letter 97-06, Degradation of Steam Generator Internals, dated March 30, 1998.

65. Generic Letter 97-06, Degradation of Steam Generator Internals, dated December 30, 1997.

66. Letter from Herbert N. Berkow, NRC, to Robert H. Bryan, Chairman Westinghouse Owners Group, TVA, Subject: Safety Evaluation of Topical Report WCAP-15666, "Extension of Reactor Coolant Pump Motor Flywheel Examination," dated May 5, 2003.

67. Westinghouse Report; "GOTHIC Mid-loop Analysis to Support Ginna Plant," June 2006.

68. Westinghouse Report: "GOTHIC Mid-loop Analysis to Support Ginna Plant," October 2006.

69. Letter from M. Korsnick (Ginna) to Document Control Desk, NRC, "License Amendment Request Regarding Extended Power Uprate," (Letter No. 1001353) dated July 7, 2005.

70. NRC Letter from P. Milano to M. Korsnick (Ginna), "R.E. Ginna Nuclear Power Plant - Amendment RE: 16.8% Power Uprate (TAC No. MC7382)," dated July 11, 2006.

71. Westinghouse Letter, "Pressurizer Relief Tank Level Setpoints for Elevated Containment Temperature Conditions at R.E. Ginna Unit 1," LTR-SEE-III-13-26, Revision 0, dated March 4, 2014.

Table 5.4-1
REACTOR COOLANT PUMP DESIGN DATA

Number of pumps	2
Pump model	93
Design pressure/operating pressure, psig	2485/2235
Hydrostatic test pressure (cold), psig	3110
Design temperature (casing), °F	650
Nameplate rating, rpm	1189
Suction temperature, °F	556
Developed head, ft	252
Net positive suction head, ft	170
Capacity, gpm	90,000
Seal-water injection, gpm	8
Seal-water return, gpm	3
Pump discharge nozzle I.D., in.	27-1/2
Pump suction nozzle I.D., in.	31
Overall unit height, ft	28.22
Water volume, ft^3	192
Pump-motor moment of inertia, $lb\text{-}ft^2$	80,000
Motor data	
Type	ac induction single speed
Voltage	4000
Phase	3
Frequency, cps	60
Starting	
Input (hot reactor coolant), kW	4000
Input (cold reactor coolant), kW	5300
Power, hp (nameplate)	6000

Table 5.4-2
REPLACEMENT STEAM GENERATOR DESIGN DATA

Normal pressure, reactor coolant/steam outlet, psig	2235/755
Design pressure, reactor coolant/steam, psig	2485/1085
Reactor coolant, hydrostatic test pressure (tube side cold), psig	3310
Normal temperature, reactor coolant, °F[a]	540 - 611.9
Design temperature, reactor coolant/steam, °F	650/556
Reactor coolant flow, lb/hr (total)	64.8×10^6
Heat transferred, Btu/hr (total)[a]	6201×10^6
Steam conditions at full load, outlet nozzle	
Steam flow, lb/hr[a]	3.94×10^6
Steam temperature, °F[a]	521.5
Steam pressure, psia[a]	823
Feedwater temperature, °F[a]	435
Overall height, ft-in	63 - 1.63
Shell O.D., upper/lower, in.	166/127.5
Reactor coolant water volume, ft^3 [b]	969.6
Secondary side volume, ft^3 [b]	4513
Supplier	Babcock and Wilcox International
Number of tubes per steam generator	4765
Tube size	0.750 in. O.D., 0.043 in. average wall thickness

a. Start-up conditions with RCS T_{AVG}=576°F and reactor power of 1811 MWt (102% of full power)

b. Volumes at both 1525 Mwt and Zero Power

Table 5.4-3
REACTOR COOLANT PUMP COMPOSITE HOT PERFORMANCE CURVE DATA

Flow (GPM)	Total Head (FT)	BHP	Hydraulic Efficiency (%)
0	475.6	4667.1	0.00
0	478.8	4577.1	0.00
5647	464.3	4611.3	10.70
5797	460.3	4701.1	10.68
11294	449.7	4642.0	20.60
11595	445.1	4729.6	20.54
16941	435.3	4671.6	29.71
17392	430.0	4755.2	29.60
22588	420.9	4705.3	38.03
23190	414.9	4783.2	37.86
28234	414.5	4779.2	46.09
28987	407.8	4850.4	45.87
33881	418.3	4957.6	53.81
34785	410.7	5022.3	53.55
39528	425.5	5142.8	61.56
40582	417.1	5199.3	61.28
45175	421.2	5174.9	69.21
46380	412.0	5218.4	68.93
50822	411.4	5345.1	73.63
52177	401.5	5376.5	73.35
56469	401.1	5537.4	76.98
57974	390.5	5555.3	76.70
62116	387.7	5641.1	80.36
63772	376.5	5641.9	80.09
67763	371.4	5689.7	83.26
69569	359.5	5670.5	83.01
73410	352.7	5708.9	85.37
75367	340.1	5667.4	85.13
79056	332.8	5693.6	86.99

Flow (GPM)	Total Head (FT)	BHP	Hydraulic Efficiency (%)
81164	319.6	5627.2	86.76
84703	310.3	5646.4	87.62
86962	296.4	5552.6	87.37
90350	285.4	5549.2	87.45
92759	270.8	5425.3	87.16
95997	257.1	5389.1	86.21
98557	242.0	5232.0	85.80
101644	227.1	5161.0	84.19
104354	211.4	4967.5	83.59
107291	194.1	4810.8	81.49
110152	177.8	4576.8	80.56

Table 5.4-4
REACTOR COOLANT PUMPS COLD PERFORMANCE CURVE DATA FOR INDIVIDUAL IMPELLERS

IMPELLER S/N 1619				IMPELLER S/N 340			
Flow (gpm)	Total Head (ft)	BHP	Hyd. Eff.[a](%)	Flow (gpm)	Total Head (ft)	BHP	Hyd. Eff.[a](%)
0	471.7	5948	0.00	0	468.6	6066	0.00
5553	457.3	5992	10.70	5701	453.5	6111	10.68
11105	443.0	6032	20.60	11402	438.5	6148	20.54
16658	428.8	6071	29.71	17104	423.6	6181	29.60
22211	414.6	6115	38.03	22805	408.8	6217	37.86
27763	408.3	6211	46.09	28506	401.7	6305	45.87
33316	412.1	6442	53.81	34207	404.7	6528	53.55
38869	419.2	6683	61.56	39908	411.0	6758	61.28
44421	414.9	6725	69.21	45610	405.9	6783	68.93
49974	405.3	6946	73.63	51311	395.6	6989	73.35
55527	395.1	7196	76.98	57012	384.7	7221	76.70
61079	381.9	7331	80.36	62713	370.9	7334	80.09
66632	365.9	7394	83.26	68414	354.1	7371	83.01
72185	347.4	7419	85.37	74115	335.1	7367	85.13
77737	327.9	7399	86.99	79817	314.9	7314	86.76
83290	305.7	7338	87.62	85518	292.0	7218	87.37
88843	281.1	7211	87.45	91219	266.8	7052	87.16
94395	253.3	7003	86.21	96920	238.4	6801	85.80
99948	223.7	6707	84.19	102621	208.3	6457	83.59
105501	191.2	6252	81.49	108323	175.2	5949	80.56

a. Hydraulic Efficiency

Table 5.4-5
REACTOR VESSEL HEAD VENT EQUIPMENT PARAMETERS

VALVES	
Solenoid-operated globe valves	Cv = 2.0
	1 in. Schedule 160S connections
	Active valve per Regulatory Guide 1.48
	Operating design pressure - 2500 psig
	Design temperature - 680 °F
	Design humidity - 100%
	Radiation environment - post-accident 10^8 rads (beta) and 1.43 x 10^7 (gamma)
	Design code - ASME Section III, 1974, Class 2
	Seismic Category I
	Quality Group B
	Fail closed
	Red/green main control board status lights (Reed switches)
Manual globe valve 500 and 500B	Design pressure - 2500 psia
	Design temperature - 650°F
	Material - austenitic stainless steel
	Design code - ASME - Section III 1995 edition with 1996 addenda, Safety Class I
	Seismic Category I
	Cv = 4.0
	Quality Group A
Manual globe valves 592A and 593A	Design pressure - 2500 psig
	Design temperature - 650°F
	Material - austenitic stainless steel
	Design Code - ASME Section III, Safety Class 2
	Seismic Category I
	Nonactive valves
	Quality Group B

PIPING

Existing vent pipe	3/4 in. Schedule 80S
	Code compliance - ANSI B31.1
New piping (to head vent system)	3/4 in. and 1 in. Schedule 160S
	Code compliance - ASME Section III, 1977, Classes 1 and 2
PIPING SUPPORTS AND SUPPORT STRUCTURES	Code compliance - ASME, Section III, 1977, Subsection NF for new supports and structures

Table 5.4-6
RESIDUAL HEAT REMOVAL SYSTEM COMPONENT DESIGN DATA

Reactor coolant temperature at startup of decay heat removal, °F	350
Time to cool reactor coolant system from 350 °F to 140 °F, hour	73/101[a]
Refueling water storage temperature, °F	Ambient
Decay heat generation at 20 hours after shutdown condition, Btu/hr	37.4×10^6
Reactor cavity fill time, hour	1
Reactor cavity drain time, hour	4
H_3BO_3 concentration in refueling water storage tank (RWST), ppm boron	2750-3050

RESIDUAL HEAT REMOVAL PUMPS

Quantity	2
Type	Horizontal centrifugal
Design rated capacity (each), gpm	1560
Head at rated capacity, ft H_2O	280
Motor horsepower	200
Material	Stainless steel
Design pressure, psig	600
Design temperature, °F	400

SUMP PUMPS (AUXILIARY BUILDING)

Quantity	2
Type	Vertical, duplex
Capacity, gpm	50
Head, ft	55
Motor horsepower	1.5
Material (wetted surface)	Stainless steel

RESIDUAL HEAT REMOVAL HEAT EXCHANGERS

Quantity	2
Type	Shell and U-tube
Heat transferred, Btu/hr	[b]24.15×10^6
Reactor coolant flow, lb/hr (tube side)	763,000
Cooling water flow (each), gpm (shell side)	2780[b]
Cooling water inlet temperature, °F	100
Material, shell/tube	Carbon steel/stainless steel
Design pressure, shell/tube, psig	150/600
Design temperature, °F	350/400

a. The 20 hour cooldown times are for 80°F/85°F lake temperatures and two functional CCW and RHR heat exchangers. Times also assume RCP heat addition from one RCP until 160°F.

b. To minimize the potential for flow induced vibration in the residual heat removal heat exchangers, as of 1994 component cooling water flow has been limited to approximately 1800 gpm through the shell side of each heat exchanger. See Section 9.2.2.4.1.6.

Table 5.4-7
PRESSURIZER DESIGN DATA

Design/operating pressure, psig	2485/2235
Hydrostatic test pressure (cold), psig	3110
Design/operating temperature, °F	680/653
Water volume,[a]	475
Steam volume, full power, ft^3	325
Surge line diameter, in	10
Spray lines (2) diameter, in.	3
Spray flow, maximum, gpm per valve	200
Surge line nozzle diameter, in./pipe schedule	14 / Sch. 140
Shell I.D., in./calculated minimum shell thickness, in.	84/4.1
Minimum clad thickness, in.	0.188
Electric heaters capacity, kW	800
Heatup rate of pressurizer using heaters only, °F/hr	55 (approximately)
POWER-OPERATED RELIEF VALVES (PORV)	
Number	2
Set pressure (open), psig	2335
Capacity, lb/hr saturated steam/valve	179,000
SAFETY VALVES	
Number	2
Set pressure, psig	2485
Capacity, lb/hr saturated steam/valve	288,000 at 2485 psig + 3% accumulation

a. Based on full power pressurizer level of 61.2% at an RCS T_{AVG}=576°F.

Table 5.4-8
PRESSURIZER RELIEF TANK DESIGN DATA

Design pressure, psig	100
Rupture disk release pressure, psig	100
Design temperature, °F	340
Normal water temperature, °F	Containment ambient
Total volume, ft^3	800
Rupture disk relief capacity, lb/hr	7.20×10^5

Table 5.4-9
VALVE AND PIPING INFORMATION

SAFETY VALVE INFORMATION	
Number of valves	2
Manufacturer	Crosby Valve and Gauge
Type	Self-actuated
Size	4K26
Steam flow capacity, lb/hr/valve	288,000
Design pressure, psig	2485
Design temperature, °F	650
Set pressure, psig	2485
Accumulation	3% of set pressure
Blowdown	5% of set pressure
Original valve procurement specification	E-676279
RELIEF VALVE INFORMATION	
Number of valves	2
Manufacturer	Copes-Vulcan
Type	Pressurizer power-operated relief
Size	3 in. - NPS
Steam flow capacity, lb/hr/valve	179,000
Design pressure, psi	2485
Design temperature, °F	680
Opening pressure, psig	2335
Closing pressure, psig	2315
SAFETY AND RELIEF VALVE INLET PIPING INFORMATION	
Design pressure, psig	2485
Design temperature, °F	650
Loop seal volume, ft^3	0.18

SAFETY AND RELIEF VALVE DISCHARGE PIPING INFORMATION

Design pressure, psig	600
Design temperature, °F	650
Pressurizer relief tank design pressure, psig	100
Backpressure, normal, psig	3 to 5
Backpressure, developed, psig	350

BLOCK VALVE INFORMATION

Number of valves	2
Manufacturer	Anchor Darling
Type	Motor-operated double-disk gate
Size	3 in.
Steam flow capacity, lb/hr/valve	179,000
Design pressure, psi	2485
Design temperature, °F	650
Leakage limit, water/hr/in. diameter	10 cm^3
Stroke time, open or close	≈12 seconds
Motor operator	Limitorque SMB-00-15

Figure 5.2-1 Figure DELETED

Figure 5.2-2 Figure DELETED

Figure Deleted

Figure 5.2-3 Reactor Coolant Leak Detection Sensitivity

METHOD 1 - SENSITIVITY

REACTOR COOLANT ACTIVITY

Xe-138 = 0.04 μCi/cc
Kr-88 = 0.01 μCi/cc

LEAK DETECTION TIME

(FOR MAXIMUM CIRCULATION IN CONTAINMENT)

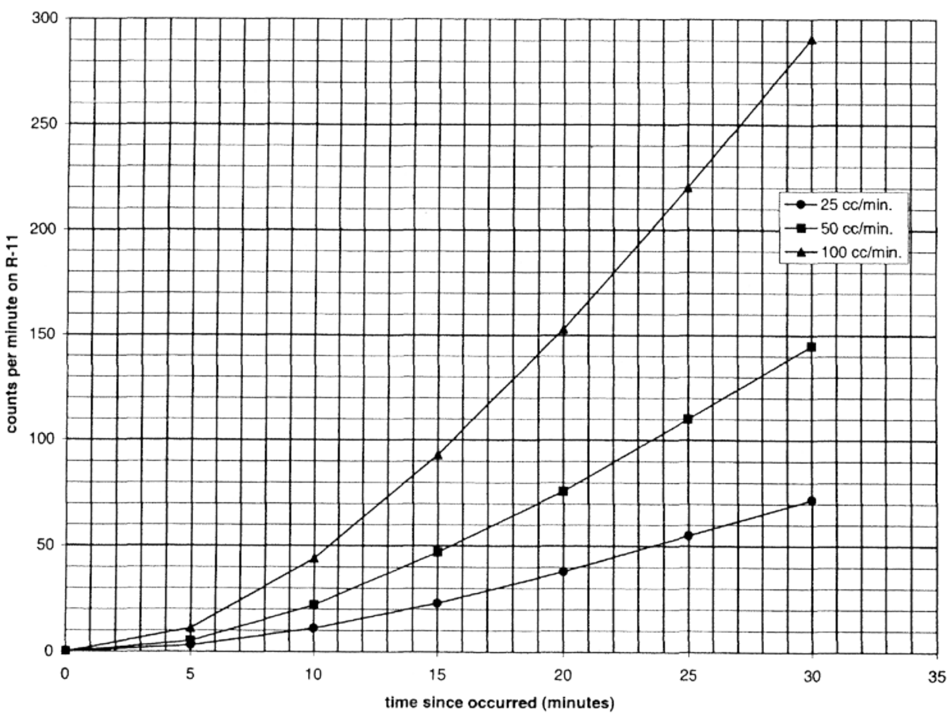
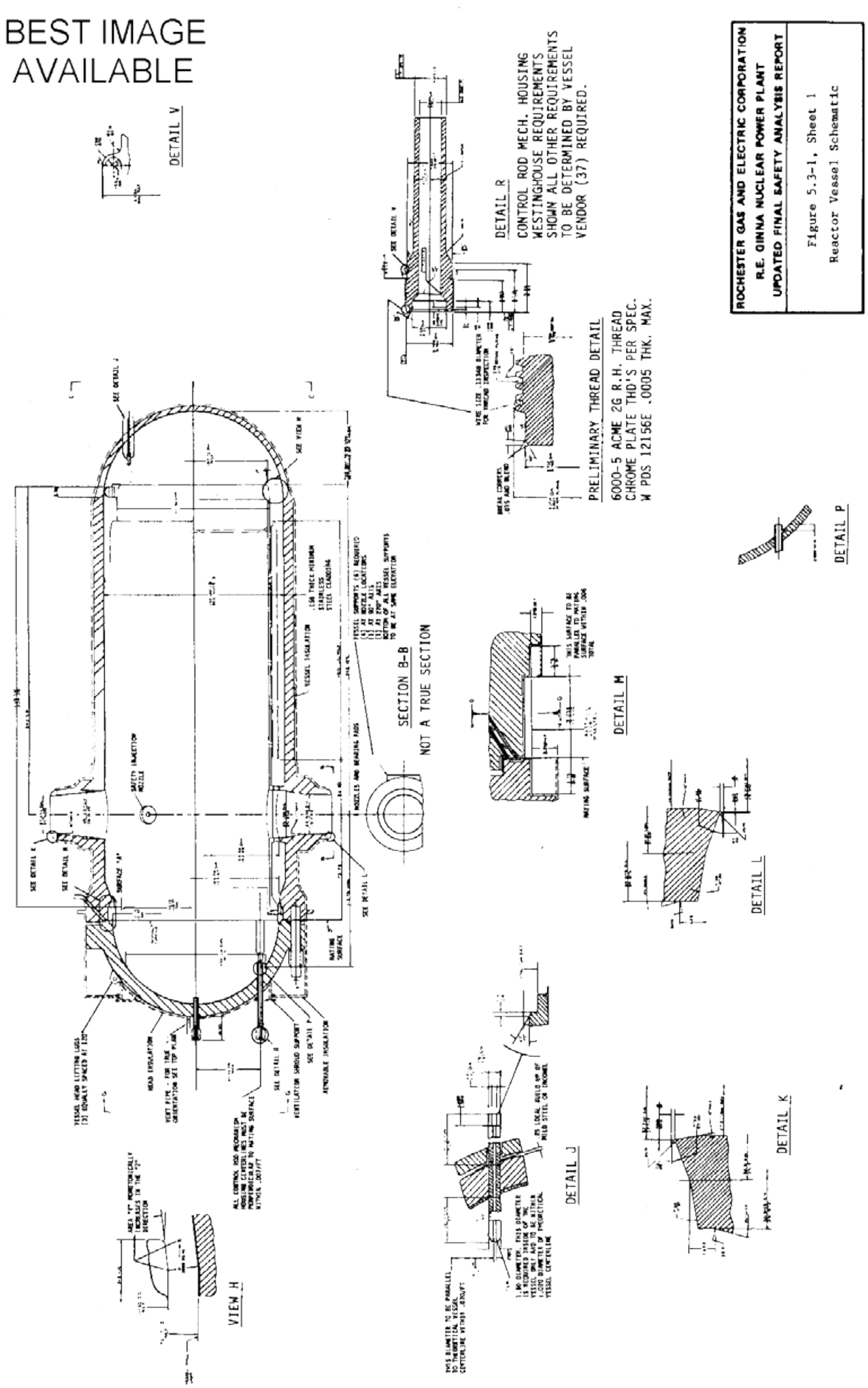


Figure 5.3-1 Reactor Vessel Schematic

BEST IMAGE
AVAILABLE



Sheet 2 of Figure 5.3-1

BEST IMAGE AVAILABLE

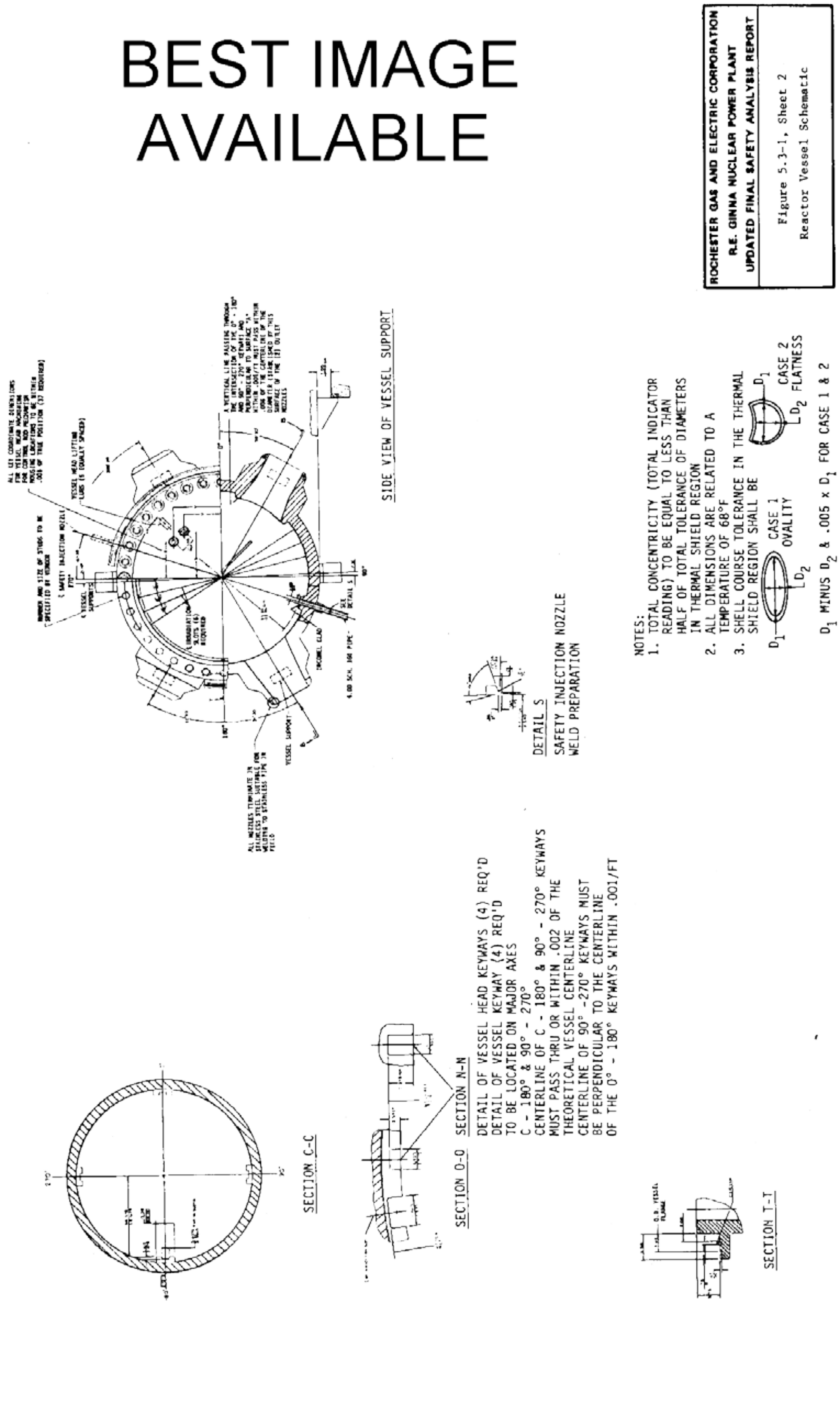


Figure 5.3-2 Identification and Location of Beltline Region Material

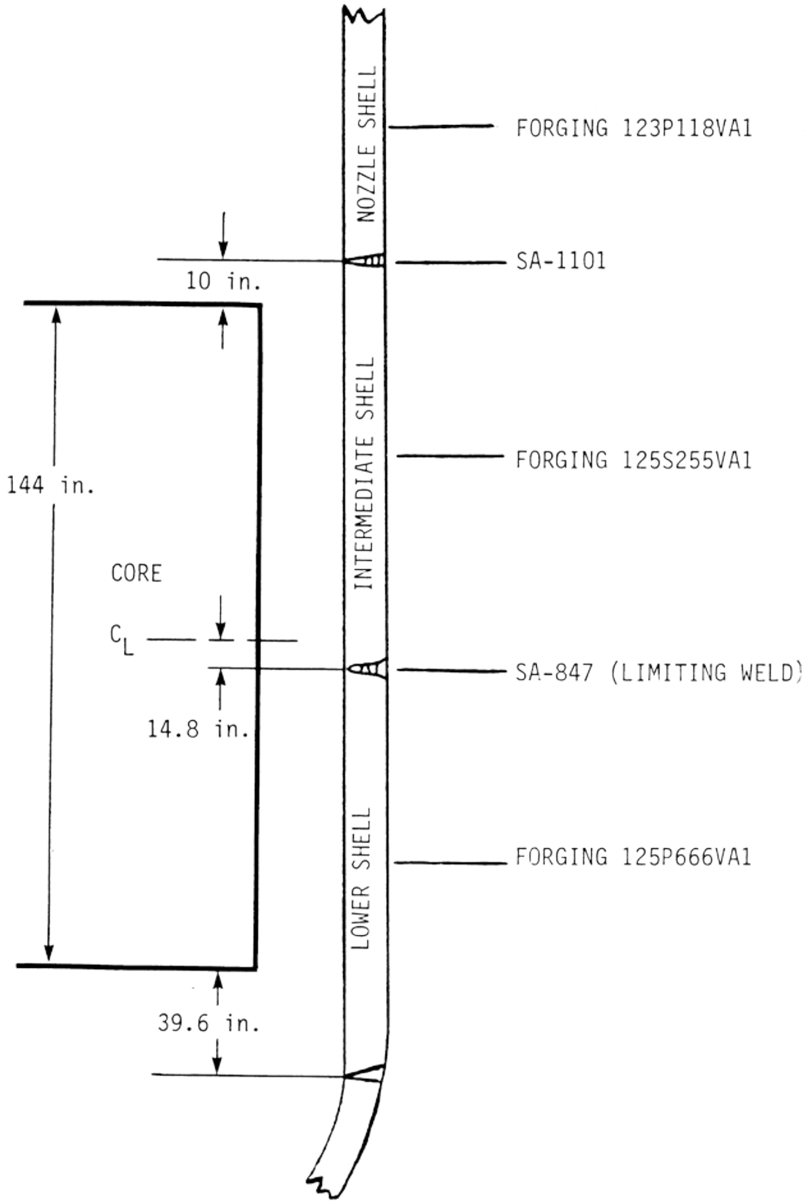


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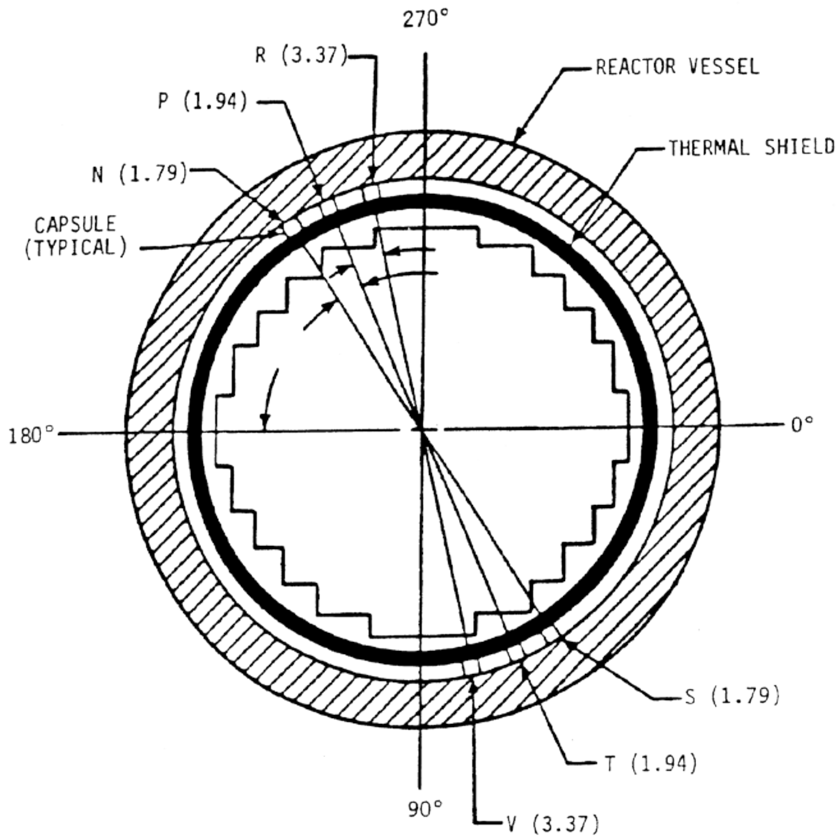
Figure 5.3-2 Identification and Location of Beltline Region Material

REV 7 12/91

Figure 5.3-3 Arrangement of Surveillance Capsules in the Reactor Vessel



NOTES:

1. Lead Factors for the capsules are shown in parentheses.

2. In chronological order, capsules V, R, T, S, and N were removed. Ref. Sections 5.3.3.2 and 5.3.3.3.

Figure 5.4-1 Reactor Coolant Pump

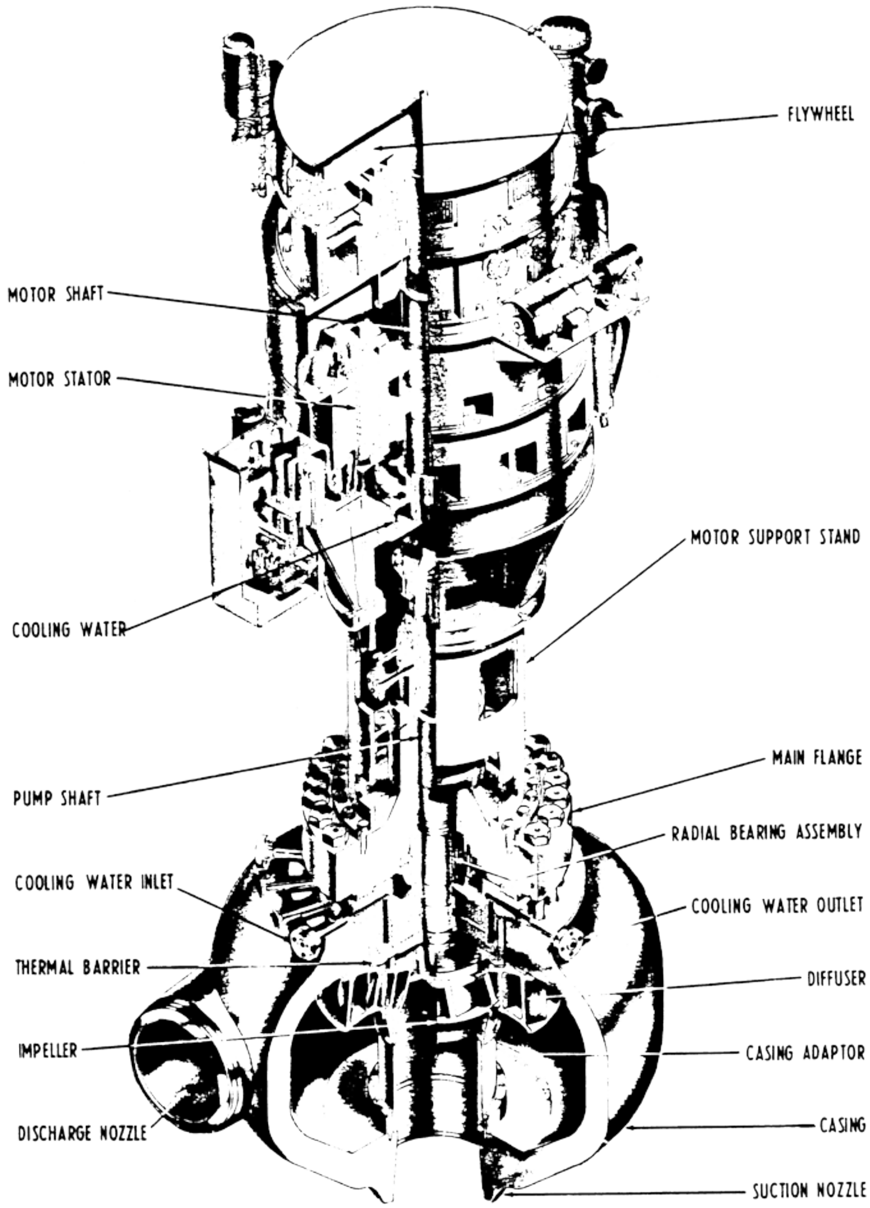


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Figure 5.4-1
Reactor Coolant Pump

Figure 5.4-2 Reactor Coolant Pump Estimated Performance Characteristics

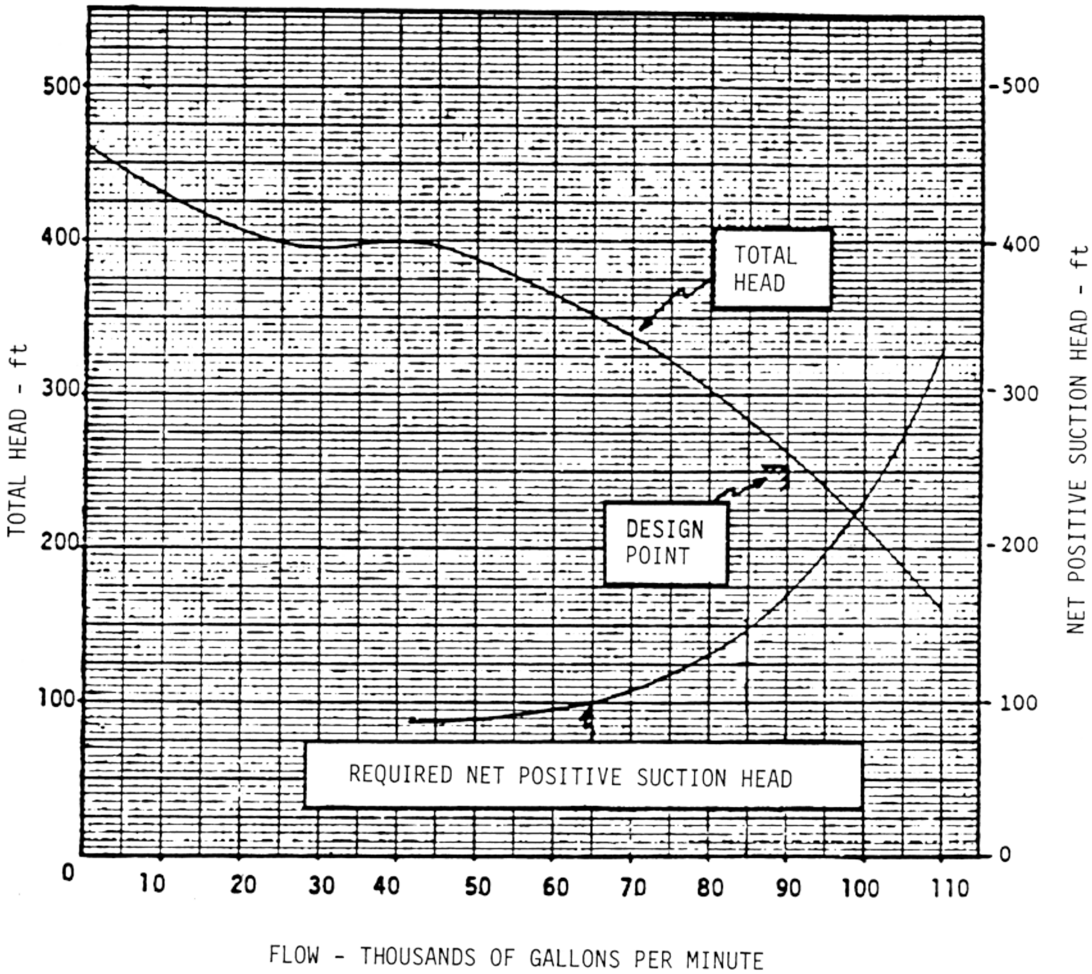

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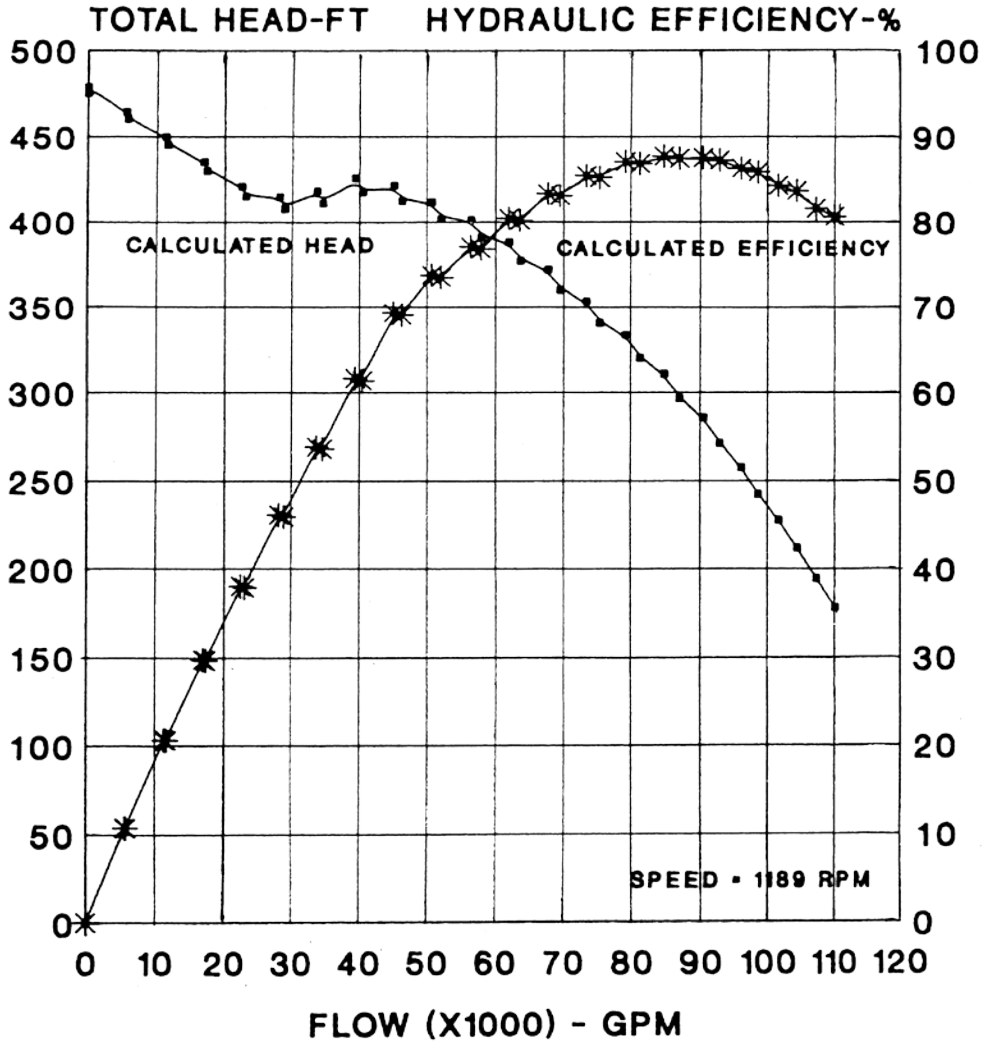
Figure 5.4-2 Reactor Coolant Pump Estimated Performance Characteristics

Figure 5.4-2a Reactor Coolant Pump Composite Curve, Calculated Hot Performance, Total Head and Hydraulic Efficiency Versus Flow



IMP. S/N 1619/340
Qdesign = 90000 GPM
Hdesign = 252 FT

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Figure 5.4-2a Reactor Coolant Pump Composite Curve, Calculated Hot Performance, Total Head and Hydraulic Efficiency Versus Flow

REV. 13-1 7/96

Figure 5.4-2b Reactor Coolant Pump Composite Curve, Calculated Hot Performance, Brake Horsepower Versus Flow

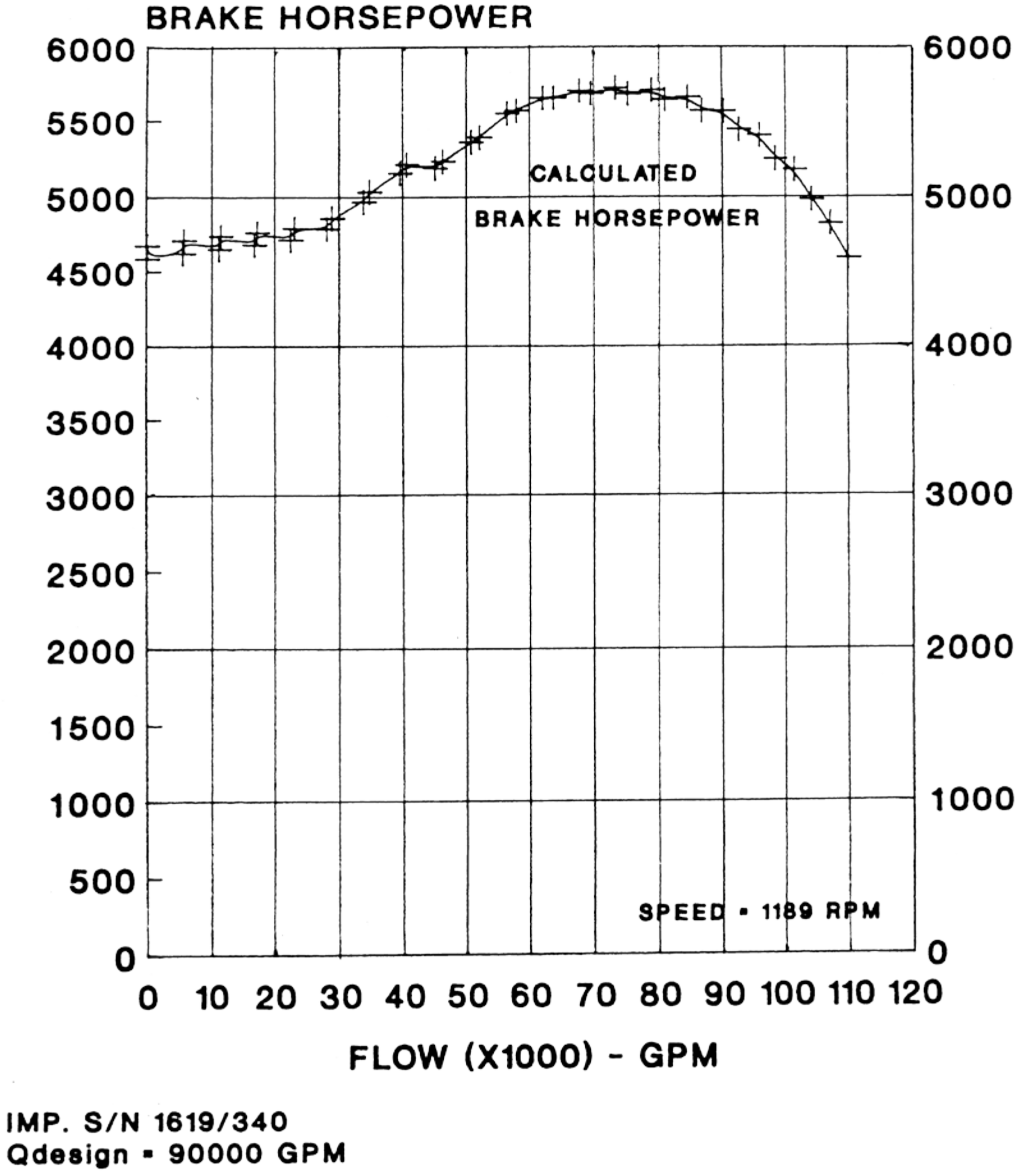

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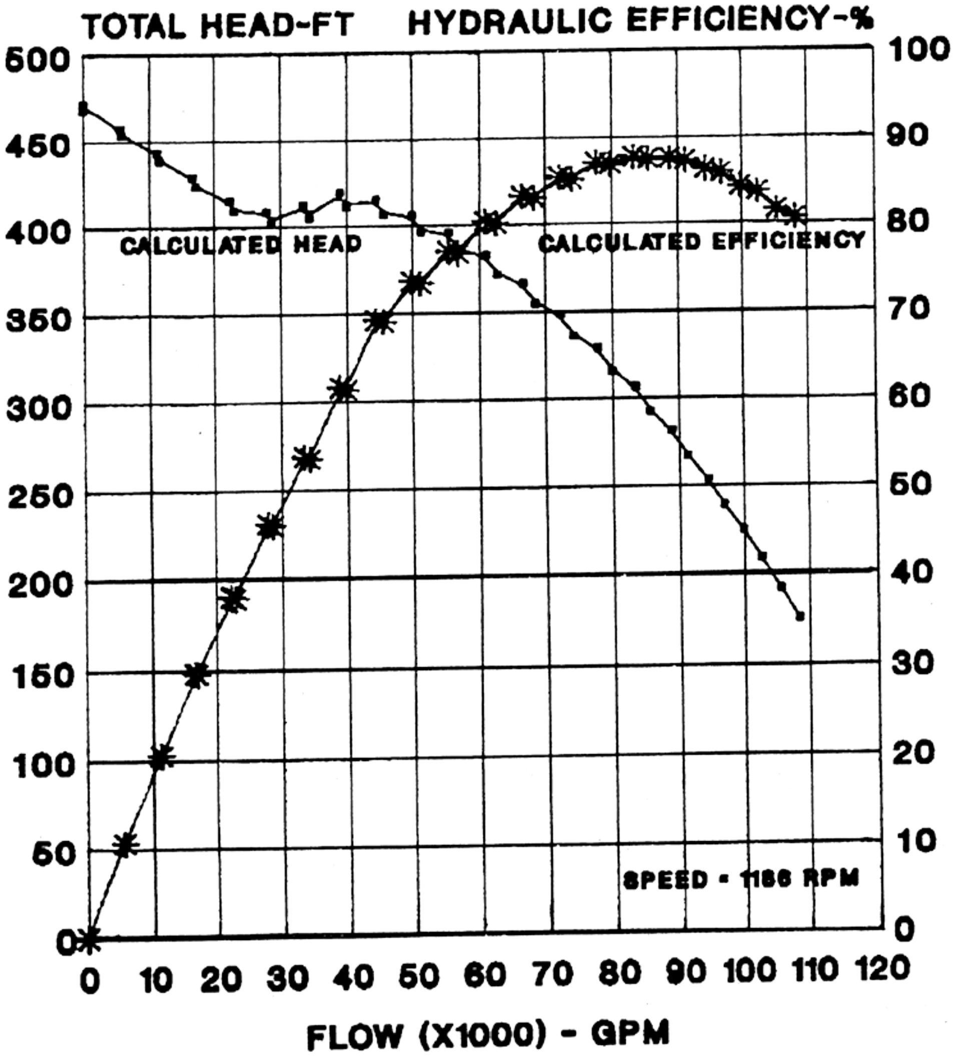
Figure 5.4-2b Reactor Coolant Pump Composite Curve, Calculated Hot Performance, Break Horsepower Versus Flow

REV. 13-1 7/96

Figure 5.4-2c Reactor Coolant Pump Composite Curve, Calculated Cold Performance, Total Head and Hydraulic Efficiency Versus Flow



IMP. S/N 1619/340
Qdesign = 90000 GPM

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Figure 5.4-2c Reactor Coolant Pump Composite Curve, Calculated Cold Performance, Total Head and Hydraulic Efficiency Versus Flow

REV. 13-1 7/96

Figure 5.4-2d Reactor Coolant Pump Composite Curve, Calculated Cold Performance, Brake Horsepower Versus Flow

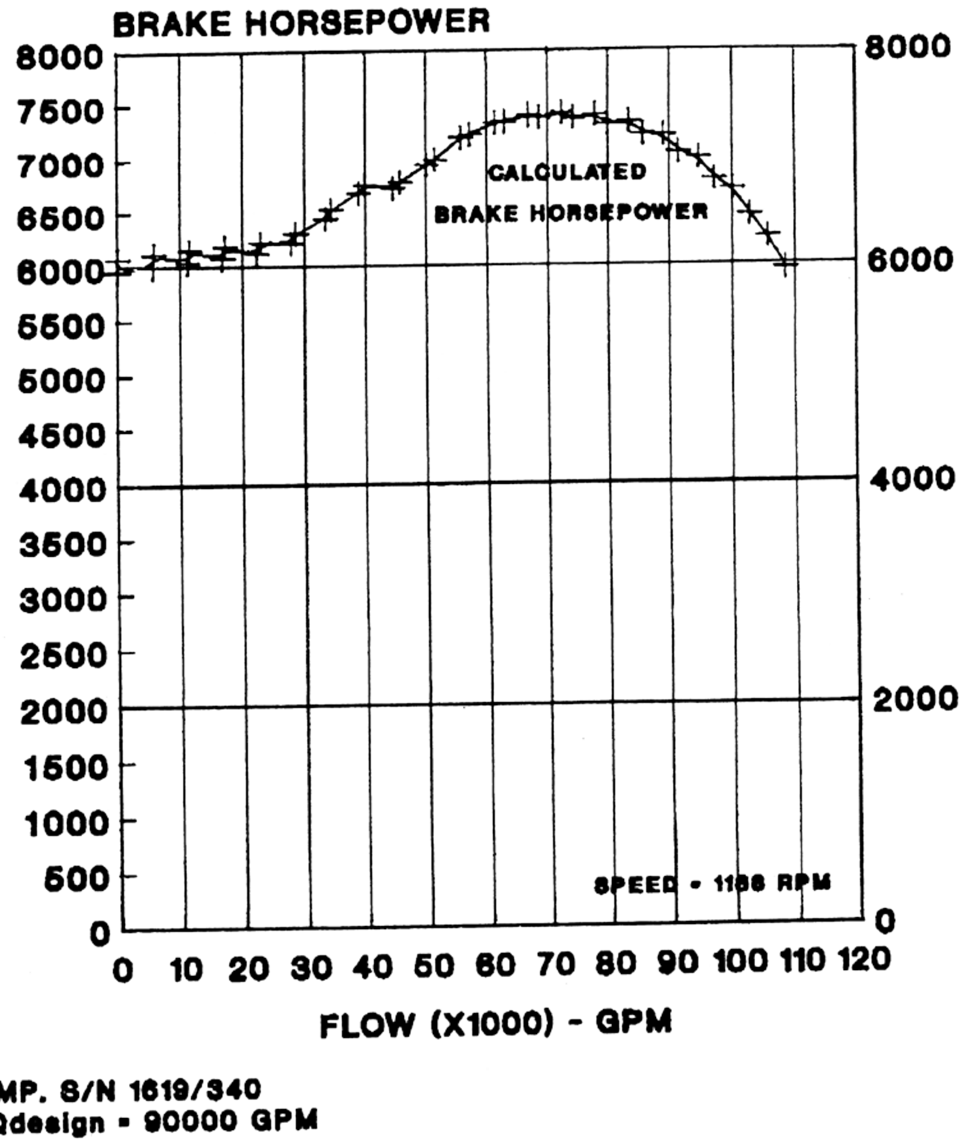

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Figure 5.4-2d Reactor Coolant Pump Composite Curve, Calculated Cold Performance, Brake Horsepower Versus Flow

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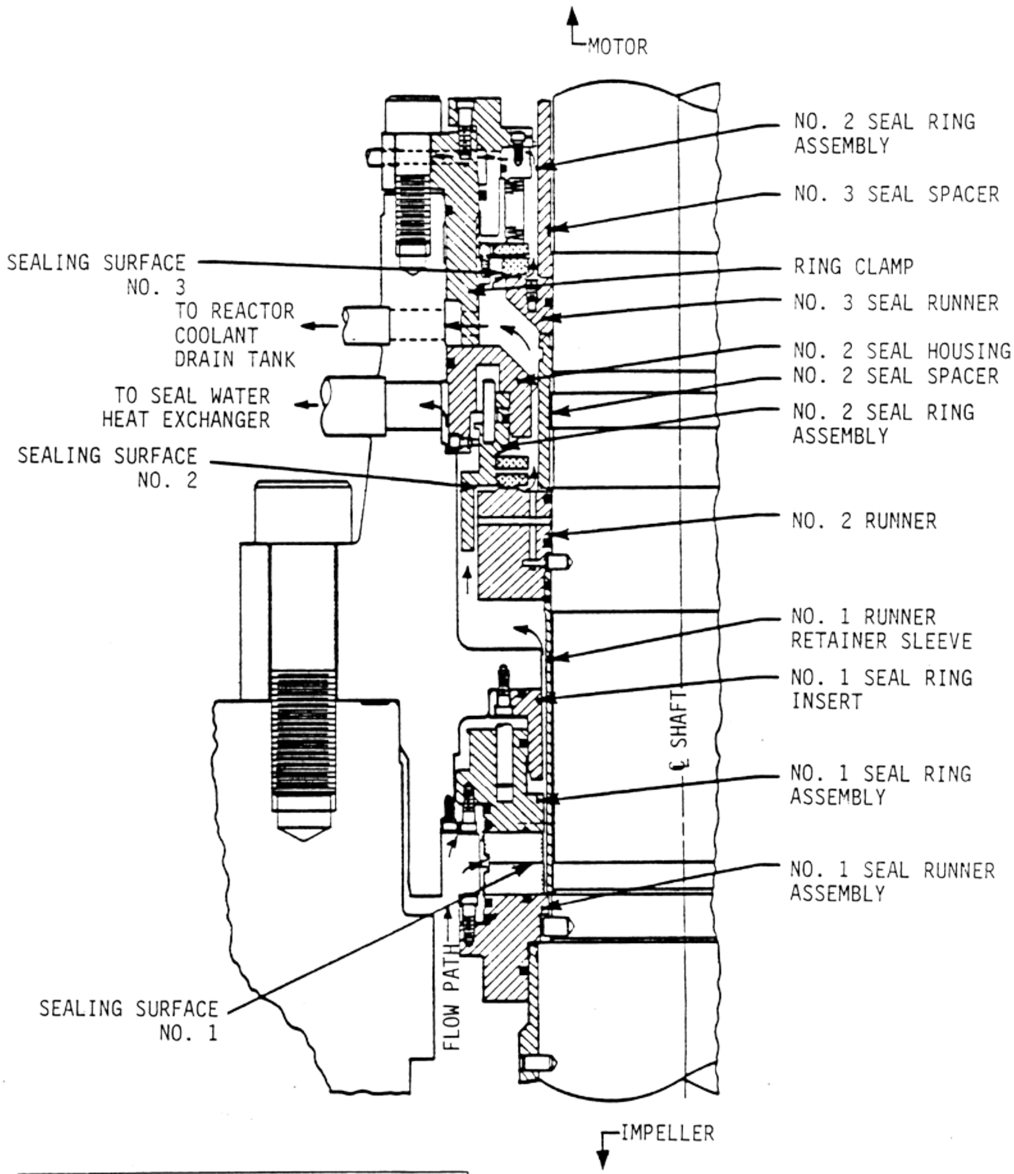
Figure 5.4-3 Reactor Coolant Pressure Shaft Seal Arrangement



PARAMETERS		
SEAL	INLET PRESS (psia)	FLOW RATE
NO. 1	∿2250	∿3 gpm
NO. 2	∿50	∿3 gph
NO. 3	∿18	∿100 cm³/hr

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Figure 5.4-3

Reactor Coolant Pressure Shaft Seal Arrangement

REV 2 12/86

Figure 5.4-4 Reactor Coolant Pump Flywheel

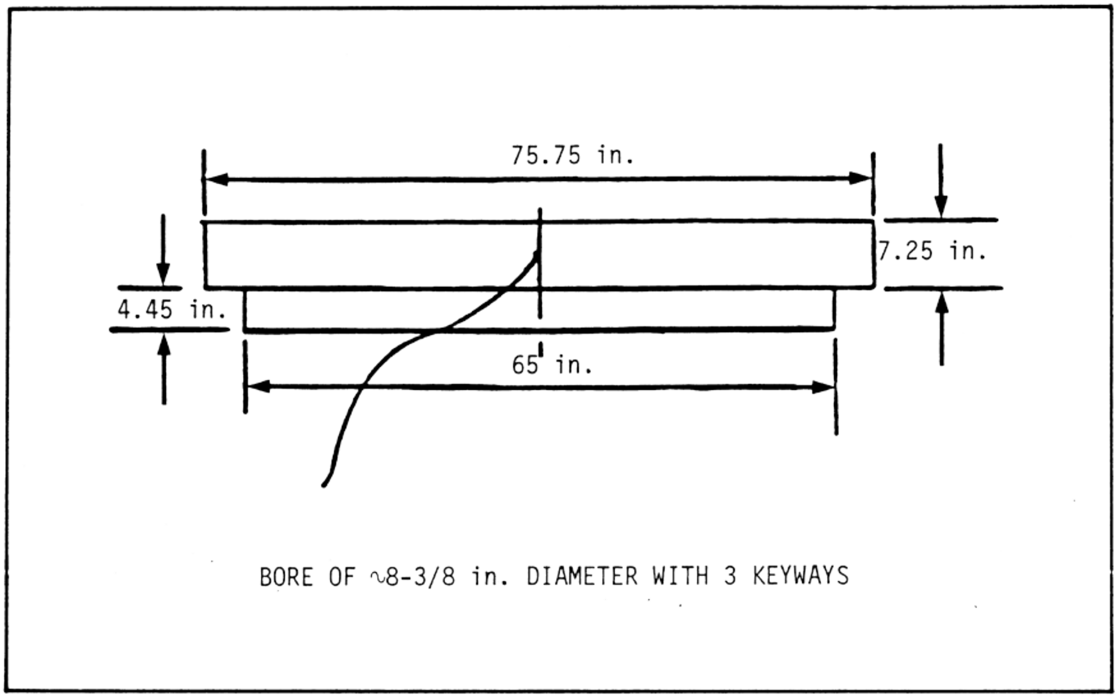

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Figure 5.4-4
Reactor Coolant Pump Flywheel

Figure 5.4-5 Reactor Coolant Pump Flywheel Primary Stress at Operating Speed

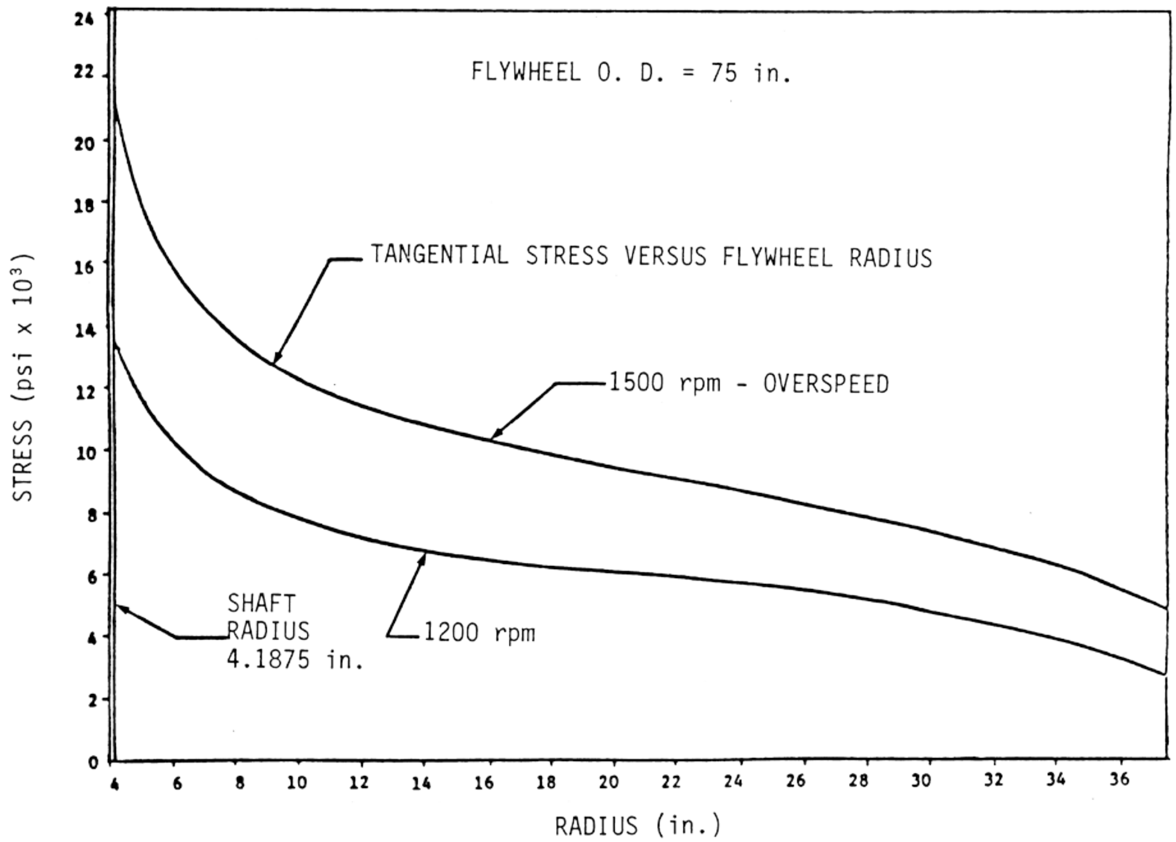

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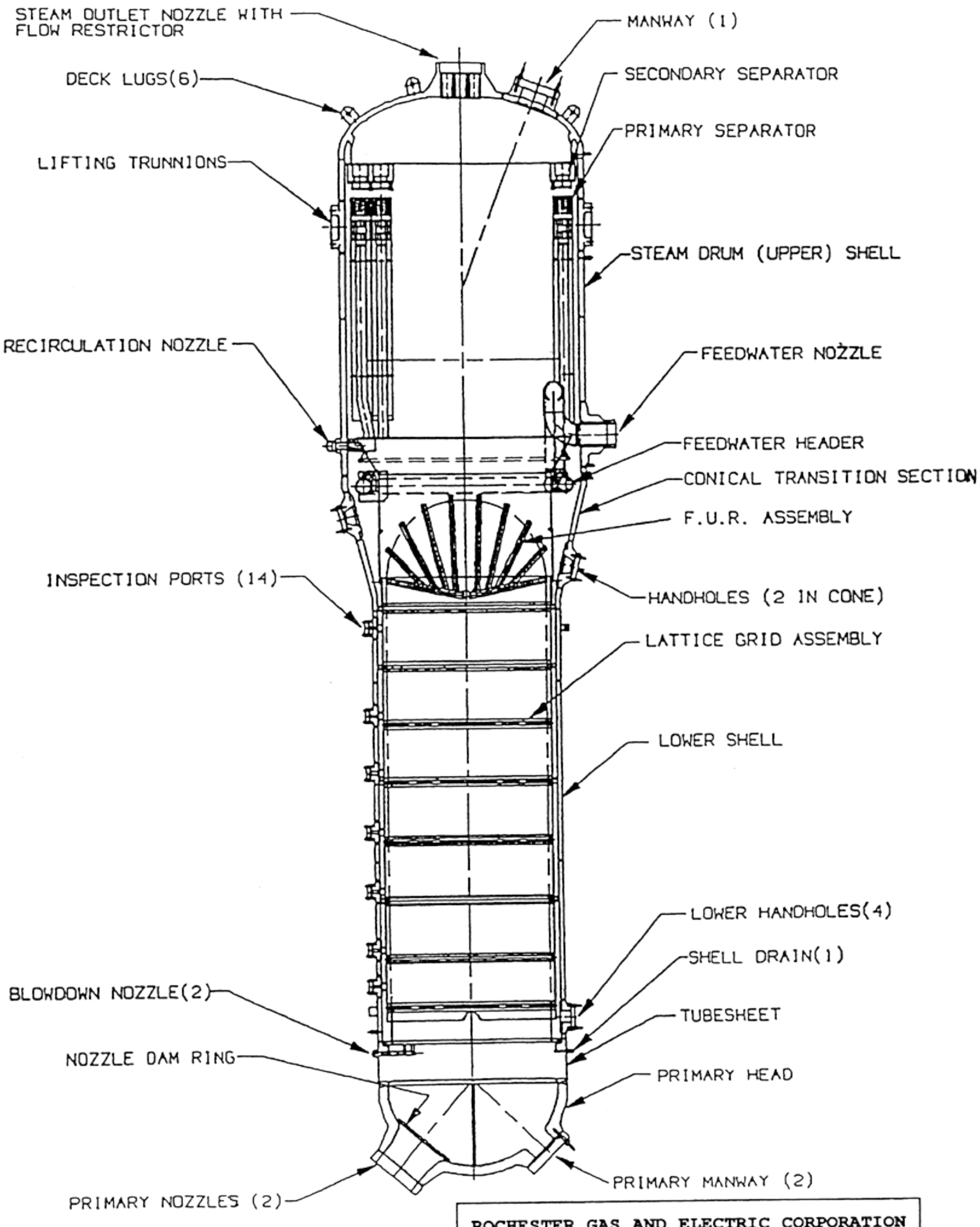
Figure 5.4-5

Reactor Coolant Pump Flywheel Primary Stress at Operating Speed

Figure 5.4-6 Replacement Steam Generator



ROCHESTER GAS AND ELECTRIC CORPORATION
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Figure 5.4-6

Replacement Steam Generator

REV. 13-1 7/96

Figure 5.4-7 Figure DELETED

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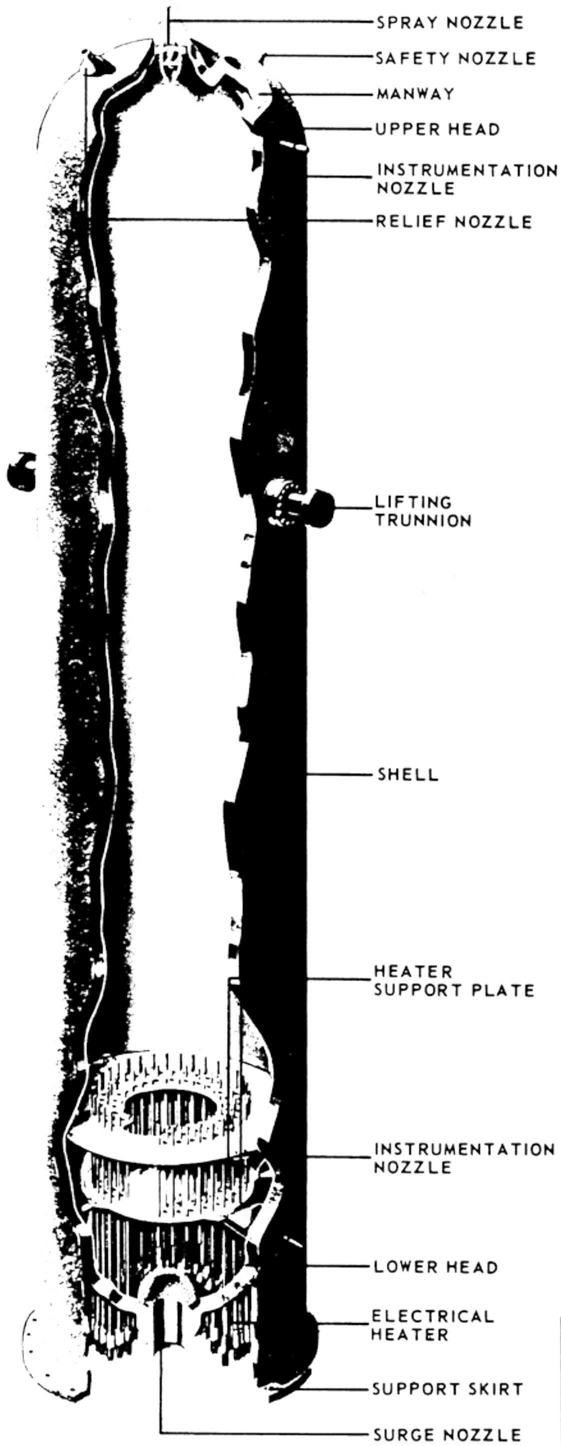
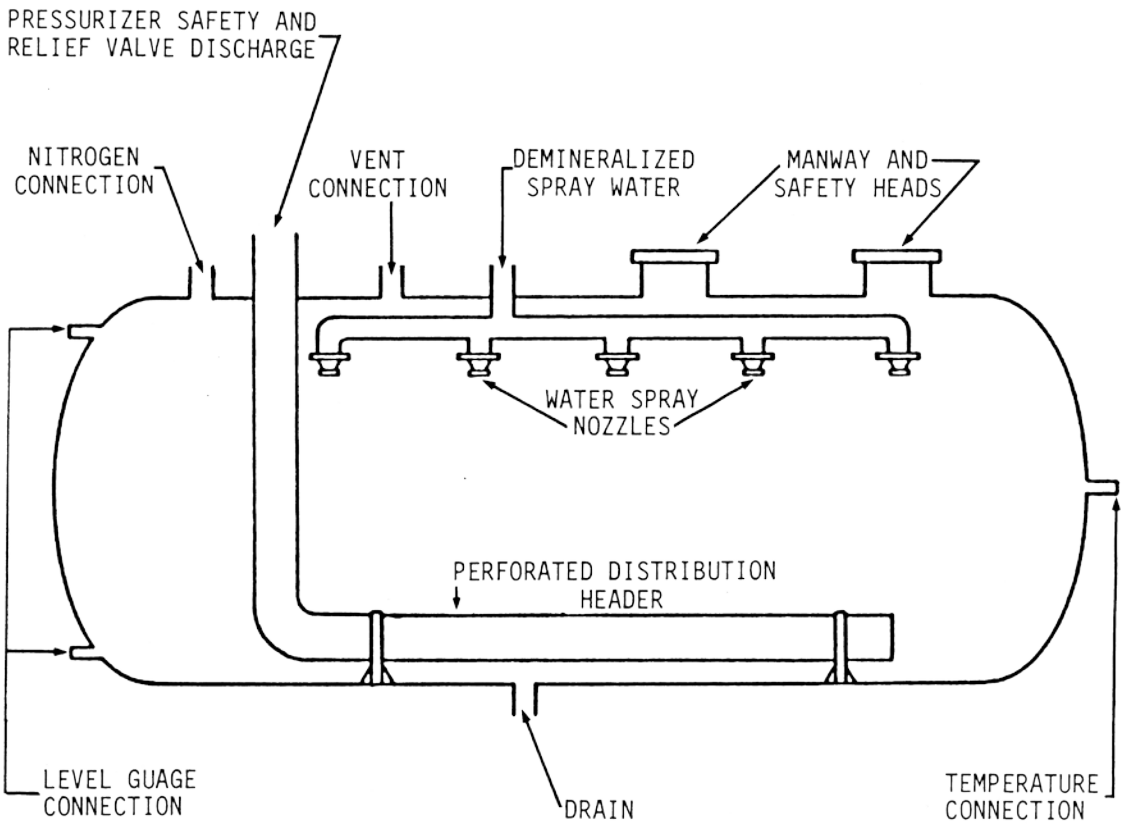
Figure 5.4-8 Pressurizer



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 5.4-8 Pressurizer

Figure 5.4-9 Pressurizer Relief Tank



ROCHESTER GAS AND ELECTRIC CORPORATION R.E. GINNA NUCLEAR POWER PLANT UPDATED FINAL SAFETY ANALYSIS REPORT
Figure 5.4-9 Pressurizer Relief Tank