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CHAPTER 3.0

3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

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3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

3.1 CONFORMANCE WITH GENERAL DESIGN CRITERIA

3.1.1 INTRODUCTION AND SUMMARY

3.1.1.1 Introduction

The General Design Criteria (GDC) in existence at the time HBR 2 was licensed (July, 1970) for operation were contained in Proposed Appendix A to 10CFR50, General Design Criteria for Nuclear Power Plants, published in the Federal Register on July 11, 1967. (Appendix A to 10CFR50, effective in 1971 and subsequently amended, is somewhat different from the proposed 1967 criteria.)

HBR 2 was evaluated with respect to the proposed 1967 GDC and the original FSAR contained a discussion of the criteria as well as a summary of the criteria by groups. Section 3.1.1.2 and 3.1.2 present that discussion without substantive change in order to preserve the original basis for licensing. It is noted, however, that in some cases additional analyses and/or plant modifications have been implemented. Thus, in those cases the margins of safety will be equal to or greater than is indicated in the following sections.

3.1.1.2 Summary

3.1.1.2.1 Overall Plant Requirements (GDC 1-GDC 5)

All systems and components of the facility are 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 an accident or result in an uncontrolled release of excessive amounts of radioactivity are 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 are designated Class II. Those items not related to reactor operation or safety are designated Class III.

Class I systems and components are essential to the protection of 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.

All systems and components designated Class I are designed so that there is no loss of function in the event of the maximum hypothetical ground acceleration acting in the horizontal and vertical directions simultaneously. The working stress for both Class I and Class II items are kept within code allowable values for the design earthquake. Similarly, measures are taken in the plant design to protect against high winds, sudden barometric pressure changes, flooding, and other natural phenomena. The Containment and Auxiliary Buildings are designed to withstand the effects of a tornado.

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Fire prevention in all areas of the nuclear unit is provided by structure and component design which maximizes the use of fire-resistant materials, optimizes the containment of combustible materials, and maintains exposed combustible materials below their ignition temperature in the design atmosphere. Fixed or portable fire fighting equipment are provided with capacities proportional to the energy that might credibly be released by fire. The Fire Protection System provided has the design capability to extinguish any probable combination of simultaneous fires which might occur at the plant.

A complete set of as-built facility plant and system diagrams, including arrangement plans and structural plans, and records of initial tests, and operation are maintained throughout the life of the reactor. A set of all the Quality Assurance (QA) data generated during fabrication and erection of the essential components of the plant, as defined by the QA program, is retained.

3.1.1.2.2 Protection by Multiple Fission Product Barriers (GDC 6-GDC 10)

The reactor core, with its related control and protection system, was 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 normal operation with appropriate margins for uncertainties and anticipated transient situations.

The Reactor Control and Protection System was 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 specified in Section 4.4.

The design of the reactor core and related protection systems ensures that there are no power oscillations which could cause fuel damage in excess of acceptable limits or any oscillation can be readily suppressed. See Section 3.1.2.7 for a discussion of how power oscillations are suppressed.

The Reactor Coolant System (RCS) in conjunction with its control and protective provisions was 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.

The design pressure and temperature of the containment exceeds the peak pressure and temperature occurring as the result of the complete blowdown of the reactor coolant through any pipe rupture of the RCS up to and including the hypothetical severance of a reactor coolant pipe.

3.1.1.2.3 Nuclear and Radiation Controls (GDC 11-GDC 18)

The plant is equipped with a Control Room which contains the controls and instrumentation necessary for operation of the reactor and turbine generator under normal and accident conditions.

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Sufficient shielding, distance, and containment integrity are provided to ensure that calculated Control Room personnel doses are within the limits described in Section 6.4.

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.

Other instrumentation and control systems are provided to monitor and maintain within prescribed operating ranges the temperatures, pressure, flow, and levels in the RCS, Steam Systems, Containment, and other Auxiliary Systems. The quantity and types of instrumentation provided are adequate for safe and orderly operation of all systems and processes over the full operating range of the plant.

The operational status of the reactor is monitored from the Control Room. When the reactor is subcritical the neutron source multiplication is continuously monitored and indicated by proportional counters located in instrument wells in the primary shield adjacent to the reactor vessel. The source detector channels are checked by the use of an incore source prior to operations in which criticality may be approached. 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. 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.

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 containment isolation, 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.

If the reactor protection system receives signals which are indicative of an approach to an unsafe operating condition, the system actuates alarms, prevents control rod motion, initiates load cutback, and/or opens the reactor trip breakers.

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The basic reactor tripping philosophy is to define an allowable region of power and coolant temperature conditions. This allowable range is defined by the primary tripping functions, the overpower high ΔT trip, over-temperature high ΔT trip, and the nuclear overpower trip. The operating region below these trip settings was designed so that no combination of power, temperatures, and pressure could result in a DNBR less than the safety limit (specified in Section 4.4) with all reactor coolant pumps (RCP) in operation. Additional tripping functions are discussed in Section 3.1.2.14.

Rod stops from nuclear overpower, overpower ΔT and over-temperature ΔT deviation are provided to prevent abnormal power conditions which could result from excessive control rod withdrawal initiated by a malfunction of the reactor control system or by operator violation of administrative procedures.

The Engineered Safety Features (ESF) and Protection Systems are designed with redundant circuits and controls to limit the effects of anticipated transients (Section 3.1.2.15).

The system for monitoring reactor coolant pressure boundary leakage is designed for detection of deviations from normal containment environmental conditions.

The containment atmosphere, fuel handling building exhaust, the plant vent, the containment fan-coolers service water discharge, the condenser vacuum pump exhaust, the steam generator (SG) blowdown effluents, and the Waste Disposal System (WDS) liquid effluent are monitored for radioactivity concentration during all normal operations, anticipated transients, and accident conditions.

Monitoring and alarm instrumentation are 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 of radioactive gases and liquids.

3.1.1.2.4 Reliability and Testability of Protection Systems (GDC 19-GDC 26)

Upon a loss of power to the coils, the rod cluster control (RCC) assemblies are released and fall by gravity into the core. The RCC assemblies are never fully withdrawn from their guide thimbles in the fuel assembly. As a result of design safeguards and the flexibility designed into the RCC assemblies, abnormal loadings and misalignments can be sustained without impairing operation of the RCC assemblies.

Protection channels are designed with sufficient redundancy for individual channel calibration and test (tripped independently by simulated signals) to be made during operation without degrading the reactor protection function. Bypass removal of one trip circuit is accomplished by placing that channel in a partial-tripped mode; i.e., a two-out-of-three channel becomes a one-out-of-two channel.

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Channel independence is carried throughout the system extending from the sensor to the relay actuating the protective function. The protective and control functions when combined are combined only at the sensor. A failure in the control circuit does not affect the protection channel. The power supplied to the channels is fed from four instrument buses. Two of the buses are supplied both by constant voltage transformers and two are supplied by inverters powered from DC batteries.

The initiation of the engineered safety features provided for loss-of-coolant accidents (LOCA), e.g., high head safety injection and residual heat removal (RHR) pumps, and Containment Spray System (CSS) is accomplished from redundant signals derived from RCS and containment instrumentation. The initiation signal for containment spray comes from coincidence of two sets of two-out-of-three high containment pressure signals. On loss of voltage of a safety features equipment power supply bus, the diesel generator (DG) will be automatically started and connected to the bus.

The components of the protective system are designed and laid out 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.

3.1.1.2.5 Reactivity Control (GDC 27-GDC 32)

In addition to the reactivity control achieved by the RCC, reactivity control is provided by the Chemical and Volume Control System (CVCS) which regulates the concentration of boric acid solution neutron absorber in the RCS. The system is designed to prevent uncontrolled or inadvertent reactivity changes which might cause system parameters to exceed design limits.

The reactivity control system provided are capable of making and holding the core subcritical from any hot standby or hot operating condition, including those resulting from power changes.

Any time that the plant is at power, the quantity of boric retained in the boric acid tanks and ready for injection will always exceed that quantity required for normal cold shutdown.

The Reactor Protection System (RPS) is designed to limit reactivity transients to a DNBR \square the safety limit specified in Section 4.4 due to any single malfunction in the deboration controls.

Limits, which include considerable safety margins, 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 (RCPB), or
- b) Disrupt the core, its support structures, or other vessel internals so as to lose capability to cool the core.

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3.1.1.2.6 Reactor Coolant Pressure Boundary (GDC 33-GDC 36)

The RCPB is shown to be capable of accommodating without rupture, the static and dynamic loads imposed as a result of a sudden reactivity insertion such as a rod ejection.

The RCPB is designed to reduce to an acceptable level the probability of a rapidly propagating type failure.

The design of the reactor vessel and its arrangement in the system permits accessibility during the service life to the entire internal surfaces of the vessel and to the following external zones of the vessel:

- a) The flange seal surface
- b) The flange outside diameter (OD) down to the Permanent Cavity Seal Plate (PCSP)
- c) The closure head except around the drive mechanism adapters, and
- d) The nozzle to reactor coolant piping welds.

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.

To define permissible operating conditions below the Design Transition Temperature (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. Since the normal operating temperature of the reactor vessel is well above the maximum expected DTT, brittle fracture during normal operation is not considered to be credible.

3.1.1.2.7 Engineered Safety Features (GDC 37-GDC 65)

The design, fabrication, testing, and inspection of the core, RCPB, 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 space 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 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 total loss of all outside power is assumed concurrent with these accidents.

The ESF consists of the Emergency Core Cooling System (ECCS) with accumulators, high and low pressure Safety Injection Systems (SIS), and the Residual Heat Removal System (RHRS), the containment with its isolation system, the Containment Air Recirculation System and the Containment Spray System.

The release of fission products from the reactor fuel is limited by the SIS which, by cooling the core and limiting the fuel clad temperature, keeps the fuel in place and substantially intact with its heat transfer geometry preserved and limits the metal-water reactor to an insignificant amount (less than one percent of the total clad reacted).

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For any rupture of a steam pipe and the associated uncontrolled heat removal from the core, the ECCS adds shutdown reactivity so that with a stuck rod, no offsite power, and minimum engineered safety features, there is no consequential damage to the primary system and the core remains in place and intact. With no stuck rod, no offsite power, and all equipment operating at design capacity, there is insignificant cladding rupture.

The ESF systems and components are designed with the following criteria:

1. Redundancy and separation
2. Testability of performance and operation
3. Capability of performance to withstand
 - a. Any single failure of active component
 - b. Missiles by using barriers and plant layout
 - c. Environmental conditions caused by a LOCA
4. Normal, standby, and emergency power sources
5. Capability to inject borated water (negative reactivity) and not cause further loss of integrity of the RCS, and
6. Ability to be inspected.

The containment was tested initially and at intervals to assure its ability to contain a design basis LOCA.

3.1.1.2.8 Fuel and waste storage systems (GDC 66-GDC 69)

The new and spent fuel storage racks are designed so that it is impossible to insert assemblies in other than fuel cell locations. Borated water with a concentration such that the core is maintained with $\geq 6\%$ $\Delta k/k$ shutdown margin is used during refueling.

The design of the fuel handling equipment, incorporated built-in interlocks and safety features, the use of detailed refueling instructions, and observance of minimum operating conditions provide assurance that no incident could occur during the refueling operations that would result in a hazard to public health and safety.

The refueling water provides a reliable and adequate cooling medium for spent fuel transfer. Heat removal is accomplished with a cooling system with redundant 100 percent capacity pumps. Connections are provided to the component cooling water system for alternative cooling.

Adequate shielding for radiation protection is provided during reactor refueling by conducting all spent fuel transfer and storage operations under water.

Auxiliary shielding for the WDS and its storage components is designed to limit the dose rate to levels not exceeding 1 mR/hr in normally occupied areas, to levels not exceeding 2.5 mR/hr in intermittently occupied areas, and to levels not exceeding 15 mR/hr in controlled occupancy areas.

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All waste handling and storage facilities are contained and equipment designed so that accidental releases directly to the atmosphere are monitored and will not exceed the guidelines of 10CFR100.

3.1.1.2.9 Plant Effluents (GDC 70)

Liquid, gaseous, and solid waste disposal facilities are designed so that discharge of effluents and offsite shipments are in accordance with applicable governmental regulations.

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3.1.2 EVALUATION PER GENERAL DESIGN CRITERIA

In addition to the summary material given in Section 3.1.1.2, detailed descriptions of how HBR met the proposed General Design Criteria are given below.

3.1.2.1 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. (GDC 1)

Response:

The Reactor Coolant System (RCS) 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 3.2). Details of the quality assurance programs, test procedures, and inspection acceptance levels are given in Section 5.3. Particular emphasis is 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 ASME Section III.

The containment system structure 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 governing the above features conform to the applicable provisions of recognized codes and good nuclear practice. The concrete structure of the reactor containment conforms to the applicable portions of American Concrete Institute (ACI) ACI-318-63. Further elaboration on quality standards of the reactor containment is given in Section 6.2.

The containment system is designed to limit the effects of any credible accident in the reactor on the RCS to an area immediately surrounding the reactor area. The system consists of the following components and subsystems.

- a) Containment structure (Section 6.2)
- b) Containment Isolation System (CIS) (Sections 6.2 and 6.9)

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- c) Safety Injection System (SIS) (Section 6.3)
- d) Containment Spray System (CSS) (Section 6.2.2), and
- e) Containment Air Recirculation Cooling System (Section 6.2.2).

3.1.2.2 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. (GDC 2)

Response:

All piping, components, and supporting structures of the RCS are designed as Class I equipment, i.e., they are capable of withstanding:

- a) The design seismic ground acceleration within code allowable working stresses (Section 3.7), and
- b) The maximum potential seismic ground acceleration acting in the horizontal and vertical direction simultaneously with no loss of function (Section 3.7).

The RCS is located in the Containment Building whose design, in addition to being a Class I structure, also considers accidents or other applicable natural phenomena. Details of the containment design are given in Section 6.2.

All components and supporting structures of the reactor containment are designed so that there will be no loss of function of such equipment in the event of maximum conceivable ground acceleration acting in the horizontal and vertical directions simultaneously. The dynamic response of the structure to ground acceleration, based on appropriate spectral characteristics of the site foundation and on the damping of the foundation and structure, is included in the report on Structural Dynamic Analysis for Seismic Analysis of Class I Structures (Section 3.7).

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The reactor containment is defined as a Class I structure for purposes of seismic design (Section 3.8). Its structural members have sufficient capacity to accept without exceeding yield stresses a combination of normal operating loads and tornado; or functional loads due to a LOCA and the loadings imposed by the maximum wind velocity; or those due to design earthquake, whichever is the larger.

All electrical systems and components vital to plant safety, including the emergency diesel generators (DG) are designed as Class I and designed or arranged so that their integrity is not impaired by the maximum hypothetical earthquake, wind storms, floods, tornado winds, or disturbances on the external electrical system. Power wires, control cabling, instrument cabling, motors, and other electrical equipment required for operation of the engineered safety features (ESF) are suitably protected against the effects of either a nuclear system accident or of severe external environmental phenomena in order to assure a high degree of confidence in the operability of such components in the event that their use is required.

3.1.2.3 Fire Protection

Criterion: The facility is designed so that the probability of fires and explosions and the potential consequences of such events do 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 ESF. (GDC 3)

Response:

Fire prevention in all areas of the nuclear unit 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 capable of suppressing anticipated fires.

This plant was designed on the basis of limiting the use of combustible materials in construction and of using fire-resistant materials to the greatest extent possible. Also, fire barrier seals (electrical seals, fire doors, and fire dampers) are used to prevent the spread of fires.

The fire protection system has the design capability to extinguish or control probable combinations of fires which might occur at the station.

The reactor containment system is designed to maintain the capability in case of fire to safely shut down the reactor and isolate the containment.

The Containment and Auxiliary Building Ventilation Systems are operable from the Control Room. Smoke or heat detectors and Control Room alarms are provided for key ventilation systems (Section 9.5.1).

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Fire detection systems exist in the Containment and Auxiliary Buildings. Some of these detection systems also actuate fire suppression systems.

Containment liner thermal insulation and insulation adhesives do not support combustion.

3.1.2.4 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.
(GDC 4)

Response:

The residual heat removal (RHR) pumps and heat exchangers serve dual functions. Although the normal duty of the RHR exchangers and RHR pumps is performed during periods of reactor shutdown, during all plant operating periods this equipment is aligned to perform the low head safety injection (SI) function. In addition during the recirculation phase of a LOCA, the system may be used for the core cooling and the containment spray cooling functions. Demonstration testing of the system, performed during each refueling period before plant startup, provides assurance of correct system alignment for the safety function of components.

During the injection phase, the SI pumps do not depend on any portion of other systems. During the recirculation phase, if RCS pressure stays high due to a small break accident, suction to the SI pumps is provided by the RHR pumps.

The Containment Air Recirculation System also serves the dual function of containment cooling during normal operation and containment cooling after an accident. Since the method of operation for both cooling functions is the same, the dual aspect of this system does not affect its function as an ESF.

3.1.2.5 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. (GDC 5)

Response:

Records of the design, of the major RCS components, and the related ESF components are maintained in the offices of Duke Energy Progress, LLC (DEP) and will be retained there throughout the life of the plant.

Records of fabrication are maintained in the manufacturers' plants as required by the appropriate code, or other requirements pending submittal to Westinghouse or DEP. They are available at any time to DEP throughout the life of the plant. Construction records are available at the construction site and in the offices of DEP where they will be retained for the life of the plant.

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Records of the design, fabrication, construction, and testing of the reactor containment will be maintained throughout the life of the reactor.

3.1.2.6 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 normal operation with appropriate margins for uncertainties and for specified transient situations which can be anticipated. (GDC 6)

Response:

The reactor core, with its related control and protection system, was 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 normal operation with appropriate margins for uncertainties and anticipated transient situations, including the effects of the loss of reactor coolant flow, trip of the turbine generator, loss of normal feedwater, and loss of all offsite power.

The Reactor Control and Protection System is designed to actuate a reactor trip for any anticipated combination of plant conditions, when necessary, to ensure a minimum Departure from Nucleate Boiling (DNB) ratio equal to or greater than the safety limit specified in Section 4.4.

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 normal operation or any anticipated transient condition:

- a) Minimum DNB ratio equal to or greater than the safety limit specified in Section 4.4
- b) Fuel center temperature below melting point of UO_2
- c) Internal gas pressure less than the nominal external pressure (2250 psia) even at the end of life
- d) Clad stresses less than the Zircaloy yield strength
- e) Clad strain less than 1 percent, and
- f) Cumulative strain fatigue cycles less than 80 percent of design strain fatigue life.

The ability of fuel designed and operated to these criteria to withstand postulated normal and abnormal service conditions is shown by analyses to satisfy the demands of plant operation well within applicable regulatory limits (Chapter 15.0).

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The reactor coolant pumps (RCP) 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 to ensure that the reduction in heat flux obtained with a low flow reactor trip prevents core damage. An inadvertent actuation of the shutdown seal (SDS) on the RCP shaft, with the shaft still rotating, will not adversely impact RCP coastdown. In the event of a loss of all seal cooling, the SDS will actuate on high seal cooling temperature to limit leakage from the RCP seal package. Leakage is limited when a thermal actuator retracts and causes the SDS piston ring and polymer ring to clamp down around the pump shaft. In the unlikely event of a turbine trip from full power without an immediate reactor trip, the subsequent reactor coolant temperature increase and volume surge to the pressurizer results in a high pressurizer pressure trip and thereby prevents fuel damage for this transient. A loss of external electrical load of 100 percent 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 an unacceptable temperature and pressure increase in the RCS. In this case, the overpower-temperature protection would guard against any combination of pressure, temperature, and power which could result in a DNB ratio less than the safety limit during the transient.

In neither the turbine trip nor the loss-of-flow events do the changes in coolant conditions provoke a 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.

3.1.2.7 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. (GDC 7)

Response:

The potential for axial oscillations of power distributions has been shown to exist for cores the size of HBR 2 when power changes occur. For xenon oscillations caused by changes in power level and/or rod movements, plant procedures exist which provide for operator initiated damping of the oscillations by control rod movement. Adverse axial power distributions caused by xenon shifts which result from routine load changes during power operation are controlled using Power Distribution Control 3 (PDC-3) procedures. The PDC-3 procedure limits the peaking factor to the Technical Specification limit by restricting xenon redistribution during power changes. This is done by monitoring the power difference between the top and bottom of the core as a function of different power levels and core conditions.

Both incore and out-of-core instrumentation are provided to obtain necessary information concerning power distribution. This instrumentation is adequate to enable the operator to monitor and control xenon induced oscillations. (Incore instrumentation is used to periodically calibrate and verify the information provided by the out-of-core instrumentation.)

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3.1.2.8 Overall Power Coefficient (GDC 8)

Refer to Section 3.1.1.2.2 and 4.3.2.3

3.1.2.9 Reactor Coolant Pressure Boundary

Criterion: The reactor coolant pressure boundary (RCPB) 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. (GDC 9)

Response:

The RCS in conjunction with its control and protective provisions was 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 (see Section 3.2.)

Fabrication of the components which constitute the pressure boundary of the RCS is carried out in strict accordance with the applicable codes. In addition, there are areas where equipment specifications for RCS components go beyond the applicable codes.

The materials of construction of the pressure boundary of the RCS 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 to a safe level.

The system is protected from overpressure by means of pressure relieving devices, as required by Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code.

Isolatable 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.

3.1.2.10 Reactor Containment

Criterion: 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 ESF 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. (GDC 10)

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Response:

The reactor containment structure is a reinforced concrete vertical cylinder with pre-stressed steel tendons in the vertical wall, a reinforced concrete hemispherical dome and supported on soil supported steel pipe friction piles.

The design pressure and temperature of the containment exceeds the peak pressure and temperature occurring as the result of the complete blowdown of the reactor coolant through any rupture of the RCS up to and including the hypothetical severance of a reactor coolant pipe.

The containment structure and all penetrations are designed to withstand within design limits the combined loadings of the design basis accident (DBA) and maximum seismic conditions.

No piping systems which penetrate the vapor barrier are anchored to the liner. The penetration for the main steam, feedwater, blowdown, and sample lines are designed so that the penetration is stronger than the piping system and that the vapor barrier will not be breeched due to a hypothesized pipe rupture combined, for the case of the steam line, with the coincident internal pressure. For the penetration design, the pipe capacity in flexure is assumed to be limited to the plastic moment capacity based upon the ultimate strength of the pipe material. All lines connected to the primary coolant system that penetrate the vapor barrier are also anchored at the secondary shield walls (i.e., walls surrounding the steam generators and RCP) 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 hypothesized rupture of the attached pipe or the loads induced by the hypothetical (larger) earthquake.

All isolation valves are supported to withstand, without impairment of valve operability, the loadings of the DBA or maximum hypothetical seismic conditions.

The design pressure will not be exceeded during any subsequent long-term pressure transient determined by the combined effects of heat sources such as residual heat and metal-water reactions with minimum operation of the emergency core cooling and the Containment Air Recirculation Cooling Spray Cooling Systems.

3.1.2.11 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 post-accident 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. (GDC 11)

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Response:

The plant is equipped with a Control Room which contains those controls and instrumentation necessary for operation of the reactor and turbine generator under normal and accident conditions.

The Control Room was designed for continuous occupancy by the operating personnel under all operating and accident conditions. If a fire or other condition causes the Control Room to be inaccessible, the reactor can be shut down from outside the Control Room (Section 7.4). Also, if a fire should cause principal equipment required for shutdown to be inoperable, a dedicated shutdown system exists which would bring the plant to a safe condition (Section 7.4).

Sufficient shielding, distance, and containment integrity are provided to ensure that calculated Control Room personnel doses are within the limits described in Section 6.4.

The Control Room air conditioning consists of a system having a large percentage of recirculated air. During a design basis accident, the system is configured to pressurize the Control Room with clean filtered air to control the intake of airborne activity.

3.1.2.12 Instrumentation and Control Systems

Criterion: Instrumentation and controls shall be provided as required to monitor and maintain within prescribed operating ranges for the essential reactor facility operating variables. (GDC 12)

Response:

Instrumentation and controls are provided to monitor and maintain all operationally important reactor parameters (including neutron flux, primary coolant pressure, flow rate, temperature, and control rod positions) within prescribed operating ranges. Other instrumentation and control systems are provided to monitor and maintain within prescribed operating ranges the temperatures, pressure, flow, and levels in the RCS, Steam Systems, Containment, and other Auxiliary Systems. Process variables which are required on a continuous basis for the startup, power operation, and shutdown of the plant are indicated, recorded, and controlled from the Control Room which is a controlled access area. The quantity and types of instrumentation provided are adequate for safe and orderly operation of all systems and processes over the full operating range of the plant.

3.1.2.13 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. (GDC 13)

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Response:

Nuclear Instrumentation is utilized primarily for reactor protection by permitting monitoring of neutron flux and by generating appropriate trip and alarm functions for various phases of reactor operating and shutdown conditions. It also provides a secondary control function and indicates reactor status during startup and power operation. The Nuclear Instrumentation System (NIS) utilizes information from the three separate types of instrumentation channels to provide three discrete protection levels. Each range of instrumentation (source, intermediate, and power) provides the necessary overpower reactor trip protection required during operation in that range.

The overlap of instrument ranges provides reliable continuous protection from source to intermediate and low power ranges. As the reactor power increases, the overpower protection level is increased administratively after satisfactory higher range instrumentation operation is obtained. Automatic reset to more restrictive trip protection is provided when reducing power.

Various types of neutron detectors, with appropriate solid state electronic circuitry, are used to monitor the leakage neutron flux from a completely shut down condition to 120 percent of full power. The power range channels are capable of recording overpower excursions up to 200 percent of full power.

The neutron flux covers a wide range between these extremes. Therefore, monitoring with several ranges of instrumentation is necessary. The lowest range (source range) covers six decades of leakage neutron flux. The lowest observed count rate depends on the strength of the neutron sources in the core and the core multiplication associated with the shutdown reactivity. This is generally greater than one count per second. The next range (intermediate range) covers eight decades. Detectors and instrumentation are chosen to provide overlap between the higher portion of the source range and the lower portion of the intermediate range.

The highest range of instrumentation (power range) covers slightly more than two decades of the total instrumentation range. This is a linear range that overlaps with the higher portion of the intermediate range. Startup-rate indication for the source and intermediate range channels is provided at the control console.

The system described above provides Control Room indication and recording of reactor neutron flux during core-loading, shutdown, startup, and power operation as well as during subsequent refueling. Reactor trip, and rod-stop control, and alarm signals are provided by this system for safe plant operation. Control and permissive signals are transmitted to the Reactor Control and Protection System for automatic plant control. Equipment failures and test status information are annunciated in the Control Room. Source range instrumentation outputs are also displayed at the charging pump room panel.

3.1.2.14 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. (GDC 14)

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Response:

If the reactor protection system (RPS) receives signals which are indicative of an approach to unsafe operating conditions, the system actuates alarms, prevents control rod motion, initiates load cutback, and/or opens the reactor trip breakers.

The basic reactor tripping philosophy is to define an allowable region of power and coolant temperature conditions. This allowable range is defined by the primary tripping functions, the overpower high ΔT trip, the over-temperature high ΔT trip, and the nuclear overpower trip. The operating region below these trip settings was designed so that no combination of power, temperatures, and pressure could result in a departure from nucleate boiling ratio less than the safety limit with all RCP in operation. Additional tripping functions such as a high pressurizer pressure trip, low pressurizer pressure trip, high pressurizer water level trip, loss of flow trip, steam and feedwater flow mismatch trip¹, steam generator low-low water level trip, turbine trip, SI trip, nuclear source and intermediate range trips, and manual trip are provided to back up the primary tripping functions for specific accident condition and mechanical failures.

Automatic rod cluster control assemblies (RCCA) withdrawal has been disabled. A dropped RCCA is indicated from individual RCCA position indicators and by a rapid flux decrease on any of the power range nuclear channels. This may cause a decrease in temperature, but there will be no automatic withdrawal of RCCA's.

Rod stops from nuclear overpower, overpower ΔT , and over-temperature ΔT deviation are provided to prevent abnormal power conditions which could result from excessive control rod withdrawal initiated by a malfunction of the reactor control system or by operator violation of administrative procedures.

3.1.2.15 ESF Protection Systems

Criterion: Protection systems shall be provided for sensing accident situations and initiating the operation of necessary ESF. (GDC 15)

Response:

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, and to limit energy release; to signal containment isolation; and to control the operation of ESF equipment.

The ESF systems are actuated by the ESF actuation channels. Each coincident network energizes an ESF actuation device that operates the associated ESF equipment, motor starters, and valve operators. The channels are designed to combine redundant sensors and independent channel circuitry, coincident trip logic, and different parameter measurements so that a safe and reliable system is provided in which a single failure will not defeat the channel function. The action initiating sensors, bistables, and logic are shown in the figures included in the detailed ESF instrumentation description (Section 7.3). The ESF instrumentation system actuates (depending on the severity of the condition) the SIS, containment isolation, the Containment Air Recirculation Cooling System, and the CSS.

¹ The steam and feedwater flow mismatch trip was deleted by License Amendment No. 234

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Availability of control power to the ESF trip channels is continuously indicated. The loss of instrument power to the sensors, instruments, or logic devices in the ESF instrumentation, places that channel in the trip mode, except for containment spray initiating channels which require instrument power for actuation.

The passive accumulators of the SIS do not require signal or power sources to perform their function. The actuation of the active portion of the SIS is obtained from redundant low pressurizer pressure.

The containment air recirculation coolers are normally in use during plant operation. These units are in the automatic sequence which actuates the ESF upon receiving the necessary signals indicating an accident condition.

Containment spray is actuated by coincident and redundant high containment pressure signals. Chemical addition to containment spray is automatic.

The containment isolation signals provide the means of isolating the various pipes passing through the containment walls as required to prevent the release of radioactivity to the outside environment in the event of a LOCA. The actuation of the containment isolation is by any SI signal, by a containment high-high pressure signal, by a high containment activity signal (isolates purge and vacuum and pressure relief valves only), or manually.

The ESF actuation circuits are designed on the same "de-energize to operate" principle as the reactor trip circuits with the exception of the containment spray actuation circuit which is energized to operate in order to avoid spray operation on inadvertent power failure.

3.1.2.16 Monitoring Reactor Coolant Leakage

Criterion: Means shall be provided to detect significant uncontrolled leakage from the reactor coolant pressure boundary. (GDC 16)

Response:

Positive indications in the Control Room of leakage of coolant from the RCS to the containment are provided by equipment which permits continuous monitoring of containment air activity. The containment humidity and runoff from the condensate collecting pans under the cooling coils of the containment air recirculation units are displayed at the Waste Disposal Panel. 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, and condensate runoff. In addition, the liquid inventory in the process systems and containment sump is monitored to detect any gross leakage.

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3.1.2.17 Monitoring Radioactivity Releases

Criterion: Means shall be provided for monitoring the containment atmosphere and the facility effluent discharge paths for radioactivity released from normal operations, 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. (GDC 17)

Response:

The containment atmosphere, the ventilation exhausts from the RHR pumps compartments, the plant vent, the containment fan-coolers service water discharge, the component cooling loop liquid, the condenser vacuum pump exhaust, the steam generators blowdown effluents, the waste disposal system liquid effluents, and the fuel handling building lower level exhaust are monitored to support normal operation. In addition to the sources monitored to support normal operation, the fuel handling building upper level exhaust is monitored to support transient conditions. To support accident conditions, the main steam lines are monitored plus the sources that are monitored for normal operation and transient conditions. High radiation activity from any of these sources is indicated and alarmed in the Control Room.

All accidental spills of liquids are contained within the Reactor Auxiliary Building, Radwaste Building or E&RC Building and collected in the building sumps for processing. Any contaminated liquid effluent released to the condenser circulating water canal is monitored. For the case of leakage from the reactor containment under accident conditions, the plant area radiation monitoring system, supplemented by portable survey equipment kept in the Control Room, provides adequate monitoring of releases during an accident. An outline of the equipment to be used in the event of an accident are discussed in Chapter 11.0. The effluent monitoring program is described in Section 11.5.

3.1.2.18 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. (GDC 18)

Response:

Monitoring and alarm instrumentation are 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 of radioactive gases and liquids.

The spent fuel pit cooling loop is flow monitored to assure proper operation.

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A ventilation system removes gaseous radioactivity from the atmosphere of the fuel handling building and discharges it to the atmosphere through roughing and high-efficiency particulate air filters (HEPA). An area radiation monitor is in continuous service in this area and it actuates a high-activity alarm on the control board annunciator.

3.1.2.19 Protection Systems Reliability

Criterion: Protection system shall be designed for high functional reliability and in-service testability necessary to avoid undue risk to the health and safety of the public.
(GDC 19)

Response:

The reactor uses a higher speed version of the Westinghouse magnetic-type control rod drive mechanisms (CRDM) used in the San Onofre, and Connecticut Yankee plants. Upon a loss of power to the coils, the full length RCCA are released and fall by gravity into the core.

The reactor internals, fuel assemblies, RCCA, and drive system components are designed as Class I equipment. The RCCA are fully guided through the fuel assembly and for the maximum travel of the control rod into the guide tube.

The alignment of the mating sections in each guide tube assembly is maintained by dowel pins to assure free rod movement under both normal operating and credible accident conditions. The alignment between the guide tube assemblies and the guide thimbles in the fuel assemblies is maintained by the guide pin in the upper core plate which mates with the fuel assembly top nozzle. As a further safeguard against "hang up" of the RCCA, the length and travel of the absorber rods are set to prevent them from being totally withdrawn from the fuel assembly guide thimbles during operation.

As a result of these design safeguards and the flexibility designed into the RCCA, abnormal loadings and misalignments can be sustained without impairing operation of the RCCA.

An analogous system has successfully undergone 4132 hr of testing in the Westinghouse Reactor Evaluation Center during which about 27,200 ft of step-driven travel and 1461 trips were accomplished with test misalignments in excess of the maximum possible misalignment that may be experienced when installed in the plant.

Protection channels are designed with sufficient redundancy for individual channel calibration and test to be made during power operation without degrading the reactor protection. Bypass removal of one trip circuit is accomplished by placing that channel in a partial-tripped mode; i.e., two-out-of-three channel becomes a one-out-of-two channel. Testing will not cause a trip unless a trip condition exists in a concurrent channel.

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Reliability and independence is obtained by redundancy within each tripping function. In a two-out-of-three logic, for example, the three channels are equipped with separate primary sensors. Each channel has its own independent electrical source. Failure on the part of one channel to de-energize when required would affect only that particular channel. The trip signal furnished by the two remaining channels would be unimpaired in this event.

3.1.2.20 Protection Systems Redundancy and Independence

Criterion: Redundancy and independence designed into protection systems shall be sufficient to assure that no single failure on 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. (GDC 20)

Response:

The RPS is designed so that the most probable modes of failure in each channel result in a signal calling for the protective trip. The protection system design combines redundant sensors and channel independence with coincident trip philosophy so that a safe and reliable system is provided in which a single failure will not defeat the channel function, cause a spurious plant trip, or violate reactor protection criteria.

Channel independence is carried throughout the system extending from the sensor to the relay actuating the protective function. When the protective and control functions are combined, it is done only at the sensor. The protective and control functions are fully isolated in the remaining part of the channel, control being derived from the primary protection signal path through an isolation amplifier. Therefore, failure in the control circuit does not affect the protection channel.

The ESF equipment is actuated by one of the redundant ESF channels. Each coincident network energizes an engineered safety actuation device that operates the associated ESF equipment motor starters and valve operators. As an example, the control circuit for a SI pump is typical of the control circuit for a large pump operated from switchgear. The actuation relay, energized by the ESF Instrumentation System, has normally open contacts. These contacts energize the circuit breaker closing coil to start the pump when the control relay is energized. The ESF Instrumentation System actuates (depending on the severity of the condition) the SIS, containment isolation, and the Containment Air Recirculation Cooling System.

In the RPS, two reactor trip breakers are provided to interrupt power to the RCCA drive mechanisms. The breaker main contacts are connected in series (with the power supply) so that opening either breaker interrupts power to all full length RCCA, permitting them to fall by gravity into core.

Further detail on redundancy is provided through the descriptions of the respective systems covered by the various subsections in this section. Required continuous power supply for the protection systems is discussed in Section 8.3.1.

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In summary, reactor protection is designed to meet all presently defined reactor protection criteria and is in accordance with the proposed Institute of Electrical and Electronic Engineers (IEEE) 279 "Standard for Nuclear Plant Protection Systems" August, 1968.

3.1.2.21 Single Failure Definition

Refer to Sections 3.1.2.20, 3.1.2.31, and 7.2

3.1.2.22 Separation of Protection and Control Instrumentation

The physical arrangement of the redundant elements of the protection system are such that the probability is reduced that a single physical event will impair the vital function of the system (Section 3.1.2.23).

3.1.2.23 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. (GDC 23)

Response:

The components of the protection system are designed and laid out 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 physical arrangement of all elements associated with the protective system reduces the probability of a single physical event impairing the vital functions of the system.

Isolation of redundant analog channels originates at the process sensors and continues along the field wiring and through containment penetrations to the analog protection racks. Physical separation is used to the maximum practical extent to achieve isolation of redundant transmitters. Isolation of field wiring is achieved using separate wireways, cable trays, conduit runs, and containment penetrations for each redundant channel. Analog equipment is isolated by locating redundant components in different protection racks. Each channel is energized from a separate AC instrument bus.

System equipment is separated between instrument cabinets so as to reduce the probability of damage to the total system by some single event.

Wiring between vital elements of the system outside of equipment housing is routed and protected so as to maintain the true redundancy of the systems with respect to physical hazards.

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3.1.2.24 Emergency Power for Protection Systems

Redundancy in emergency power is provided in that there are two DG sets capable of supplying separate 480 volt buses. One complete set of safety features equipment is therefore independently supplied from each DG. A third dedicated shutdown diesel is provided for redundant power for loads required for safe shutdown of the reactor in the event of a fire (10CFR50, Appendix R) or Station Blackout (10CFR50.63).

Diesel engine cranking is accomplished by a stored energy system supplied solely for the associated DG. The stored energy (air start) systems for each DG are cross connected such that either system will start either DG. The cross connection is normally isolated. The undervoltage relay scheme is designed so that loss of 480 volt power does not prevent the relay scheme from functioning properly.

The ability of the DG sets to start within the prescribed time and to carry load can periodically be checked. The DG breaker is not closed automatically after starting during this testing. The generator may be manually synchronized to the 480 volt bus for loading. Blocking the closure of the DG breaker causes the bus tie breaker to close after a time delay (10 sec) sufficient for normal DG starting.

Emergency power is discussed in Section 8.3.1.

3.1.2.25 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. (GDC 25)

Response:

Each protection channel in service at power is capable of being calibrated and tripped independently by simulated signals to verify its operation without tripping the plant. The testing scheme includes checking through the trip logic to the trip breakers. Thus, the operability of each trip channel can be determined conveniently and without ambiguity.

3.1.2.26 Protection System 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 system, loss of energy (e.g., electrical power, instrument air), or adverse environments (e.g., extreme heat or cold, fire, steam, or water) are experienced. (GDC 26)

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Response:

Each reactor trip circuit is designed so that trip occurs when the circuit is de-energized; 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 de-energize when required is, therefore, a mode malfunction that can affect only one channel. The trip signal furnished by the two remaining channels is unimpaired in this event.

Reactor trip is implemented by simultaneously interrupting power to the magnetic latch mechanisms on each drive allowing the full length rod clusters to insert by gravity. The entire protection system is thus inherently safe in the event of a loss of power. This equipment is selected to withstand the most adverse environmental conditions to which it will be subjected including post-accident conditions within the containment.

The ESF actuation circuits are designed on the same "de-energized to operate" principle as the reactor trip circuits with the exception of the containment spray actuation circuit which is energized to operate in order to avoid spray operation on inadvertent power failure.

Certain fires may cause multiple failures which could prevent reactor shutdown; therefore, a dedicated shutdown system was installed to bring the plant to a safe shutdown condition.

3.1.2.27 Redundancy of Reactivity Control

Criterion: Two independent control systems, preferably of different principles, shall be provided. (GDC 27)

Response:

One of the two reactivity control systems employs rod cluster control assemblies to regulate the position of the neutron absorbers within the reactor core. The other reactivity control system employs the Chemical and Volume Control System to regulate the concentration of boric acid solution neutron absorber in the Reactor Coolant System. The system is designed to prevent uncontrolled or inadvertent reactivity changes which might cause system parameters to exceed design limits.

3.1.2.28 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. (GDC 28)

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Response:

The reactivity control systems provided are capable of making and holding the core subcritical from any hot standby or hot operating 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 life of the initial core.

The RCCA are divided into two categories comprising control and shutdown rod groups. The control banks 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 RCCA 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.

The reactivity control systems provided are capable of making and holding the core subcritical from any hot standby or hot operating condition, including those resulting from power changes.

3.1.2.29 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. (GDC 29)

Response:

The reactor core, together with the Reactor Control and Protection System is designed so that the minimum allowable DNBR is equal to or greater than the DNB limit established for the applicable DNB correlation as specified in UFSAR Section 4.4.2 and there is no fuel melting during normal operation including anticipated transients.

The shutdown groups are provided to supplement the control group of RCCA to make the reactor at least one percent subcritical ($K_{\text{eff}} = 0.99$) following a trip from any credible operating condition to the hot, zero power condition assuming the most reactive RCCA remains in the fully withdrawn position.

Sufficient shutdown capability is also provided to avoid fuel damage during the most severe anticipated cooldown transient associated with a single active failure, e.g., accidental opening of a steam bypass, or relief valve. This is achieved with combination of control rods and automatic boron addition via the emergency core cooling system with the most reactive rod assumed to be fully withdrawn. Manually controlled boric acid addition is used to maintain the shutdown margin for the long term conditions of xenon decay and plant cooldown.

3.1.2.30 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. (GDC 30)

Response:

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 2 sec following a trip signal 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 reactor is at power, the quantity of boric acid retained in the boric acid tanks and ready for injection always exceeds that quantity required for the normal cold shutdown. This quantity always exceeds the quantity of boric acid required to bring the reactor to hot shutdown and to compensate for subsequent xenon decay.

Boric acid is pumped from the boric acid tanks by one of two boric acid transfer pumps to the suction of one of three charging pumps which inject 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 offsite power. Boric acid can be injected by one pump at a rate which takes the reactor to hot zero power with no rods inserted in less than forty-five minutes. This provides the capability to shutdown the reactor in the event of an Anticipated Transient Without Scram. In forty-five additional minutes, enough boric acid can be injected to compensate for xenon decay although xenon decay below the equilibrium operating level does not begin until approximately 15 hr after shutdown. If two boric acid pumps are available, these time periods are reduced. Additional boric acid injection is employed if it is desired to bring the reactor to 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, thus achieving the measure of reliability implied by the criterion.

Alternately, boric acid solution at lower concentration can be supplied from the refueling water tank. This solution can be transferred directly by the charging pumps or alternately by the SI pumps. The reduced boric acid concentration lengthens the time required to achieve equivalent shutdown.

3.1.2.31 Reactivity Control Systems Malfunction

Criterion: The reactor protection systems 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. (GDC 31)

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Response:

The RPS is designed to limit reactivity transients to DNBR □ the safety limit (specified in Section 4.4) due to any single malfunction in the deboration controls.

The RPS 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 and continuous deboration are described in Sections 15.4 and 9.3.4, respectively.

3.1.2.32 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 RCPB, and
- b) Disrupt the core, its support structures, or other vessel internals sufficiently to lose capability of cooling the core. (GDC 32)

Response:

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, and
- 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, approximately 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.

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The maximum insertion rate is analyzed in the detailed plant analysis assuming two of the highest worth groups to be accidentally withdrawn at maximum speed, yielding reactivity insertion rates of the order of $8 \times 10^{-4} \Delta k/\text{sec}$. This value 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 rod cluster to be withdrawn at a speed greater than 77 steps per minute ($\square 48$ in. per minute).

3.1.2.33 RCPB Capability

Criterion: The RCPB shall be capable of accommodating without rupture the static and dynamic load 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. (GDC 33)

Response:

The reactor coolant boundary is shown to be capable of accommodating without 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.

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 boron dilution is used to compensate for core depletion, only the RCCA 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, from possible excessive pressure surges.

The failure of a rod mechanism housing resulting in a rod cluster being 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 RCS and the reactor containment. The environmental consequences of rod ejection are less severe than from the hypothetical LOCA, for which public health and safety is shown to be adequately protected.

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3.1.2.34 RCPB Rapid Propagation Failure Prevention

Criterion: The RCPB shall be designed and operated to reduce to an acceptable level the probability of 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 (RPV) 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
- c) To the design and construction of RCPB piping and equipment in accordance with applicable codes.(GDC 34)

Response:

The RCPB 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 exposure to fast neutrons. 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 reached 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 temperature, and as confirmed by the experimental data obtained from irradiated specimens of reactor vessel materials during the plant lifetime.

All pressure-containing components of the RCS are designed, fabricated, inspected, and tested in conformance with the applicable codes (Section 3.2).

3.1.2.35 RCPB Brittle Fracture Prevention

Refer to Sections 3.1.2.34, 3.1.2.36, and 5.3

3.1.2.36 RCPB Surveillance

Criterion: RCPB components shall have provisions for inspection, testing, and surveillance of criteria 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. (GDC 36)

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Response:

The design of the reactor vessel and its arrangement in the system permits access during the service life to the entire internal surfaces of the vessel and to the following external zones of the vessel:

- a) The flange seal surface
- b) The flange outside diameter (OD) down to the Permanent Cavity Seal Plate (PCSP)
- c) The closure head except around the drive mechanism adapters, and
- d) The nozzle to reactor coolant piping welds.

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 plates forgings, weldments, and associated heat treated zones is performed in accordance with American Society for Testing and Materials (ASTM) E185 Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors (the revision in effect when the vessel was designed). Samples of reactor vessel plate materials were retained and catalogued 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 tests. 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 on startup and shutdown 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. Since the normal operating temperature of the reactor vessel is well above the maximum expected DTT, brittle fracture during normal operation is not considered to be credible.

3.1.2.37 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 RCPB, 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. (GDC 37)

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Response:

The design, fabrication, testing and inspection of the core, RCPB 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 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 SIS which, by cooling the core, keeps the fuel in place and substantially intact and limits the metal water reaction to an insignificant amount (\square one percent).

The SIS consists of high and low head centrifugal pumps driven by electric motors, and passive accumulator tanks which are self actuated and act independently of any actuation signal or power source.

The release of fission products from the containment is limited in three ways:

- a) Blocking the potential leakage paths from the containment. This is accomplished by:
 - 1) A steel-lined, concrete reactor containment with testable penetrations and liner weld channels
 - 2) Isolation of process lines by the Containment Isolation System (CIS) which imposes double barriers in each line which penetrates the containment
 - 3) An Isolation Valve Seal Water System which creates a leaktight seal between the valves in each line which penetrates the containment.
- b) Reducing the fission product concentration in the containment atmosphere by spraying chemically treated borated water which removes airborne elemental iodine vapor by washing action.
- c) Reducing the containment pressure and thereby limiting the driving potential for fission product leakage by cooling the containment atmosphere using the following independent systems of essentially equal heat removal capacity:
 - 1) Containment Spray System
 - 2) Containment Air Recirculation Cooling System

3.1.2.38 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. (GDC 38)

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Response:

A comprehensive program of plant testing is formulated for all equipment systems and system control vital to the functioning of engineered safety features. The program consists of performance tests of individual pieces of equipment in the manufacturer's shop, integrated tests of the system as a whole, and periodic tests of the actuation circuitry and mechanical components to assure reliable performance, upon demand, throughout the plant lifetime.

The initial tests of individual components and the integrated test of the system as a whole, complement each other to assure performance of the system as designed and to prove proper operation of the actuation circuitry.

The engineered safety features components are designed to provide for routine periodic testing.

Plant Technical Specifications specify the test frequency and acceptance criteria to be used for periodic verification of the operability of engineered safety features actuation circuits and components.

The testing of the analog channel inputs is accomplished in the same manner as for the reactor protection system. The engineered safety features logic system is tested by means of test switches to simulate inputs from the analog channels. The test switches interrupt the logic matrix output to the master relay to prevent actuation. Verification that the logic is accomplished is indicated by the matrix test light. Upon completion of the logic checks, verification that the circuit from the logic matrices to the master relay is complete is accomplished by a continuity check. Additional verification is provided by periodically operating the safeguards pumps by means of their normal controls.

3.1.2.39 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. (GDC 39)

Response:

Independent alternate power systems are provided with adequate capacity and testability to supply the required engineered safety features and protection systems.

The plant is supplied with normal, standby and emergency power sources as follows:

1. The normal source of auxiliary power during plant operation is the generator. Power is supplied via the unit auxiliary transformer which is connected to the main leads of the generator.

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2. Power required during plant startup, shutdown, and after reactor trip is supplied from the DEP 115 kV system by a tap from the Robinson 115 kV switchyard to startup transformer No. 2.
3. Two diesel generator sets are connected to the emergency buses to supply power in the event of loss to all other AC auxiliary buses.
4. A dedicated shutdown system exists which will bring the plant to a safe shutdown condition in the event of a fire (10CFR50, Appendix R) or Station Blackout (10CFR50.63).
5. Emergency power supply for vital instruments, control, and some emergency lighting is supplied from two 125 V DC station batteries.

The DG sets are located in the Reactor Auxiliary Building and are connected to separate 480 V auxiliary system buses. Each set will be started automatically on a SI signal or upon under-voltage on its corresponding 480 V auxiliary bus. Each diesel is adequate to supply the engineered safety features for the hypothetical accident concurrent with loss of outside power. This capacity is adequate to provide a safe and orderly plant shutdown in the event of loss of outside electrical power. The dedicated shutdown diesel is located in a separate enclosure and carries loads to bring the plant to a safe shutdown if all other power and control is lost including the DG (Section 8.3).

3.1.2.40 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. (GDC 40)

Response:

Due to the low probability of failure, a double-ended guillotine break of primary coolant loop piping need not be postulated as a design basis event for defining structural loads or for requiring installation of pipe whip or jet impingement devices (Section 3.6) The dynamic effects during blowdown following a LOCA 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 RCS 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.

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A LOCA 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 was considered in the layout of plant equipment and missile barriers. Fluid and mechanical driving forces were calculated, and consideration was given to the possibility of damage due to fluid jets and missiles which might be produced by the action jets. Consideration was given during the design to potential sources of missiles.

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. Individual 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 for the hot leg SIS and RHR system or outside containment for the cold leg SIS. 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 such that no damage outside the missile barrier is possible.

The containment structure is capable of withstanding the effects of missiles originating outside the containment and which might be directed toward it so that no LOCA can result.

All hangers, stops, and anchors were designed in accordance with United States American Standards (USAS) B31.1 Code for Pressure Piping and ACI 318 Building Code Requirements for Reinforced Concrete which provide minimum requirements on material, design and fabrication with ample safety margin for both dead and dynamic loads over the life of the plant.

3.1.2.41 Engineered Safety Features Performance Capability

Criterion: Engineered safety features, such as the emergency core cooling system 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. (GDC 41)

Response:

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 limits of public exposure are taken as the levels and time periods presently outlined in 10CFR100, i.e. 300 rem to the thyroid in two hours at the exclusion radius and 300 rem to the thyroid over the duration of the accident at the low population zone distance. The accident condition considered is the hypothetical case of a release of fission products per the Atomic Energy Commission's technical information report TID 14844. Also, the total loss of all outside power is assumed concurrently with this accident. However, operation of the SIS, considering the single failure criterion, limits the release of fission products from the core to only the gap activity between the fuel pellet and clad.

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Under the above accident condition, the Containment Air Recirculation System and the CSS were designed and sized so that either system is able to supply the necessary post-accident cooling capacity to rapidly reduce the containment pressure following blowdown and cooling of the core by SI. The spray system was designed to provide adequate iodine removal with partial system effectiveness. Partial effectiveness is defined as operation of a system with one active component failure.

Each of the auxiliary cooling systems which serves an emergency function provides sufficient capability in the emergency operational mode 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 plant personnel and the public.

3.1.2.42 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 LOCA to the extent of causing undue risk to the health and safety of the public. (GDC 42)

Response:

All active components of the SIS (with the exception of the hot leg SIS isolation valves) and the CSS are located outside the containment and not subject to containment accident conditions. The accumulators are located in a missile shielded area.

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 (Section 3.11).

The SIS pipes serving each loop are anchored at the missile barrier in each loop area to restrict potential accident damage to the portion of piping beyond this point. The anchorage is designed to withstand, without failure, the thrust force of any branch line severed from the reactor coolant pipe and discharging fluid to the atmosphere, and to withstand a bending moment equal to the ultimate strength of the pipe or equivalent to that which produces failure of the piping under the action of free end discharge to atmosphere or motion of the broken reactor coolant pipe to which the emergency core cooling pipes are connected. This prevents possible failure at any point upstream from the support point including the branch line connection into the piping header.

3.1.2.43 Accident Aggravation Prevention

Criterion: Protection against any action of the engineered safety features which would accentuate significantly the adverse after-effects of a loss of normal cooling shall be provided. (GDC 43)

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Response:

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 control rods insert and remain inserted.

The delivery of cold SI water to the reactor vessel following accidental expulsion of reactor coolant does not cause further loss of integrity of the RCS boundary (See Section 5.3).

3.1.2.44 Emergency Core Cooling System Capability

Criterion: An Emergency Core Cooling System 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 emergency core cooling system shall be evaluated conservatively in each area of uncertainty. (GDC 44)

Response:

Adequate emergency core cooling is provided by the SIS (which constitutes the Emergency Core Cooling System) whose components operate in three modes. These modes are delineated as passive accumulator injection, active SI and residual heat removal recirculation.

The primary purpose of the SIS is to automatically deliver cooling water to the reactor core in the event of a LOCA. This limits the fuel clad temperature and thereby ensures that the core will remain intact and in place, with its heat transfer geometry preserved. This protection is afforded for:

- a) All pipe break sizes up to and including the hypothetical instantaneous circumferential rupture of a reactor coolant loop, assuming unobstructed discharge from both ends
- b) A loss of coolant associated with the rod ejection accident, and
- c) A steam generator tube rupture.

The basic design criteria for LOCA evaluations are:

- a) The cladding temperature is to be less than:
 - 1) The melting temperature of Zircaloy-4
 - 2) The temperature at which gross core geometry distortion, including clad fragmentation may be expected
- b) The total core metal-water reaction will be limited to less than one percent.

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Thus the core geometry is retained to such an extent that effective cooling of the core is not impaired.

For any rupture of a steam pipe and the associated uncontrolled heat removal from the core, the SIS adds shutdown reactivity so that with a stuck rod, no offsite power and minimum engineered safety features, there is no consequential damage to the RCS and the core remains in place and intact.

Redundancy and segregation of instrumentation and components is incorporated to assure that postulated malfunctions will not impair the ability of the system to meet the design objectives. The system is effective in the event of loss of normal plant auxiliary power coincident with the loss of coolant, and can accommodate the failure of any single component or instrument channel to respond actively in the system. During the recirculation phase of a LOCA, the system can accommodate a loss of any part of the flow path since back up alternative flow path capability is provided.

3.1.2.45 Inspection of Emergency Core Cooling System

Criterion: Design provisions shall, where practical, be made to facilitate physical parts of the Emergency Core Cooling System, including reactor vessel internals and water injection nozzles. (GDC 45)

Response:

Design provisions are made to facilitate access to the critical parts of the reactor vessel internals, injection nozzles, pipes, valves, and SI pumps for visual or boroscopic inspection for erosion, corrosion, and vibration wear evidence, and for non-destructive inspection where such techniques are desirable and appropriate.

3.1.2.46 Testing of Emergency Core Cooling System Components

Criterion: Design provisions shall be made so that components of the Emergency Core Cooling System can be tested periodically for operability and functional performance. (GDC 46)

Response:

The design provides for periodic testing of active components of the SIS for operability and functional performance.

Power sources are arranged to permit individual actuation of each active component of the SIS.

The SI pumps can be tested periodically during plant operation using the minimum flow recirculation lines provided. The residual heat removal pumps are used every time the residual heat removal loop is put into operation. All remote operated valves can be exercised and actuation circuits can be tested during routine plant operation.

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3.1.2.47 Testing of Emergency Core Cooling System

Criterion: Capability shall be provided to test periodically the operability of the Emergency Core Cooling System up to a location as close to the core as is practical.
(GDC 47)

Response:

An integrated system test can be performed during the late stages of plant cooldown when the residual heat removal loop is in service. This test would not introduce flow into the RCS but would demonstrate the operation of the valves, pump circuit breakers, and automatic circuitry upon initiation of SI.

The accumulator tank pressure and level are continuously monitored during plant operation and flow from the tanks can be checked at any time using test lines.

The accumulators and the SI piping up to the final isolation valve is maintained full of borated water at refueling water concentration while the plant is in operation. The accumulators and injection lines will be refilled with borated water as required by using the SI pumps to recirculate refueling water through the injection headers. A small bypass line and a return line are provided for this purpose.

Flow in each of the hot leg injection lines and in the main flow line for the residual heat removal pumps is monitored by a flow indicator. Pressure instrumentation is also provided for the main flow paths of the high head and residual heat removal pumps. Level and pressure instrumentation are provided for each accumulator tank.

3.1.2.48 Testing of Operational Sequence of Emergency Core Cooling System

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 into action, including the transfer to alternate power sources.
(GDC 48)

Response:

The design provides for capability to test initially, to the extent practical, the full operational sequence up to the design conditions for the SIS to demonstrate the state of readiness and capability of the system. These functional tests provide information to confirm valve operating times, pump motor starting time, the proper automatic sequencing of load addition to the DG, and delivery rates of the injection water to the RCS.

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3.1.2.49 Containment Design Basis

Criterion: The reactor containment structure, including 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 LOCA, 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 will not result in undue risk to the health and safety of the public. (GDC 49)

Response:

The following general criteria were followed to assure conservatism in computing the required structural load capacity:

- a) In calculating the containment pressure, rupture sizes up to and including a double-ended severance of reactor coolant pipe were considered.
- b) In considering post-accident pressure effects, various malfunctions of the emergency systems were evaluated. Contingent mechanical or electrical failures are assumed to disable one of the DG, two of the four fan-cooler units and one of the two containment spray units.
- c) The pressure and temperature loading obtained by analyzing various LOCA conditions, when combined with operating loads and maximum wind or seismic forces, do not exceed the load-carrying capacity of the structure, its access opening or penetrations.

The most stringent case of these analyses is summarized below:

Discharge of reactor coolant through a double-ended rupture of the main loop piping, followed by operation of only these engineered safety features which can run simultaneously with power from one emergency on-site DG (one high head SI pump, one residual heat removal pump, two fan cooler units, one spray pump), results in a sufficiently low radioactive materials leakage from the containment structure that there is not undue risk to the health and safety of the public.

3.1.2.50 NDT Requirement for Containment Material

Criterion: The selection and use of containment materials shall be in accordance with applicable engineering codes. (GDC 50)

Response:

The selection and use of containment materials comply with the applicable coded and standards tabulated in Section 3.2.

The reinforced containment is not susceptible to a low temperature brittle fracture.

The containment liner is enclosed within the containment and thus will not be exposed to the temperature extremes of the environs. The containment ambient temperature during operation will be between 50 and 120°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 have been designed to the NDTT + 30°F Criterion.

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3.1.2.51 Reactor Coolant Pressure Boundary Outside Containment

Does not apply to HBR 2.

3.1.2.52 Containment Heat Removal System

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. (GDC 52)

Response:

Adequate containment heat removal capability for the Containment is provided by two separate, full capacity, engineered safety feature systems, the CSS (Section 6.2.2) whose components operate in sequential modes, and the Containment Air Recirculation Cooling System (Section 6.2.2).

The primary purpose of the CSS is to spray cool water into the containment atmosphere when appropriate in the event of a LOCA and thereby ensure that containment pressure does not exceed its design value which is 42 psig at 263°F (100 percent relative humidity). This protection is afforded for all pipe break sizes up to and including the hypothetical instantaneous circumferential rupture of a reactor coolant pipe. Although the water in the core after a LOCA is quickly subcooled by the SIS, the CSS design is based on the conservative assumption that the core residual heat is released to the containment as steam.

The Containment Air Recirculation Cooling System is designed to recirculate and cool the containment atmosphere in the event of a LOCA and thereby ensure that the containment pressure cannot exceed its design value of 42 psig at 263°F (100 percent relative humidity). Although the water in the core after a LOCA is quickly subcooled by the SIS, the Containment Air Recirculation Cooling System is designed on the conservative assumption that the core residual heat is released to the containment as steam:

Any of the following combinations of equipment will provide sufficient heat removal capability to maintain the post-accident containment pressure below the design value, assuming that the core residual heat is released to the containment as steam:

- a) All four containment cooling units
- b) Both containment spray pumps, and
- c) Two of the four containment cooling units and one containment spray pump.

Each of the auxiliary cooling systems which serves an emergency function to prevent exceeding containment design pressure, provides sufficient capability in the emergency operational mode to accommodate any single failure of an active component and still perform its required function.

3.1.2.53 Containment Isolation Valves

Criterion: Penetrations that require closure for the containment function shall be protected by redundant valving and associated apparatus. (GDC 53)

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Response:

Isolation valves are provided as necessary for all fluid system lines penetrating the containment to assure at least two barriers for redundancy against leakage of radioactive fluids to the environment in the event of a LOCA. 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 safeguards systems.

With respect to numbers and locations of isolation valves, the criteria applied are generally those outlined by six classes described in Section 6.2.4.

3.1.2.54 Initial Containment Leakage Rate Testing

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 after 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. (GDC 54)

Response:

After completion of the containment structure and installation of all penetrations and weld channels, an integrated leak test was performed in accordance with American Nuclear Society Standard, ANS 7.60 on the total containment volume to ensure that the leakage rate was not greater than 0.1 percent by weight of the containment volume per 24 hr period.

Following the initial integrated leak test a structural strength test on the containment was conducted.

After successful completion of the initial integrated leak and tests an initial sensitive leakage rate test was conducted in accordance with American Nuclear Society Standard, ANS 7.60.

3.1.2.55 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. (GDC 55)

Response:

A leak rate test at the peak calculated accident pressure using the same method as the initial sensitive leak rate test can be performed at any time during the operational life of the plant.

3.1.2.56 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. (GDC 56)

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Response:

A permanently piped system is provided such that all penetrations can be intermittently pressurized throughout the operating life of the plant.

Penetrations are designed with double seals so as to permit pressurization of the interior of the penetration. The large access openings such as the equipment hatch and personnel air locks are equipped with double gasket seals with the space between them connected to the pressurization system. The system utilizes a supply of clean, dry, compressed air which will place the penetrations under an internal pressure greater than the peak calculated accident pressure.

When in Service, leakage from the pressurization system is checked by measurement of the makeup air flow. In the event excessive leakage is discovered, each penetration can then be checked separately at any time.

3.1.2.57 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. (GDC 57)

Response:

Capability is provided to the extent practical for testing the functional operability of valves and associated apparatus during periods of reactor shutdown.

Initiation of containment isolation employs coincident circuits which allow checking of the operability and calibration of one channel at a time. Removal or bypass of one signal channel places that circuit in the half-tripped mode.

The main steam and feedwater barriers and isolation valves in systems which connect to the RCS are hydrostatically tested.

Valves in the Residual Heat Removal System are not considered to be isolation valves in the usual sense inasmuch as the system would be in operation under accident conditions.

3.1.2.58 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, and sumps. (GDC 58)

Response:

Where practicable, all active components and passive components of the CSS are inspected periodically to assure system readiness. The pressure containing systems are inspected for leaks from pump seals, valve packing, flanged joints and safety valves. During operational testing of the containment spray pumps, the portions of the systems subjected to pump pressure are inspected for leaks.

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Design provisions are made to the extent practical to facilitate access for periodic visual inspection of all important components of the Containment Air Recirculation Cooling System.

3.1.2.59 Testing of Containment Pressure - Reducing Systems Components

Criterion: The containment pressure reducing systems shall be designed, to the extent practical so that active components, such as pumps and valves, can be tested periodically for operability and required functional performance. (GDC 59)

Response:

All active components in the CSS were adequately tested both in pre-operational performance tests in the manufacturer's shop and in-place testing after installation. Thereafter, periodic tests are also performed after any component maintenance. Testing of the components of the SIS used for containment spray purposes are described in Section 6.5.2.

The component cooling water pumps and the service water pumps which supply the cooling water to the residual heat exchanger are in operation on a relatively continuous schedule during plant operation. Those pumps not running during normal operation may be tested by changing the operation pump(s).

The Containment Air Recirculation Cooling System was designed to the extent practical so that the components can be tested periodically, and after any component maintenance, for operability and functional performance.

The air recirculation cooling units, and the service water pumps and booster pumps which supply the cooling units, are in operation on an essentially continuous schedule during plant operation; and therefore no additional periodic tests are required to assure the system is operational. However, periodic tests are conducted as described in Section 3.9 to verify certain operational parameters.

3.1.2.60 Testing of Containment Spray System

Criterion: A capability shall be provided to the extent practical to test periodically the delivery capability of the containment spray system at a position as close to the spray nozzles as is practical. (GDC 60)

Response:

Permanent test lines for both the containment spray loops are located so that all components up to the isolation valves at the containment may be tested. These isolation valves are checked separately.

The air test lines, for checking that spray nozzles are not obstructed, connect downstream of the containment spray isolation valves. Air flow through the nozzles is monitored by the use of thermographs.

3.1.2.61 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. (GDC 61)

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Response:

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 CSS, including the transfer to the alternate emergency DG power source.

Means are provided to test initially to the extent practical the full operational sequence of the Air Recirculation Cooling System including transfer to alternate power sources.

3.1.2.62 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 damper. (GDC 62)

Response:

Access is available for visual inspection of the CSS components.

3.1.2.63 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. (GDC 63)

Response:

All active components of the CSS are adequately tested both in pre-operational performance tests in the manufacturer's shop and in place testing after installation. Thereafter, periodic tests are also performed after component maintenance and in accordance with a periodic maintenance schedule.

3.1.2.64 Testing Air Cleanup Systems

Criterion: A capability shall be provided to the extent practical for onsite 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. (GDC 64)

Response:

Permanent test lines are provided for the containment spray headers and located so that all components up to the isolation valve at the containment may be tested. These isolation valves are checked separately. Air test lines for checking the spray nozzles are connected downstream of the isolation valves. Air flow through the nozzles is monitored by a smoke generator or tell tales.

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3.1.2.65 Testing of Operational Sequence of Air Cleanup Systems

Criterion: A 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. (GDC 65)

Response:

Means are provided to test initially under conditions as close to design as is practical the full operational sequence that would bring the CSS into action, including transfer to the emergency DG power source.

3.1.2.66 Prevention of Fuel Storage Criticality

Criterion: Criticality in the new and spent fuel storage pits shall be prevented by physical systems or processes. Such means as geometrically safe configurations shall be emphasized over procedural controls. (GDC 66)

Response:

During reactor vessel head removal and while loading and unloading fuel from the reactor, the boron concentration is maintained such that $\geq 6\%$ $\Delta k/k$ shutdown margin exist in the core on the basis of all RCC assemblies inserted and a complete core installed. This shutdown margin maintains the core subcritical even if all control rods are withdrawn from the core as required by the Post-LOCA sub-criticality event. Weekly checks of the refueling water storage tank boron concentration ensure the proper shutdown margin. A check is made once per 72 hours during core refueling operations and strict administrative controls are used to monitor any dilution of the refueling water in the reactor vessel.

The new and spent fuel storage racks are designed so that the fuel remains subcritical for all normal and postulated accident conditions. In addition, the spent fuel pit has an area set aside for accepting the spent fuel shipping casks. This operation is also done under water. Borated water is used to fill the spent fuel storage pit at a concentration to match that used in the reactor cavity and refueling canal during refueling operations. The fuel in the spent fuel and new fuel storage pits is stored vertically in an array. In conformance with 10 CFR 50.68, K_{eff} shall remain: \square 0.95 for new fuel racks flooded with unborated water or \square 0.98 with optimum moderation, \square 0.95 for low density spent fuel racks flooded with unborated water and subcritical for the high density spent fuel racks flooded with unborated water or \square 0.95 with 200 ppm Boron concentration.

Detailed instructions are available for use by refueling personnel. These instructions, the minimum operating conditions, and the design of the fuel handling equipment, incorporating built-in interlocks and safety features, provide assurance that no incident could occur during the refueling operations that would result in a hazard to public health and safety.

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3.1.2.67 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. (GDC 67)

Response:

The refueling water provides reliable and adequate cooling medium for spent fuel transfer and heat removal from the spent fuel pit is provided by a cooling system with redundant 100 percent capacity pumps. Connections are provided to the component cooling water system for alternative cooling.

3.1.2.68 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. (GDC 68)

Response:

Auxiliary shielding for the Waste Disposal System and its storage components is designed to limit the dose rate to levels not exceeding 1 mR/hr in normally occupied areas, to levels not exceeding 2.5 mR/hr in intermittently occupied areas and to levels not exceeding 15 mR/hr in controlled occupancy areas.

Gamma radiation is continuously monitored at various locations in the Auxiliary Building. A high level signal is alarmed locally and annunciated in the Control Room.

Adequate shielding for radiation protection is provided during reactor refueling 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, less than 2.5 mR/hr, for periodic occupancy of the area by operating personnel. Pit water level is indicated, and water removed from the pit must be pumped out since there are no gravity drains. Shielding is provided for waste handling and storage facilities to permit operation within requirements of 10CFR20.

Gamma radiation is continuously monitored in the Auxiliary Building. A high level signal is alarmed locally and is annunciated in the Control Room.

3.1.2.69 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. (GDC 69)

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Response:

All waste handling and storage facilities are contained and equipment designed so that accidental releases directly to the atmosphere will not exceed the limits of 10CFR100. The components of the Waste Disposal System are not subjected to any high pressures or stresses and are seismic Class I. In addition, the tanks which have a design pressure greater than atmospheric pressure, piping and valves of the system are designed to codes given in Table 3.2.2-5. Hence the probability of a rupture or failure of the system is exceedingly low.

All fuel storage facilities are contained and equipment designed so that accidental releases of radioactivity directly to the atmosphere are monitored and do not exceed the guidelines of 10 CFR 100.

The reactor cavity, refueling canal and spent fuel storage pit are reinforced concrete structures with seam-welded stainless steel plate liners. These Class I structures are designed to withstand the anticipated earthquake loadings such that there will be no liner leakage even in the event the reinforced concrete develops cracks. All operating areas in the fuel storage facilities contain ventilation systems.

All vessels in the Waste Disposal System which are used for waste storage are seismic Class I.

3.1.2.70 Control of Releases of Radioactivity to the Environment

Criterion: The facility design shall include those means necessary to maintain control over the plant radioactive 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 10CFR20 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. (GDC 70).

Response:

Liquid, gaseous, and solid waste disposal facilities are designed so that discharge or effluents and offsite shipments are in accordance with applicable governmental regulations.

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 identification if necessary. Before discharge, radioactive fluids are processed as required and then released under controlled conditions. The system design and operation are characteristically directed toward minimizing

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releases to unrestricted areas. Discharge streams are appropriately monitored and safety features are incorporated to preclude releases in excess of the limits of 10CFR20.

The bulk of the radioactive liquids discharge from the RCS are processed through resin bed and/or evaporators prior to release. This minimizes liquid input to the Waste Disposal System which processes relatively small quantities of generally low-activity, high conductivity floor drain wastes. The processed water from both sources 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 re-used to minimize gaseous wastes. During normal operation, gases are discharged intermittently at a controlled rate from these tanks through the monitored plant vent. The system is provided with holdup capacity and discharge controls for gaseous wastes such that plant operations will not be limited by environmental conditions.

Filter cartridges, spent resins from the demineralizers and concentrates from the evaporators are packaged and stored onsite until shipment offsite for disposal. Suitable containers are used to package these solids at the higher practical concentrations to minimize the number of containers shipped for burial.

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS AND SYSTEMS

3.2.1 SEISMIC CLASSIFICATION

Section 3.7 discusses seismic design of structures, systems, equipment, and components. Section 3.10 discusses seismic qualifications of Class I instrumentation and electrical equipment.

All systems and components of the facility are classified according to their importance. Those items vital to safe shutdown and isolation of the reactor or whose failure either singularly or in combination with the failure of another structure or piece of equipment could result in radiation doses with consequences potentially exceeding guidelines of 10CFR100 or whose failure might cause or increase the severity of an accident are given the classification of Class I. Those items important to reactor operation but not essential to safe shutdown and isolation of the reactor or systems involving orders of magnitude lower radioactive material inventories where a hypothetical accident could result in the release of such inventory and the resulting dose rate at the site boundary would not approach the guideline limit of 10CFR100 are given the classification of Class II. Those items not related to reactor operation or safety were designated Class III.

Class I systems and components are essential to the protection of the health and safety of the public. Quality standards of material selection, design fabrication, and inspection conformed to the applicable provisions of recognized codes, and good nuclear practice.

All systems and components designated Class I were designed so that there would be no loss of function in the event of the maximum hypothetical ground acceleration acting in the horizontal and vertical directions simultaneously. The working stress for both Class I and Class II items is kept within code allowable values for the design earthquake.

3.2.1.1 Definition of Seismic Design Classifications

All equipment and structures are classified as Class I, and Class II, or Class III as recommended in:

- a) TID-7024, "Nuclear Reactors and Earthquakes" August, 1963 (Reference 3.2.1-1) and,
- b) G. W. Housner, "Design of Nuclear Power Reactors Against Earthquakes", Proceedings of the Second World conference on Earthquake Engineering, Vol. I, Japan 1960 (Reference 3.2.1-2) pp. 133, 134 and 137.

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3.2.1.2 Classification of Particular Structures and Equipment

Examples of particular structure and equipment classifications are given as Tables 3.2.1-1 and 3.2.1-2.

The condensate storage tank (Class I) supplies water to the Steam Driven Auxiliary Feedwater (SDAFW) pump, located in the Class I portion of the turbine bay, via a Class I piping system which is primarily routed in a covered (provides protection) trench in the ground floor of the turbine building (Class III). The auxiliary feedwater pump discharge piping is all run and supported in a Class I structure. The steam supply lines to the SDAFW pump are seismic Class I lines routed through the Class I portion of the turbine building. Redundancy is supplied by the motor driven auxiliary feed pumps located in the auxiliary building (Class I) and a Class I piping system that discharges directly in the containment.

The Fire Protection System is Class I in all Class I areas except for reactor coolant pump bay B and C pre-action sprinkler systems which are not pressurized (see Appendix 9.5.1B, Section A.5). The system is provided with five isolation valves in the loop header. These valves can be used to isolate a break in a Class II or III structure. Fire water can then be provided to the Class I structures from the unaffected portion of the loop. The fire pumps are all located on the intake structure which is Class I. The main fire loop is connected with the east fire loop at two locations with two normally closed isolation valves. When this cross connection is used, the supply pipe isolation valve is closed to isolate the fire pumps.

The location of the S/G Drain (flash) Tank and its access platform (a "non-Q" item) are close to the feedwater header (a "Q-listed" item). In order to prevent the tank from falling over and damaging the feedwater header, the foundation and anchor system are designed to seismic Class 1. Since the continued function of neither the tank nor its access platform is required during or after a seismic event, these components have been designed such that failure will not cause damage to a Q-listed component.

The concrete missile shield wall and the support slab for the above-ground portions of the Service Water System North Header are Class I.

The Radwaste Facility was designed as a separate and complete structure from the remainder of the H. B. Robinson facility, and as such reflects current design philosophy that is equal or superior to previous design practices. To facilitate design of the Radwaste Facility, a design basis document was developed.

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TABLE 3.2.1-1

SEISMIC CLASSIFICATION OF BUILDINGS AND STRUCTURES

STRUCTURE	CLASS
Containment (including all penetrations and air locks, the concrete shield, the liner and the interior structures)	I
Spent fuel	I
Control Room	I
Diesel generator room	I
Intake structure (to the extent that water is always available to the service water pumps)	I
Auxiliary building	I
Turbine Class I Bay	I
Turbine Building	III
Radwaste Facility	II
Buildings containing conventional facilities	III

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TABLE 3.2.1-2

SEISMIC CLASSIFICATION OF EQUIPMENT, PIPING AND SUPPORTS

ITEM	CLASS
Reactor Control and Protection System	I
Radiation Monitors	
1. Control Room, Containment, and Auxiliary Building (Except Local/Remote Indicators)	II
2. Support Building (Ex. Turbing, E&RC, EOF, ETC)	III
Process Instrumentation and Controls	I
Reactor	I
Vessel and its supports	
Vessel internals	
Fuel assemblies	
RCC assemblies and drive mechanisms	
Supporting and positioning members	
Incore instrumentation structure	
Reactor Coolant System	I
Piping and valves (including safety and relief valves)	
Steam generators	
Pressurizer	
Reactor coolant pumps	
Supporting and positioning members	
Engineered Safety Features	I
Safety Injection System (including safety injection and residual heat removal pumps, refueling water storage tank, accumulator tanks, boron injection tank, residual heat exchangers and connecting piping and valving)	
Containment Ventilation system (including fans, coolers ducts, valves, absolute filters and demisters)	
Auxiliary Building Ventilation System	I
Condensate storage tanks	I

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TABLE 3.2.1-2 (Cont'd)

ITEM	CLASS
Pressurizer relief tank	II
Residual heat removal loop	I
Containment Penetration Pressurization System	I
Component cooling loop	I
Isolation Valve Seal Water System	I
Sampling System	II
Spent fuel pit cooling loop	II
Fuel transfer tube	I
Emergency Power Supply System	I
Diesel generators and fuel oil storage tank	
DC power supply system	
Power distribution lines to equipment required	
for transformers and switchgear supplying	
the engineered safety features	
Control panel boards	
Motor control centers	
Control Equipment, facilities and lines necessary for	
the above Class I items	I
Waste Disposal System	I
Chemical drain tank	
Waste holdup tanks	
Sump tank	
Gas decay tanks	
Spent resin storage tank	
Reactor coolant drain tank	
Compressors	
Waste evaporator	
Waste evaporator feed pump	
Waste holdup tank pumps	
Sump tank pump	
Interconnecting waste gas piping	

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TABLE 3.2.1-2 (Continued)

ITEM	CLASS
Waste Disposal System	III
All elements not listed as Class I	
Containment polar crane	I
Manipulator and other cranes	III
Conventional equipment, tanks and piping, other than I and II Classes	III
AFW suction and discharge piping and pumps, Service Water and Fire Protection Systems* pumps and piping	I
The Chemical and Volume Control System (except the batching tank, monitor tank, monitor tank pumps, chemical mixing tank, the resin fill tank, evaporator condensate demineralizers, and boron recirculation process and storage system (minus hold-up tanks and recirculation pumps))	I
Batching tank	II
Monitor tanks	II
Monitor tank pumps	II
Chemical mixing tank	II
Resin fill tank	III
Main steam piping from steam generator up to and including the first isolation valve	I
Steam supply lines to the steam-driven AFW pump	I
Steam generator safety and relief valves	I
Service water booster pumps and piping	I
Spent fuel pit storage racks	I
Primary water storage tank	I
Containment vacuum and pressure relief systems	I
Containment purge system	I
Hydrogen Recombiner Supply and Return Piping	III

*Parts as specified in Section 9.5.1 (only parts in Class I areas)

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TABLE 3.2.1-2 (Cont'd)

ITEM	CLASS
Emergency lighting (except for dedicated shutdown emergency lighting in non-seismic areas)	I
Emergency communications	I
Those portions of the process and reactor instruments and controls necessary for no loss of function of Class I systems and equipment.	I
Environmental and Radiation Control Building Lab/Waste Sump	II

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3.2.2 SYSTEM QUALITY GROUP CLASSIFICATION

3.2.2.1 Applicable Industrial Codes for Individual Components and Systems

Industrial Code requirements for particular systems and equipment are given as Tables 3.2.2-1 through 3.2.2-10. The codes applicable are those in effect when the system, equipment, or component was designed. The design took place during the sixties with plant startup occurring in 1970. The order for the reactor vessel and other major components was placed in the early sixties and the major piping was designed in the mid-sixties.

The steam generator lower assemblies were replaced in 1984. The new lower assemblies were designed and fabricated by Westinghouse in 1983. The new lower assemblies utilized the existing channel heads and steam domes with the steam domes being modified to some extent.

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TABLE 3.2.2-1

MAJOR NSSS COMPONENT CODE REQUIREMENTS

ITEM	CODE
Reactor Vessel	ASME* III Class A
Steam Generator	
Tube Side	ASME III Class A
Shell Side	ASME III Class C
Pressurizer	ASME III Class A
Pressurizer Relief Tank	ASME III Class C
Pressurizer Safety Valves	ASME III (1968 Edition, No Addenda)
Pressurizer PORV	ANSI B16.5 (1968 Edition)***
Pressurizer Block Valves	USAS B16.5 (1968 Edition)****
Reactor Coolant Piping	USAS** B31.1

* ASME - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code

** USAS - US of America Standard, Code for Pressure Piping

*** ANSI B16.5 (1968 Edition) – Code for the pressure retaining parts of the valve assembly, ASME, Code for Pumps and Valves for Nuclear Power (November 1968 Draft) for NDE.

**** USAS B16.5 (1968 Edition) – Code for the pressure retaining parts of the valve assembly, ASME B&PV Code Section III Appendix IX for Liquid Penetrant and Magnetic Particle Inspection, ASTM E-94 for Radiographic Inspection.

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TABLE 3.2.2-2

CHEMICAL AND VOLUME CONTROL SYSTEM CODE REQUIREMENTS

ITEM	CODE
Regenerative heat exchanger	ASME III*, Class C
Non-regenerative heat exchanger	ASME III, Class C, tube side, ASME VIII, shell side
Mixed bed demineralizers	ASME III, Class C
Reactor coolant filter	ASME III, Class C
Volume control tank	ASME III, Class C
Seal water heat exchanger	ASME III, Class C, tube side ASME VIII, shell side
Excess letdown heat exchanger	ASME III, Class C, tube side ASME VIII, shell side
Chemical mixing tank	ASME VIII
Deborating demineralizers	ASME III, Class C
Cation bed demineralizer	ASME III, Class C
Seal water injection filters	ASME III, Class C
Holdup tanks	ASME III, Class C
Boric acid filter	ASME III, Class C
Gas stripper package	ASME III, Class C
Evaporator condensate demineralizers	ASME III, Class C
Concentrates filter	ASME III, Class C
Cation ion exchanger	ASME III, Class C
Ion exchanger filter	ASME III, Class C
Condensate filter	ASME III, Class C
Piping and valves	USAS B31.1**

*ASME III - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code Section III, Nuclear Vessels.

**USAS B31.1 - Code for Pressure Piping, and special nuclear cases where applicable.

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TABLE 3.2.2-3

AUXILIARY COOLANT SYSTEM CODE REQUIREMENTS

ITEM	CODE
Component cooling heat exchangers	ASME VIII*
Component cooling surge tank	ASME VIII
Component cooling loop piping and valves	USAS B31.1**
Residual heat exchangers	ASME III***, Class C, tube side ASME VIII, shell side
Residual heat removal piping and valves	USAS B31.1
Spent fuel pit filter	ASME III, Class C
Spent fuel heat exchanger	ASME III, Class C, tube side ASME VIII, shell side
Spent fuel pit demineralizer	ASME III, Class C
Spent fuel pit loop piping and valves	USAS B31.1 shell

*ASME VIII - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section VIII

**USAS B31.1 - Code for Pressure Piping, and special nuclear cases where applicable

***ASME III - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Nuclear Vessels.

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TABLE 3.2.2-4

SAMPLING SYSTEM CODE REQUIREMENTS

ITEM	CODE
Sample heat exchangers***	ASME III *, Class C, tube side ASME VIII, shell side
Sample pressure vessels	ASME III, Class C
Piping and valves	USAS B31.1**

*ASME III - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Nuclear Vessels.

**USAS B31.1 - Code for Pressure Piping and special nuclear cases, where applicable.

***Tube and shell side of replacement HXs are manufactured to ASME Section VIII 1995 Edition, or later.

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TABLE 3.2.2-5

WASTE DISPOSAL SYSTEM CODE REQUIREMENTS

ITEM	CODE
Chemical Drain Tank	No Code
Reactor Coolant Drain Tank	ASME III,* Class C
Sump Tanks	No Code
Spent Resin Storage Tank	ASME III, Class C
Gas Decay Tanks	ASME III, Class C
Waste Holdup Tank	No Code
Waste Condensate Tank	No Code
Laundry and Hot Shower Tank	No Code
Waste Evaporator	No Code
Waste Filter	ASME III, Class C
Piping and Valves	USAS-B31.1** Section I
Polishing Demineralizer	ASME III, Class C
Filter	ASME III, Class C

*ASME III-American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section IV, Nuclear Vessels

**USAS-B31.1-Code for pressure piping and special nuclear cases where applicable.

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TABLE 3.2.2-6

SAFETY INJECTION SYSTEM CODE REQUIREMENTS

ITEM	CODE
Refueling Water Storage Tank	AWWA D100-65*
Residual Heat Exchanger	
Tube Side	ASME Section III** Class C
Shell Side	ASME Section VIII
Accumulators	ASME Section III Class C
Boron Injection Tank	ASME Section VIII Division 2
Valves	USAS B16.5***
Piping	USAS B31.1

* American Water Works Association

** ASME Boiler and Pressure Vessel Code

*** United States of America Standard

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TABLE 3.2.2-7

STEAM AND POWER CONVERSION SYSTEM CODE REQUIREMENTS

ITEM	CODE
System pressure vessels and pump casing	ASME VIII*
Steam generator vessel	ASME III, Class A, tube side** ASME III, Class C, shell side***
System valves, fittings and piping	USAS B31.1****

*American Society of Mechanical Engineers, Boiler and Pressure Vessel Code. Section VIII.

**American Society of Mechanical Engineers, Boiler and Pressure Vessel Code. Section III, Nuclear Vessels.

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

****Code for Pressure Piping, and special nuclear cases where applicable.

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TABLE 3.2.2-8

CONTAINMENT SPRAY SYSTEM CODE REQUIREMENTS

ITEM	CODE
Spray Additive Tank	ASME Section III Class C
Valves	USAS B16-5
Piping (including headers and spray nozzles)	USAS B31.1

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TABLE 3.2.2-9

SERVICE WATER PIPING AND VALVE CODE REQUIREMENTS

ITEM	CODE
Piping	AWWA Class C.200 USAS B31.1 -1955
Valves	USAS 150 lb, B16.5 USAS 125 lb, B16.1, B16.2

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TABLE 3.2.2-10

AUXILIARY FEEDWATER PIPING AND VALVE CODE REQUIREMENTS

ITEM	CODE
Piping	ASA Power Piping B31.1- 1955/USAS B31.10-1967
Valves	Per the pressure and temperature ratings of ASA/USAS B16.5

3.3 WIND AND TORNADO LOADINGS

3.3.1 WIND LOADINGS

Wind loading was based upon the standard wind loading criteria as specified by the American Standards Association (ASA) in "American Standard Code Requirements for Minimum Design Loads in Buildings and Other Structures" (A58.1-1955). This code designates the site as being in a 25 psf zone; however, a 30 psf basic wind loading was used for conservatism. This is equivalent to a gust wind velocity of 108 mph.

In American Society of Civil Engineers (ASCE) Paper No. 3269 "Wind forces on Structures" (Reference 3.3.1-1), Figs. 1(a) and 1(b) specify, for the site, a wind velocity of 80 mph for a 50 year period of recurrence and a wind velocity of 90 mph for 100 year period of recurrence. These are velocities of fastest mile of wind at 30 feet above the ground. The paper recommends a gust factor of 1.1 for a 10 second duration gust for a 125 ft structure transverse (horizontally) to the wind which results in gust winds of 88 mph and 99 mph for the 50 and 100 year recurrence winds. These wind velocities are equivalent to velocity pressures of 20 psf and 25 psf.

The 30 psf ASA loading for a 50 year recurrence is more conservative than the ASCE values for 100 year recurrence due to the fact that the ASA wind loading uses a gust factor of 1.3 and a shape factor of 1.3 which are cumulative.

Further, more accurate data for conditions local to the site have been produced by the National Weather Records Center (NWRC) (U.S. Department of Commerce).

These data show that the annual extreme wind velocities for 50 and 100 year mean recurrences are 72 mph and 80 mph respectively.

It is considered therefore that the choice of a 30 psf loading is very conservative. This is summarized in Table 3.3.1-1.

Wind load was distributed on the structure in accordance with Table 4 (f) of Reference 3.3.1-1. Results of this distribution are shown on Figure 3.3.1-1.

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TABLE 3.3.1-1

COMPARISON OF VARIOUS WIND LOADS

		<u>ASA</u>		<u>ASCE</u>		<u>NWRC</u>	
			50 Year	100 Year	50 Year	100 Year	
Design Load	25 psf zone 30 psf used including 1.3 shape factor	-	-	-	-	-	-
Fastest Mile of Wind	83 mph	80 mph	90 mph	72 mph	80 mph		
Gust Factor	1.3	1.1	1.1	1.1	1.1		
Gusted Wind	108 mph	88 mph	99 mph	79 mph	88 mph		
Velocity Pressure	30 psf	20 psf	25 psf	16 psf	20 psf		

3.3.2 TORNADO LOADINGS

The basic philosophy for passage of a tornado directly across the site was to design against damage to critical systems, accept limited damage to the remainder of the plant, and shut the plant down if necessary for repairs.

3.3.2.1 Applicable Design Parameters

The facility was designed such that a tornado will not interfere with the plant's capability to cope with long term recovery aspects associated with the design basis accident or prevent safe shutdown of the plant. Additionally, the facility was designed such that a tornado will not affect vital structures, systems, or components so as to cause release of radioactivity to the environment.

The containment structure and other buildings and structures that house vital equipment are capable of withstanding the passage of a tornado without loss of function. The walls of these buildings which are constructed of concrete a minimum of 8 inches thick provide considerable protection for the equipment inside from debris thrown about by the tornado.

3.3.2.2 Tornado Loadings

The Reactor Auxiliary Building is a reinforced concrete Class I structure with several doorways located in its exterior walls. The passage of a tornado across the building would create a pressure differential, causing the higher pressure in the building to blow out the doors. This would create a large vent area which would result in the rapid equalization of the pressure between the interior and exterior of the building.

Using a conservative approach to assure that the building will safely withstand the direct passage of a tornado, the building is designed for such an occurrence with no venting. The tornado is assumed to be a 300 mph wind (stagnation pressure 230 psf) with a concurrent pressure drop of 3 psi. The building is designed for no loss of function as defined by the load factor equation:

$$C = 1.0D + 0.1D + 1.0L' + 1.0W_t + 1.0P_t$$

Where:

C = Required Load Capacity Section

D = Dead Load

L' = The weight of equipment in place

W_t = Wind loading due to 230 psf stagnation pressure distributed in accordance with ASCE Paper 3269 (Reference 3.3.1-1)

P_t = Pressure loading due to 3.0 psi internal positive pressure

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The Reactor Auxiliary Building, which includes the Control Room, is designed to withstand, without failure, a 300 mph wind loading coincident with an atmospheric pressure drop of 3 psi due to a tornado. The structure was designed in accordance with the provisions for computation of structural capacity of members of American Concrete Institute (ACI) 318-63 Part IV-B.

The intake structure is inherently safe from tornado loads as it is mostly beneath the lake water level. It was not specifically designed for the tornado loads. However the pressure drop (3 psi) would have no effect on it as it is open so that venting would occur and the water level inside would respond to the drop in pressure thereby equalizing the pressure. The wind load would act on an insignificant portion of the structure. The structure is designed for the Class I earthquake criteria.

The Class I portion of the turbine structure houses the steam driven auxiliary feed pump which provides backup to redundant motor driven auxiliary feed pumps located in the tornado proof Reactor Auxiliary Building.

Unlike the South Service Water header, which is buried piping. A portion of the North Service Water header is routed above-ground. Accordingly, the North Service Water header has been designed with protective barriers to ensure that this portion of the Service Water System is capable of withstanding the passage of a tornado without a loss of function. The protective barriers provided for the above-ground portion of the North Service Water header are comprised of a double layer of turbine deck-type grating and a poured concrete wall of varying thickness (minimum thickness of 12 inches), in the area near Service Water Pit #3. These missile protective barriers provide acceptable protection against a 2 in x 4 in x 10 foot plank. The concrete structure is designed as Class I.

An evaluation of the effects of tornados was performed to determine the availability of the Auxiliary Feedwater System (AFWS) (References 3.3.1-2, 3.3.1-3, and 3.3.1-4). This evaluation concluded that the redundancy and separation of the AFWS sources, together with the low observed frequency of tornadoes, result in a high confidence that the AFWS will be available following a tornado strike.

Tornado loading capabilities of the fuel handling building are:

- a) Evaluation of the aluminum siding and its attachments located around the spent fuel pit area indicates an expected "blow off" at wind loads corresponding to approximately 125 mph. For this loading condition, prior to the loss of siding the steel structure will have stresses below 95 percent of yield.
- b) Once the siding has "blown off", the structure is vented and is capable of withstanding the full tornado loading at stresses less than yield.

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The Spent Fuel Pit is a Class I structure and can withstand, without failure, a 300 mph wind loading coincident with an atmospheric pressure drop of 3 psi.

The design for the primary water storage tank and the diesel fuel oil tank includes consideration for tornado wind loads (i.e. 300 mph wind and 3 psi negative pressure).

3.3.2.3 Effect of Failure of Structures or Components Not Designed for Tornado Loads

All components necessary for safe operation which are located outdoors and exposed to damage from tornado debris are parts of redundant systems and as such have sufficient backup to provide reasonable assurance that no loss-of-function of the systems will result because of tornado damage.

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The redundancy and location of vital equipment is as follows:

1. Emergency steam generator feed is provided by a steam-driven pump backed up by two motor-driven pumps. Both motor driven pumps are inside buildings. In the event the steam lines supplying steam to the turbine-driven pump are damaged, the motor driven pumps, powered by the emergency diesel generators located in Auxiliary Building, can be used.
2. The four service water pumps are located in three separate bays in the intake structure, the middle bay containing two pumps. The pumps are sufficiently isolated to make it unlikely that a missile could damage more than one pump. The walls separating the bays and the deck above the piping are two and one half foot thick reinforced concrete. Thus it is highly unlikely that a missile could get to the pumps.
3. The Condensate Storage Tank could be pierced by a missile. However, missile impact would have to occur at the bottom of the tank to cause total loss of water. Also, the service water system or the well water system could be used to supply water to the secondary side of the Steam Generators to remove decay heat.
4. If a tornado or tornado debris destroys the outside electrical power supply, the unit could be tripped and either of the two emergency diesel generators, located in the Auxiliary Building, would supply sufficient power to place and maintain the plant in the safe shutdown condition. Also, a dedicated shutdown diesel is available for plant shutdown.

The DSDG and appurtenances are not specifically qualified to remain operable during and following a tornado event. However, as an element of the Station Blackout (10CFR50.63) Coping Analysis, the structural adequacy of the DS power supply system, under hurricane wind loading, was assessed, and minor modifications were implemented as appropriate. These modifications established adequate supports for conduits and electrical ducts of the DSDG power system that are exposed to hurricane wind forces. In addition, minor structural modifications were implemented to strengthen the DSDG battery enclosure, and the 4.16 kV switchgear room, which contains DS electrical distribution equipment.

The results of the DS power system wind-loading evaluation are documented in the SBO Coping Analysis, 8S19-P-101.

5. Piping and electrical connections from the Auxiliary Building to the Containment are each in two separate concrete enclosures following different routes. Other vital piping and equipment is located in below-grade trenches or pits with concrete covers.

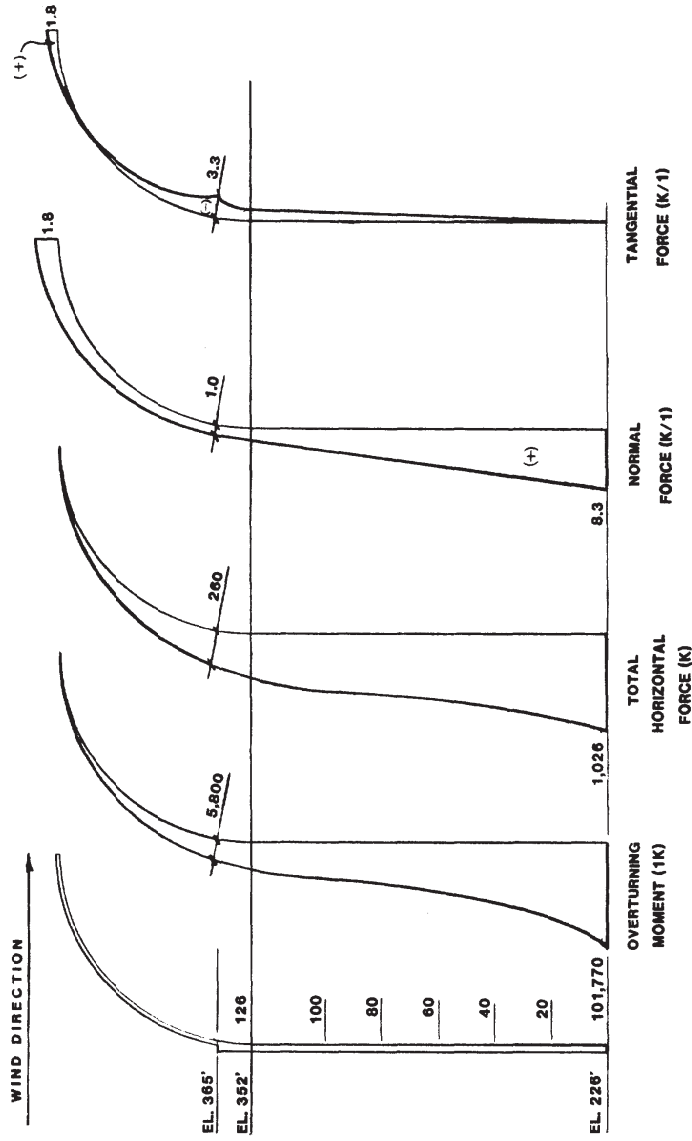
Therefore it is concluded that the plant can withstand the effects of the design tornado without endangering the health and safety of the public.

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REFERENCES: SECTION 3.3

- 3.3.1-1 ASCE PAPER NO. 3269 "Wind Forces on Structures", Transactions of the American Society of Civil Engineers, Vol. 126, Part II (1961).
- 3.3.1-2 Letter, LAP-83-385, dated August 29, 1983, from S. R. Zimmerman (CP&L) to G. Requa (NRC), Auxiliary Feedwater System Evaluation.
- 3.3.1-3 Letter, NLS-85-168, dated March 14, 1985, from S. A. Varga (NRC) to E. E. Utley (CP&L), Feedwater System Tornado Missile Protection (TAC No. 49223).
- 3.3.1-4 Letter, NLS-85-208, dated June 13, 1985, from S. R. Zimmerman (CP&L) to S. A. Varga (NRC), Auxiliary Feedwater System Tornado Protection.
- 3.3.2-1 "Station Blackout Coping Analysis Report," H. B. Robinson, Unit 2, Document No. 8S19-P-101.

WIND LOADING



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UNIT 2
Carolina Power & Light Company
UPDATED FINAL SAFETY ANALYSIS REPORT

WIND LOADING

FIGURE 3.3.1 - 1

3.4 WATER LEVEL (FLOOD) DESIGN

Flooding is a physical impossibility at this site since the maximum cooling lake level which can be maintained by the drain and appurtenant structures is below plant grade.

3.5 MISSILE PROTECTION

Engineered Safety Features and associated systems are protected from loss of function due to dynamic effects and missiles which might result from a loss-of-coolant accident (LOCA). Protection is provided by missile shielding and/or segregation of redundant components.

The Reactor Coolant System is surrounded by concrete shield walls. These walls provide shielding to permit access into the containment during full power operation for inspection and maintenance of miscellaneous equipment. These shielding walls also provide missile protection for the containment liner plate. The concrete deck over the Reactor Coolant System also provides for shielding and missile damage protection.

A structure is provided over the pressurizer to block any missiles generated from fracture of the pressurizer valves.

The control rod drive mechanism is effectively shielded from missiles generated by a LOCA by the reactor cavity wall which is 6 ft thick reinforced concrete. The types of missiles considered were the same as for the crane wall. The 3 ft thick crane wall was checked and shown to preclude the possibility of missile penetration. Thus, the 6 ft thick reactor cavity wall will preclude missile penetration.

All injection lines penetrate the containment adjacent to the auxiliary building.

For most of the routing, the safety injection lines are outside the reactor and steam generator shielding, and hence are protected from missiles originating within these areas.

Systems which contain hot pressurized fluids and which might affect the engineered safeguards components have been carefully checked for possible missile sources. The general criterion adopted has been to protect, when necessary, against the generation of missiles rather than to allow missile formation and then to contain their effects.

Once the design requirement that the above systems are not to be sources of missiles has been set forth, identification of potential deficiencies and generation of adequate fixes took place through the quality assurance program.

The following sections illustrate how this approach has been implemented.

3.5.1 MISSILE SELECTION AND DESCRIPTION

3.5.1.1 Reactor Coolant Pump

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 any small fragments would be contained by the heavy casing.

The primary coolant pump flywheels are shown in Figure 3.5.1-1. 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-operation 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 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 finished flywheels are subjected to 100 percent 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 (shown in Figure 3.5.1-2) less than 50 percent of the minimum specified material yield strength at room temperature (100 to 150°F). Bursting speed of the flywheels has been calculated on the basis of Griffith-Irwin's results (References 3.5.1-1 and 3.5.1-2), 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 percent,
- b) A crack through the thickness of the flywheel at the bore, and
- c) 400 cycles of start up operation.

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 rate was 0.030 in. to 0.060 in. per 1000 cycles.

The reactor coolant pump motor bearings are of conventional design - the radial bearings are the segmented pad type, and the thrust bearings are tilting pad Kingsbury bearings. All are oil lubricated - the lower radial bearing and the thrust bearings are submerged in oil, and the upper radial bearing is oil fed from an impeller integral with the thrust runner. Low oil levels would signal an alarm in the Control Room and require shutting down of the pump. Each motor bearing contains embedded temperature detectors, and so initiation of failure, separate from loss of oil, would be indicated and alarmed in the Control Room as a high bearing temperature. This, again, would require pump shutdown. Even if these indications were ignored, and the bearing proceeded to failure, the low melting point Babbitt metal on the pad surfaces would ensure that no sudden seizure of the bearing would occur. In this event the motor would continue to

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drive, as it has sufficient reserve capacity to operate, even under such conditions. However, it would demand excessive currents and at some stage would be shut down because of high current demand.

It may be hypothesized that the pump impeller might severely rub on a stationary member and then seize. Analysis has shown that under such conditions, assuming instantaneous seizure of the impeller, the pump shaft would fail in torsion just below the coupling to the motor. This would constitute a loss of coolant flow in the one loop.

Following the seizure, the motor would continue to run without any overspeed, and the flywheel would maintain its integrity, as it would still be supported on a shaft with two bearings.

There are no other credible sources of shaft seizure other than impeller rubs. Any seizure of the pump bearing would be precluded by shearing of the graphitar in the bearing. Any seizure in the seals would result in a shearing of the anti-rotation pin in the seal ring. An inadvertent actuation of the shutdown seal (SDS) on the RCP shaft, with the shaft still rotating, will not interrupt core cooling flow provided by the RCP or adversely impact RCP coastdown. In the event of a loss of all seal cooling, the SDS will actuate on high seal cooling temperature to limit leakage from the RCP seal package. Leakage is limited when a thermal actuator retracts and causes the SDS piston ring and polymer ring to clamp down around the pump shaft. The motor has adequate power to continue pump operation even after the above occurrences. Indications of pump malfunction in these conditions would be initially by high temperature signals from the bearing water temperature detector, excessively high No. 1 seal leakoff indications, and off-scale No. 1 seal leakoff indications, respectively. Following these signals, pump vibration levels would be checked. These would show excessive levels, indicating some mechanical trouble. Again, the pump would be shut down for investigation.

The 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. Besides examining the externally produced loads from the nozzles and support lugs, an analysis is made of the effect of gyroscopic reaction on the flywheel and bearings and in the shaft, due to rotational movements of the pump about a horizontal axis, during the maximum seismic disturbance.

The pump would continue to run unaffected by such conditions. In no case does any bearing stress in the pump or motor exceed or even approach a value which the bearing could not carry.

The design requirements of the bearings are primarily aimed at ensuring a long life with negligible wear, so as to give accurate alignment and smooth operation over long periods of time. To this end, the surface bearing stresses are held at a very low value, and even under the most severe seismic transients or other accidents, do not begin to approach loads which cannot be adequately carried for short periods of time.

Because there are no established criteria for short-time stress-related failures in such bearings, it is not possible to make a meaningful quantification of such parameters as margins to failure, safety factors, etc. A qualitative analysis of the bearing design, embodying such considerations, gives assurance of the adequacy of the bearing to operate without failure.

As is generally the case with machines of this size, the shaft dimensions are predicated on avoidance of shaft critical speed conditions, rather than actual levels of stress.

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There are many machines as large as, and larger than these, that are designed to run at speeds in excess of first shaft critical. However, it is considered desirable in a superior product to operate below first critical speed, and the reactor coolant pumps are designed in accordance with this philosophy. This results in a shaft design which, even under the severest postulated transient, gives very low values of actual stress. While it would be possible to present quantitative data of imposed operational stress relative to maximum tolerable levels, if the mode of postulated failure were clearly defined, such figures would have little significance in a meaningful assessment of the adequacy of the shaft to maintain its integrity under operational transients. However, a qualitative assessment of such factors gives assurance of the conservative stress levels experienced during these transients.

So in each of these cases, where it is the functional requirements of the component that controls its dimensions, it can be seen that if these are met, the stress-related failure cases are more than adequately satisfied.

It is thus considered to be out of the bounds of reasonable credibility that any bearing or shaft failure could occur that would endanger the integrity of the pump flywheel.

The flywheels were visually examined at the first refueling. At the fourth refueling the outside surfaces were examined by ultrasonic methods as part of a plant surveillance program.

During normal operation, the reactor coolant pumps are supplied from the unit auxiliary bus and are therefore tied to the turbine generator frequency (speed). On occurrence of unit (turbine) trip the pump electrical buses are tripped from the auxiliary transformer without any intentional delay.

On most electrical and mechanical events which cause the turbine to be tripped, the reactor coolant pump buses and the unit are tripped simultaneously and the pumps will therefore not exceed their normal or pretrip running speed. If for some unlikely reason the only plant trip is a turbine overspeed trip (mechanical-hydraulic trip), then the pump trip will be initiated by the turbine hydraulic system and the trip point will be between 106 and 110 percent of the turbine generator synchronous speed. The turbine overspeed trip point will be set at 111.0 percent of synchronous speed (1998 rpm), and the maximum resulting turbine overspeed will be 120 percent. If it is further postulated that a failure in the pump trip occurs at the same time, then this would result in a maximum overspeed for that pump equal to that of the turbine, i.e., 120 percent. The design overspeed of the pump is 125 percent.

3.5.1.2 NSSS Valves

All the valves installed in the Nuclear Steam Supply System (NSSS) have stems with back seats. This rules out the probability of ejecting valve stems as analysis shows that even if it were assumed that the stem threads fail, the back seat or the upset end cannot penetrate the bonnet and thereby become a missile. Additional interference is encountered with air and motor operated valves.

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Valves with nominal diameter larger than 2 in. have been designed against bonnet-body connection failure and subsequent bonnet ejection by the following means:

- a) Using the design practice of American Society of Mechanical Engineers (ASME) Section VIII which limits the allowable stress of bolting material to less than 20 percent of its yield strength
- b) Using the design practice of ASME Section VIII for flange design; and,
- c) By controlling the load during the bonnet body connection stud tightening process.

The pressure containing parts, except the flanges and studs, were designed per criteria established by the US of America Standard (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. 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 are radiographed and dye penetrant tested to verify soundness.

Stud material is ASTM A193-B7 and nut material is ASTM A194-2H or ASTM A194-7. Design criteria limit the stress of the studs to the allowable limits established in the ASME Code, i.e., 20,000 psi. This stress level is far below the material yield, i.e., about 105,000 psi. However, body to bonnet stud bolt and nut material for valves CVC-350 and CVC-381 has been changed to ASTM B637, UNS07718. The design criteria limit the stress of the studs for these valves to the allowable limits established in the ASME Code, i.e., 37,000 psi. This stress level is far below the material yield, i.e., about 150,000 psi. Inservice bolting stresses in these valves are maintained at valves high enough to ensure a leak-free connection. These inservice allowable bolting stresses exceed the design criteria of 20% of yield strength. This practice is allowed by Section VIII to ensure a leak-free connection.

An alternative approach to a thru c is to design, fabricate and maintain the valves in accordance with ASME Section III, Class 1 and Class 2 requirements.

Valves with nominal diameters of 2 in. or smaller were forged and have screwed bonnet with canopy seals. The canopy seal is the pressure boundary while the bonnet threads are designed to withstand the hydrostatic end force. The pressure containing parts were designed per criteria established by the USAS B16.5 specification.

3.5.1.3 Turbine Missiles

3.5.1.3.1 Consequences of Turbine Generator Unit Overspeed.

3.5.1.3.1.1 High Pressure Turbine.

Due to the very large margin between the high pressure spindle bursting speed and the maximum speed at which the steam can drive the unit with all the admission valves fully open, the probability of an HP spindle failure is practically zero (Reference 3.5.1-4).

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Based on the admission steam thermodynamic properties and blade geometry, the maximum theoretical speed at which the unit may run is 208 percent of nominal.

Based on the stress analysis of the low pressure disks, the maximum actual speed at which the unit may run is 175 percent of nominal.

The minimum bursting speed of the spindle, based on the minimum specified mechanical properties of the spindle material, is 270 percent of nominal. The actual bursting speed is closer to 300 percent of nominal than 270 percent.

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Hence, the actual margin between the bursting speed and the maximum running speed is on the order of 125 percent of nominal, i.e., 300 percent minus 175 percent.

No failure of the high pressure turbine is anticipated as a consequence of a unit runaway; therefore, no missiles are expected to be generated.

3.5.1.3.1.2 Low Pressure Turbine

A missile probability analysis was performed for the Robinson 2 BB81R-18m2 rotors with Advanced Disc Design shrunk-on discs. Based on conservative assumptions, the probability of an external missile for speeds up to 120% of rated speed is $1.76\text{E-}7$ for a disc inspection interval of 100,000 operating hours (Reference 3.5.1-5).

As documented in WCAP-16054-P, Robinson 2 has elected to perform turbine valve test intervals at a frequency of every 6 months. This results in a probability of overspeed of $2.90\text{E-}6$ per year.

As summarized in the missile report (Reference 3.5.1-5), these probabilities are well below the Nuclear Regulatory Commission (NRC) limit of $1.0\text{E-}5$ per year for an unfavorably oriented unit.

The missile analysis methodology used was submitted to the NRC and approved in March 2004 (Reference 3.5.1-6, which includes the NRC Safety Evaluation). While the Topical Report was submitted for the 13.9m2 design, the Safety Evaluation applies to other Siemens Advanced Disc Designs, such as the 18m2.

DEP has chosen to concentrate their effort to minimize the likelihood of a turbine-generator unit overspeeding above the design speed by performing a detailed probabilistic safety analysis and testing the turbine steam valves accordingly (reference WCAP-11525 and WCAP-16054). Since the likelihood of an overspeed is reduced to practically non-credible by this turbine valve testing program, no additional design consideration has been given to the consequences of an overspeed event.

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3.5.1.3.2 Overspeed Analysis

For determination of the maximum design overspeed, the following conservative sequence is assumed:

- a) The unit is operating at full load with all turbine valves wide open
- b) The entire turbine load is dropped instantaneously (no credit is taken for plant auxiliary load)
- c) The auxiliary governor is assumed to operate improperly (i.e., does not respond to turbine load mismatch)
- d) Trip is initiated at the emergency overspeed setpoint (111 percent overspeed), and
- e) From this point on, the turbine valves operate in the prescribed manner.

At the instant the load is dropped, the unit is assumed to accelerate at a constant maximum rate corresponding to the initial steam flow and rotational inertia of the unit until the unit reaches the emergency overspeed trip setpoint plus a time delay of 0.1 second. Flow into the turbine is then calculated during valve closure and is modified for flow versus lift characteristics. It takes approximately 0.3 second to close fully all of the turbine valves following the initial 0.1 second delay. Once the valves close completely, additional overspeeding is calculated using the energy stored in the turbine, the moisture separators and the related piping.

The resulting maximum overspeed calculated in this manner is nominally 114.9 percent with the LP turbine upgrade to 18m² (Reference 3.5.1-7). Considering the uncertainties in valve characteristics and variations in closing times, it is estimated that the calculated overspeed is within 2 percent. Thus, an overspeed of 120 percent is conservatively assumed.

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3.5.2 STRUCTURES, SYSTEMS AND COMPONENTS TO BE PROTECTED FROM
EXTERNALLY GENERATED MISSILES

The buildings and structures that house vital equipment are capable of withstanding the passage of a tornado without loss of function, as is the containment structure. This protection is afforded by the solid reinforced concrete walls and roof of the building which will prevent penetration of such a missile. Where openings exist in the outside concrete wall, any vital equipment inside the building is shielded from a missile penetrating through the opening by a reinforced concrete shield wall or a reinforced concrete wall already existent within the building.

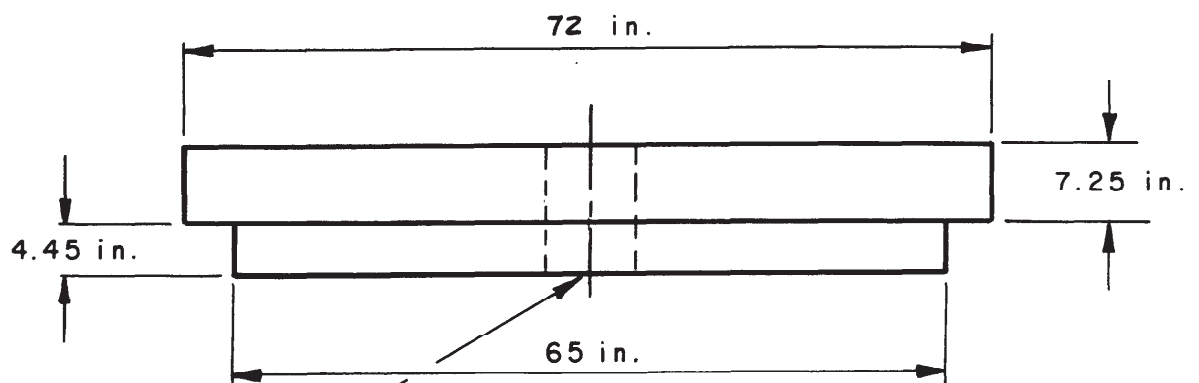
The walls of these buildings are concrete, at least 8 in. thick, and thus provide considerable protection for the equipment inside from debris thrown about by the tornado. For example, a 10 ft length of wooden 2 x 4 traveling at a velocity of 300 mph striking the surface, end-on, would barely penetrate a 8 in. thick wall and would penetrate less than 4 in. into a concrete wall 12 in. thick or thicker.

The design of structures against tornado loadings is discussed in Section 3.3.2.

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REFERENCES: SECTION 3.5

- 3.5.1-1 Final Facility Description and Safety Analysis Report, Carolina Power and Light, H. B. Robinson Plant Unit 2, Section 14.1.13.
- 3.5.1-2 Report, "H. B. Robinson Unit No. 2, Likelihood and Consequences of Turbine Overspeed", June 5, 1970.
- 3.5.1-3 Letter dated November, 20, 1981, J.F. Sturge, Westinghouse Electric Corporation to E. G. Hollowell, CP&L "Inspection Intervals for Robinson Unit No. 2".
- 3.5.1-4 EC-02262, "Missile Generation Risk Assessment for Original and Retrofit HP Rotors", Robin L. Banks, December 17, 2002, Siemens Westinghouse Power Corporation.
- 3.5.1- 5 CT-27454, Revision 1, "Missile Probability Analysis Report - Progress Energy, Robinson 2 - BB81R-18m2", March 19, 2009, Siemens Power Generation.
- 3.5.1-6 TP-04124, "Missile Probability Analysis for the Siemens 13.9m2 Retrofit Design of Low-Pressure Turbine by Siemens AG", June 7 , 2004, Submitted t o the Nuclear Regulatory Commission as Topical Report TP- 041 2 4- NP- A, For Public Record.
- 3.5.1-7 EC-09007 , Progress Energy Robinson Unit #2 , 1 8m2 LP Upgrade Retained Components Study , January 13 , 20 09 , Siemens Energy, Inc .



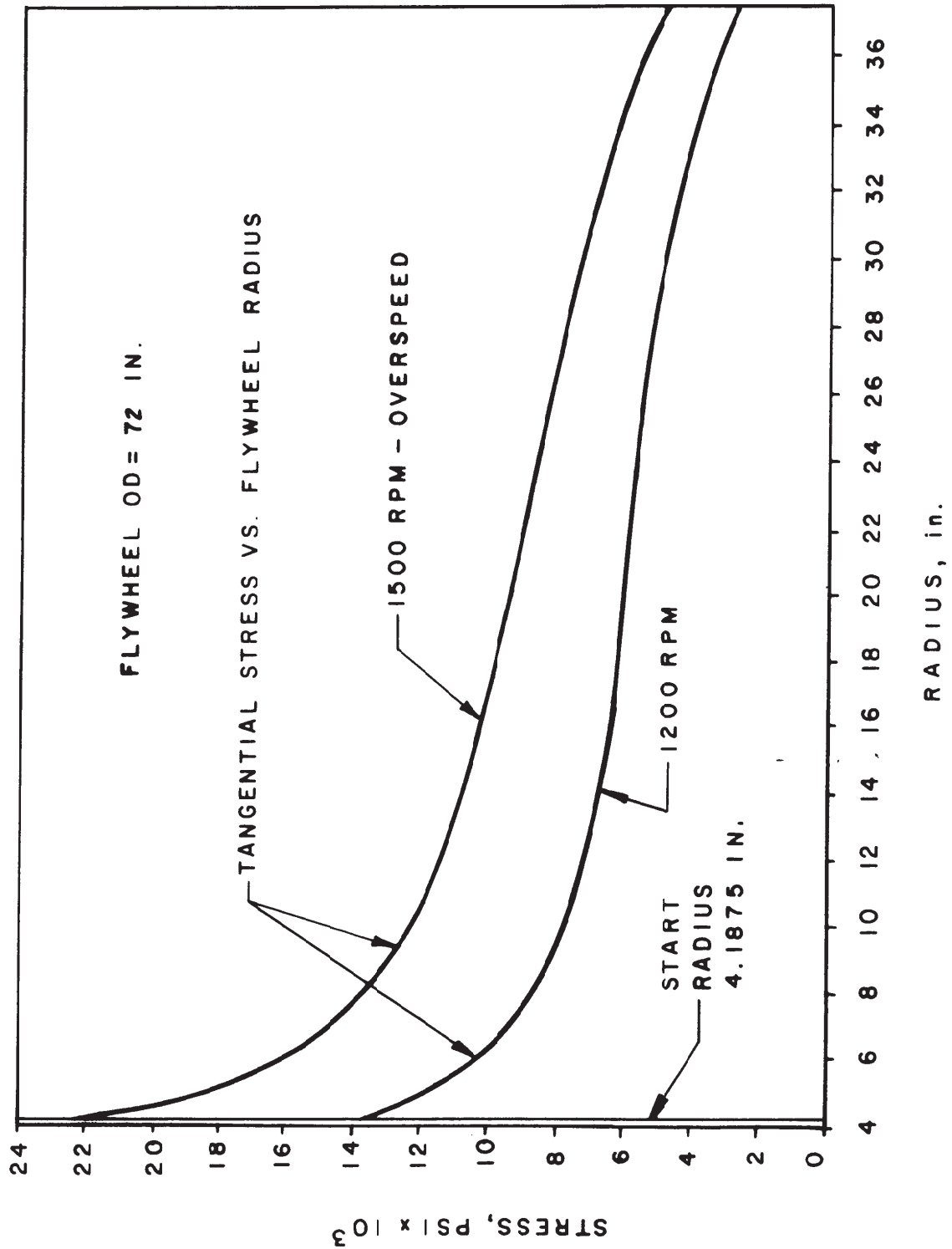
BORE OF ~ 8-3/8 in. DIAMETER WITH 3 KEYWAYS

Revision No. 14

**H. B. ROBINSON
UNIT 2**
Carolina Power & Light Company
**UPDATED FINAL
SAFETY ANALYSIS REPORT**

REACTOR COOLANT PUMP MOTOR
FLYWHEEL DIMENSIONS

FIGURE
3.5.1 - 1



Revision No. 14

H. B. ROBINSON
UNIT 2
Carolina Power & Light Company
UPDATED FINAL
SAFETY ANALYSIS REPORT

REACTOR COOLANT PUMP MOTOR
FLYWHEEL STRESSES

FIGURE
3.5.1 - 2

3.6 PROTECTION AGAINST DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

3.6.1 POSTULATED PIPING FAILURES IN FLUID SYSTEMS INSIDE CONTAINMENT

Pipe restraints are provided which provide reasonable assurance of protection against pipe reactions in accordance with General Design Criteria 40 and 42 (Section 3.1).

The design criteria and technical approach to protection against dynamic effects are discussed below. Loading combinations, stress criteria, etc., are discussed in Sections 3.7, 3.8, and 3.9 of the Updated FSAR.

The internal structural system is designed to mitigate loading due to incident rupture in the reactor coolant line, feedwater, and steam lines. Incident rupture is considered in only one line at a time. The snubber and key systems are designed to deliver rupture thrusts on the steam generator into the internal structural system.

References 3.6.2-5, 3.6.2-6, and 3.6.2-7 demonstrated that the probability of failure in the primary system is low enough that a double-ended guillotine break (DEGB) need not be postulated as a design basis event for defining structural loads or for requiring installation of pipe whip or jet impingement devices. This conclusion eliminated the previous requirement to address asymmetric loads due to a DEGB in the primary system, which includes the RPV, its internals and the primary coolant loop. The technical information also demonstrated that piping would leak a detectable amount well in advance of any crack growth that could result in a break. This concept is referred to as Leak-Before-Break.

WCAP-15628 (Reference 3.6.1-1) provides a Leak-Before-Break (LBB) analysis of the HBR, Unit No. 2, large bore RCS piping and components that meets modified requirements for LBB required by 10 CFR 50, Appendix A, General Design Criterion 4, and that uses the recommendations and criteria from the NRC Standard Review Plan for LBB evaluations. The analysis uses material property values that account for thermal aging and thermal cycles appropriate for the 60-year operating period projected for license renewal.

The pipe restraints have been placed such that pipe whip associated with a loss-of-coolant accident (LOCA) is restrained to the extent that:

- a) The containment integrity would not be violated
- b) For breaks in small pipes containing reactor coolant, high head safety injection capability to at least one leg of the affected loop will be maintained. Small pipes are considered to be lines which have an inside diameter equal to or less than 4 in.
- c) For breaks in pipes larger than 6 in. inside diameter, Emergency Core Cooling System (ECCS) for the entire loop is assumed to be inoperative
- d) A pipe break would not propagate between loops

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- e) A pipe break in one loop would not defeat ECCS capability to the other loops
- f) The primary equipment supports would continue to perform their design function
- g) Rupture of steam and feedwater lines does not occur.

The coolant loop supports are designed to restrict the motion to about one-tenth of an in., whereas the attached safety injection piping can sustain a 3 in. displacement without exceeding the working stress range.

The steam lines are strapped to the crane wall at intervals selected to prevent a whipping pipe from contacting the liner. The straps are designed so that no interference with the normal thermal expansion modes of the steam lines results, and are anchored and sized to restrain the whipping pipe within the elastic limits of the straps.

Those portions of the feedwater and the main steam piping which are located in the annular space formed by the crane wall and the lined wall of the containment structure are restrained to the crane wall. The spacing of the restraints is less than the radial distance from the outside of the pipe to the containment wall so that in the event of a pipe rupture a completely severed pipe cannot strike the containment liner, regardless of the manner in which it whips.

The containment structure and all penetrations are designed to withstand, within design limits, the combined loadings of the design basis accident and maximum seismic conditions.

No piping systems which penetrate the vapor barrier are anchored to the liner. 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 vapor barrier will not be breached due to a hypothesized pipe rupture combined, for the case of the steam line, with the coincident internal pressure. For the penetration design, the pipe capacity in flexure was assumed to be limited to the plastic moment capacity based upon the ultimate strength of the pipe material. All lines connected to the primary coolant system that penetrate the vapor barrier are also anchored in the secondary shield walls (i.e., walls surrounding the steam generators and reactor coolant pumps) 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 hypothesized rupture of the attached pipe or the loads induced by the hypothetical (larger) earthquake (Section 3.7).

All isolation valves are supported to withstand, without impairment of valve operability, the loadings of the design basis accident or maximum seismic conditions.

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3.6.2 POSTULATED PIPING FAILURES IN FLUID SYSTEMS OUTSIDE OF CONTAINMENT

High energy piping systems outside containment have been evaluated to assure that failures would not result in unacceptable degradation of plant safety (References 3.6.2-1 through 3.6.2-4). These analyses are summarized in the following sections.

The effect of rupture of fire water system piping is discussed in Section 9.5.1.4.4.3.

3.6.2.1 Basis of Analysis

For this analysis, high energy piping systems are defined as those which have both a service temperature above 200°F and a design pressure above 275 psig. The plant operational conditions, under which this definition applies, includes normal reactor operation and the operational basis earthquake defined as an upset condition.

The systems of HBR 2 which fall under the definition listed above, are:

- a) Main Steam System
- b) Feedwater System
- c) Steam Generator Blowdown System
- d) Chemical and Volume Control System, and
- e) Steam Supply to Auxiliary Feedwater Pump Turbine.

Design bases piping break locations were determined for ASME Section III, Code Class II and III, or ANSI B31.1 piping breaks. There is no ASME Section III, Code Class I piping outside containment for HBR 2. Breaks were postulated to occur at the following locations in each piping run or branch run:

- a) The terminal ends
- b) At intermediate locations where:
 - 1) A high stress point has been calculated (minimum of 2 points)
 - 2) The circumferential or longitudinal stresses, derived on an elastically calculated basis under the loadings associated with seismic events and operational plant conditions, exceed $0.8 (S_h + S_A)$ or the expansion stresses exceed $0.8 S_A$.

S_h is the stress calculated by the rules of NC-3600 and ND-3600 for Class II and III components, respectively, of the ASME Code, Section III, Winter 1973 Addenda.

S_A is the allowable stress range for expansion stress calculated by the rules of NC-3600 of the ASME Code, Section III, or the ANSI Standard Code for Pressure Piping, ANSI B31.1.0-1967.

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The analysis considered pipe whip, compartment pressure differentials, jet impingement loads, and integrity of the containment.

3.6.2.2 Plant Operability Following Postulated Pipe Rupture

The criteria used to assure plant operability are:

- a) The control room will be maintained habitable and its essential equipment functional; the capability to bring the reactor to a cold shutdown from the control room will be maintained
- b) The capability to mitigate the consequences of an accident and bring the reactor to a cold shutdown condition will be assured. Loss of redundancy of equipment (as a consequence of the postulated break) required to mitigate the consequences of an accident and obtain a safe hot shutdown will not occur. Environmentally induced failures caused by a leak or rupture which would not in itself result in protective action, but may disable protective equipment will also be considered. In this regard a loss of redundancy will be permitted but a loss of function will not be permitted. For such situations the capability for bringing the plant to a safe cold shutdown will be assured.

3.6.2.3 Results

The results of the analyses of postulated high energy pipe breaks outside of containment are:

- a) A postulated steam line break at the turbine would whip and impact containment. The resultant penetration and effective impactive load are limited to about 20 percent of the allowable. It can, therefore, be concluded that containment integrity is maintained
- b) The steam header does not fail as the result of a postulated steam line break
- c) The safeguards switchgear is not compromised by a postulated feedwater line break
- d) The pressure, temperature, and flooding levels in the auxiliary building pipe chase, the charging pump room, and auxiliary motor driven feedwater pump rooms are not sufficient to cause structural problems nor failure of other essential systems.

Flooding in the motor driven auxiliary feedwater pump room caused by a break in the steam generator blow down line will cause failure of the motor driven pumps. However, functional capability for auxiliary feedwater remains available via the steam driven auxiliary feedwater pump.

- e) A jet impingement shield has been installed to protect the steam system pressure transmitters from a postulated crack in the feedwater line.

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REFERENCES: SECTION 3.6

- 3.6.1-1 WCAP-15628, "Technical Justification for Eliminating Large Primary Loop Pipe Rupture as the Structural Design Basis for the H. B. Robinson Unit 2 Nuclear Power Plant for the License Renewal Program," dated July 2001.
- 3.6.2-1 Letter, NG-73-138, dated June 29, 1973, E. Utley (CP&L) to R. Schemel (NRC), w/enclosure - "Postulated Pipe Failure Analysis Outside of Containment," CP&L, June 29, 1973.
- 3.6.2-2 Letter, NG-73-387, dated September 12, 1973, E. Utley (CP&L) to R. Schemel (AEC).
- 3.6.2-3 Letter, NG-73-594, dated December 21, 1973, N. Bessac (CP&L) to R. Schemel (AEC).
- 3.6.2-4 Letter, NG-74-1295, dated November 1, 1974, E. Utley (CP&L) to K. Goller (AEC).
- 3.6.2-5 WCAP-9558, "Mechanistic Fracture Evaluation of Reactor Coolant Piping Containing a Postulated Circumferential Through-Wall Crack," dated May 1981.
- 3.6.2-6 WCAP-9787, "Tensile and Toughness Properties of Primary Piping Weld Metal For Use In Mechanistic Fracture Evaluation," dated May 1981.
- 3.6.2-7 Generic Letter 84-04, "Safety Evaluation of Westinghouse Topical Report Dealing with Elimination of Postulated Pipe Breaks in PWR Primary Main Loops," dated February 1, 1984.

3.7 SEISMIC DESIGN

3.7.1 SEISMIC INPUT

Earthquake loading was predicated upon a design (operational basis) earthquake and a larger assumed hypothetical earthquake (now known as safe shutdown earthquake, design basis earthquake, and no loss of function earthquake). These earthquakes have horizontal ground accelerations of 0.10g and 0.20g, respectively. A vertical component of two-thirds of the magnitude of the horizontal component was applied simultaneously. Additional information on seismology is contained in Section 2.5.

3.7.2 SEISMIC SYSTEM ANALYSIS

The containment structure and the Reactor Auxiliary Building (RAB), including the control room, were dynamically analyzed using modal analysis and response spectrum techniques to obtain equivalent static loads.

The dynamic analysis was based on the theory of modal superposition. Since it was known that the modal maxima do not occur at the same instant in time, it was the accepted practice to combine the various mode contributions on the square root of the sum of the squares basis as outlined in Reference 3.7.2-1.

The average response spectra shown in Figures 3.7.2-1 and 3.7.2-2 were used for design, with damping factors of 2 percent for the design earthquake (0.1g) and 5 percent for the hypothetical earthquake (0.2g).

The resulting loadings on the structure due to the earthquakes are shown in Figures 3.7.2-3 and 3.7.2-4.

3.7.2.1 Response Spectrum Analysis

The following method of analysis was applied to Class I structures and components, including instrumentation (For instrumentation, see Section 3.10 for an alternate method of design):

- a) The natural period of vibration of the structure or component was 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 design earthquake (0.067g acting in the vertical and 0.1g acting in the horizontal planes simultaneously) were calculated and checked against the limits imposed by the design standard, and
- d) Stresses and deflections resulting from the combined influence of normal loads and the seismic loads due to the assumed hypothetical earthquake (0.133g acting in the vertical and 0.2g acting in the horizontal planes simultaneously) are calculated and checked to verify that deflections do not cause loss of function and that stresses do not produce rupture.

3.7.2.2 Damping Factors

The damping factors shown in Table 3.7.2-1 are those that were used in the design in all cases except that a damping factor of 5 percent of critical was utilized for the reactor containment shell and internal concrete for the design basis (hypothetical, 0.2g) earthquake. These damping factors are at least as conservative as those given in many published papers and reports such as TID-7024, "Nuclear Reactors and Earthquakes." The factored load method of design resulted in stresses in the range of 50 percent of yield for the operational basis (design, 0.1g) earthquake and 90 percent of yield for the design basis (hypothetical, 0.2g) earthquake. The damping factors used in the analysis are conservative for these stress ranges based on the best information available at this time. This includes papers delivered by Dr. N.

Newmark to the International Atomic Energy Agency (IAEA) Panel on "Aseismic Design and Testing of Nuclear Facilities," held in Tokyo, Japan in June 1967, and the World Conference on Earthquake Engineering held in Santiago, Chile, in January, 1969.

The modal analysis was performed utilizing the same damping factors for each mode.

3.7.2.3 Dynamic Characteristics of Structures Inside Containment

The dynamic characteristics of the concrete inner structures are:

a) Periods of vibration (including rocking)

T1 = 0.166 sec

T2 = 0.046 sec

T3 = 0.018 sec

T4 = 0.006 sec

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b) Absolute accelerations

	<u>0.1g Earthquake</u>	<u>0.2g Earthquake</u>
E1 275 (operating floor)	0.3g	0.44g
E1 251 (intermediate floor)	0.14g	0.23g
E1 226 (base mat)	0.1g	0.2g

c) Relative Displacement

	<u>0.1g Earthquake</u>	<u>0.2g Earthquake</u>
E1 275 (operating floor)	0.0007 inches	0.001 inches
E1 251 (intermediate floor)	0.0003 inches	0.00048 inches

The dynamic analysis for the internal concrete structure of the reactor containment was performed by the Modal Analysis Method using average damping factors for all modes of vibration, including rocking. The damping factors used were 2 percent of critical for the operational basis earthquake (OBE) (design, 0.1g) and 5 percent for the design basis earthquake (DBE) (hypothetical, 0.2g).

The concrete inner structure was analyzed as a free standing cantilever beam having no structural connection to the reactor containment above the foundation mat. The effect of the containment structure on the vibration of the concrete inner structure was considered in the multi-mass model by including the mass moment of inertia of the containment with that of the foundation mat to account for its contribution to foundation rocking.

3.7.2.4 Containment Crane

The containment crane was analyzed as a single mass coupled to the multi-mass model of the reactor containment inner structure which serves as its support. The first five mode shapes and the associated periods of vibration were computed utilizing stiffness matrix techniques. The response of each mode of vibration was computed utilizing the methods of modal analysis for the operational response spectra 0.1g and 0.2g design response spectra. Design values of total response were obtained by combining the individual modal contributions on a root-mean square basis. The model used in design is shown in Figures 3.7.2-5 and 3.7.2-6.

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An examination of the mode shapes reveals that the first mode of vibration represents (almost entirely) the vibration of the crane structure above the operating floor. The second and higher modes are primarily the rocking and translational vibration of the reactor containment inner structure with very minor contribution of the crane structure.

Both the trolley and the crane columns are provided with positive stop devices to prevent motion when the crane is not in operation. In addition, rail anchorages are provided to prevent the crane from leaving the rails if uplift should occur. The crane is always in the parked and locked condition when the reactor is in operation.

3.7.2.5 Intake Structure

The intake structure is a Class I structure and as such has been dynamically analyzed in accordance with the procedure described for the containment structure. In addition, the load due to the contained and surrounding water has been computed under seismic conditions in accordance with the procedures in "Nuclear Reactor and Earthquakes" - TID 7024, Chapter 6. The intake structure was designed for this load. The structure was designed in accordance with the stress limits in ACI 318-63 part IVB.

3.7.2.6 Spent Fuel Pit and Superstructure

The spent fuel pit was designed to withstand the anticipated earthquake loading as a Class I structure. A dynamic analysis was performed to determine the dynamic characteristics and responses of the superstructure [Fuel Handling Building (FHB)] and the refueling crane (FHB crane) subjected to ground excitations due to earthquake loading.

The FHB is a Class III structure and was designed for seismic loads in accordance with the Uniform Building Code. The building was also designed to carry a 150 ton crane and all its associated loads. The actual crane installed has a rated hook load of 125 tons and an auxiliary hook rated at 5 tons. Based on the overdesign of the FHB steel for a 150 ton crane and the induced impact to longitudinal loads associated with the design of the crane and its supports, no failure of the FHB superstructure is expected for the design basis (hypothetical, 0.2g) earthquake. The spent fuel cask handling crane can be parked for periods of approximately sixteen hours to facilitate safe maintenance activities provided the activity is worked continuously to completion, the crane is continuously attended, or the crane is returned to its normal storage location if unattended. Engineering approval is required in advance of any anticipated work requiring parking of the crane over the

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spent fuel pit. The FHB crane will not be stored in a position over the spent fuel pit. Maximum loading of this crane would occur during the lifting of the cask with the spent fuel elements.

Hold down lugs have also been provided on both trucks and trolley of the refueling crane to prevent its wheels from lifting from its rails when subject to a vertical earthquake force of 0.133 times the unladen weight of the crane for the lugs on the truck or 0.133 times the deadweight of the trolley for the lugs on the trolley. These lugs were designed so as to cause no interference to crane or trolley motion. The crane is also provided with stops to prevent lateral movements of the crane when in a parked position.

3.7.2.7 Foundation Models

In the course of design, four different foundation models were considered:

1. Fixed Base

$$T1 = 0.151 \text{ sec}$$

2. Rotational spring with stiffness computed from pile test data

$$T1 = 0.2613 \text{ sec}$$

$$T2 = 0.0932 \text{ sec}$$

$$T3 = 0.0490 \text{ sec}$$

3. Rotational spring plus translational spring computed from lateral pile test data, and

$$T1 = 0.2625 \text{ sec}$$

$$T2 = 0.1544 \text{ sec}$$

$$T3 = 0.0573 \text{ sec}$$

4. Rotational spring plus translational spring computed from the properties of the soil mass assuming no contribution from the pile group

$$T1 = 0.3150 \text{ sec}$$

$$T2 = 0.2524 \text{ sec}$$

$$T3 = 0.0601 \text{ sec}$$

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Analysis of the above models led to the conclusion that for case 3 and 4 above an increase in damping would be justified to account for significantly greater energy dissipation in the ground. These analyses resulted in reduced response accelerations throughout the structure. In addition, the range of natural period was such that it would not significantly effect the ultimate response characteristics of equipment and piping which are arranged and supported to be rigidly attached within the structure.

It was decided that a conservative design would result from the model described in the Preliminary Safety Analysis Report (PSAR) which is case 2 above.

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TABLE 3.7.2-1
DAMPING FACTORS

<u>Component</u>	<u>Percent of Critical Damping</u>
Containment Structure	2.0
Concrete Support Structure of Reactor Vessel	2.0
Steel Assemblies:	
(a) Bolted or Riveted	2.5
(b) Welded	1.0
Vital Piping Systems	0.5
Concrete Structures above Ground:	
(a) Shear Wall	5.0
(b) Rigid Frame	5.0

3.7.3 SEISMIC SUBSYSTEM ANALYSIS

3.7.3.1 Reactor Coolant System

Westinghouse provided a dynamic analysis of the reactor coolant loop using WESDYN code in 1970. The analysis is summarized below, with more information given in Reference 3.7.3-1. This analysis was performed using algebraic summation for intramodal responses. In response to IE Bulletin 79-07, the analysis was redone using an updated version of WESDYN which used an absolute sum rather than algebraic summation. The maximum stress determined in the reanalysis was 3.0 ksi as compared to the allowable stress of 13.1 ksi. Documentation of the verification of the WESDYN code is contained in Reference 3.7.3-2.

Some results of the original analysis are compared to those of the revised analysis in Tables 3.7.3-1, 3.7.3-2, and 3.7.3-3.

3.7.3.1.1 Reactor Coolant System Dynamic Analysis

This Section describes the 1970 seismic pipe stress analysis for the reactor coolant loop. The completed analysis shows that the loop piping will experience seismic stresses below allowables and therefore is adequate.

The maximum seismic stress occurs at the crossover leg/reactor coolant pump suction nozzle interface and is 3184 psi which is below the allowable seismic stress.

This section contains:

- a) A description of the system considered, and the mathematical model used
- b) The method of analyses
- c) A detailed description of the computer input constraint assumptions, and
- d) The resultant equipment support constraint loads under seismic conditions.

3.7.3.1.2 Floor Response Spectra

Floor response spectra were generated for specified elevations of the containment, the containment internal structure, the Auxiliary Building, and the Class I bay of the Turbine Building. These are shown in Reference 3.7.3-1. These floor response spectra were generated by exciting the multidegree of freedom dynamic model of the buildings with the normalized time history forcing function acceleration which gave a ground response spectrum acceleration at least as large as the response spectrum acceleration defined in Figure 3.7.2-1 of the Updated FSAR. The time history accelerations at each mass point of the building models were then used to construct a floor response spectrum for a one degree of freedom system which represented the maximum response of the equipment located at the mass point which represented the building elevation under consideration.

3.7.3.1.3 Reactor Coolant System Thermal and Seismic Analysis

The seismic analysis was performed on the reactor coolant loop which consists of the reactor vessel, steam generator, reactor coolant pump, the pipe connecting these components, and the large component supports. The components and piping were modeled as a system of lumped masses connected by springs whose values were computed from elastic properties inputs. A simplified support model was arrived at by representing the structural support system as equivalent springs rather than as member beams and columns.

The analysis was performed using a proprietary computer code called WESTDYN. As input, the code uses system geometry, inertia values, member sectional properties, elastic characteristics, support and restraint characteristics, and the appropriate DEP seismic floor response spectrum for 0.5 percent critical damping. Both horizontal and vertical components of the seismic response spectrum were applied simultaneously. The seismic shock spectra were applied simultaneously along the Y and Z axis. Previous analysis indicated the Z direction to be the most critical horizontal direction for maximum pipe stress.

With this input data, the overall stiffness matrix $[K]$ of the three dimensional piping system was generated (including translation and rotational stiffnesses). Zero rows and columns representing restraints were deleted, and the stiffness matrix was inverted to give the flexibility matrix $[F]$ of the system.

$$[F] = [K]^{-1}$$

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A product matrix is formed by the multiplication of the flexibility and mass matrices. This product matrix forms the dynamic matrix, [D], from which the modal matrix is computed.

$$[D] = [F] [M]$$

The eigenvalues and eigenvectors representing the frequency and associated mode shape for each mode were generated using a modified Jacobi method.

$$(w^2 [M] - [K]) [X] = 0$$

From this information, the modal participation factor was combined with mode shapes and the appropriate seismic response spectrum values to give the structural response for each mode. Then the forces, moments, deflections, rotations, constraint reactions, and stresses are calculated for each significant mode. The modal stresses were then summed by the square root of the sum of the square method for each significant point in the system to determine the total stress.

The restraints, supports, and other constraints assumed for input into the seismic computer model are given below (See Figure 3.7.3-1 for axis orientation):

- a) Reactor Vessel - The vessel is rigid
- b) Steam Generator - The steam generator at the upper support point is permitted to translate along and rotate about the X, Y, and Z axis, but translations along X and Z are resisted by springs representing the upper support. The steam generator at the lower support point is permitted to translate along and rotate about the X, Y, and Z axis, but all movements are resisted by springs representing the lower supports' stiffness, and
- c) Reactor Coolant Pump - The reactor coolant pump is permitted to translate along and rotate about the X, Y, and Z axis, but all movements are resisted by springs representing the supports' stiffness.

3.7.3.2 Class I Piping Other Than Reactor Coolant

The following is a general description of the design approach used for Class I piping other than reactor coolant. It is excerpted from Reference 3.7.3-1, which contains additional information.

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In response to IE Bulletin No. 79-14 "Seismic Analysis for As-Built Safety-Related Piping Systems," a dynamic analysis was performed on some safety-related piping systems. The approach used is discussed in Section 3.7.3.2.9.

3.7.3.2.1 Method of Arriving at Static Coefficients

Class I piping is housed by and supported from the containment inner structure, the reactor containment structure (supports at penetrations only), the Class I bay of the Turbine Building, and the Reactor Auxiliary Building. A dynamic analysis was performed on these structures to determine the expected accelerations at various levels within the structure for both the operating basis and design earthquakes. Details of the dynamic analysis of Class I structures are contained in Sections 3.7.1 and 3.7.2. In arriving at the static coefficients used for the design of piping systems, the horizontal acceleration values at the various levels of the structures were amplified by a ratio equal to the peak ground response for 0.5 percent damping as shown on the Housner curves divided by the ground acceleration. In this case the peak ground response is 0.96g for the 0.2 ground acceleration earthquake so that all building response accelerations were multiplied by a factor of 4.8.

3.7.3.2.2 Range of Static Coefficients for the Various Buildings

The seismic response of the structures was amplified by a factor of 4.8 for the static coefficients that were applied to the piping systems. For the DBE (0.2g ground acceleration) these amplified values of horizontal accelerations ranged as follows for the various structures:

a) Reactor Containment Structure

0.96g at E1 226
3.00g at E1 413

b) Containment Inner Structure

0.96g at E1 226
2.15g at E1 275

c) Reactor Auxiliary Building

0.96g E1 226
1.65g E1 262

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d) Turbine Building - Class I Bay

0.96g at E1 226

1.85g at E1 266

In all cases, the vertical acceleration applied to the piping was assumed to be two-thirds of the peak ground response acceleration for 0.5 percent damping, i.e., 0.32g for the OBE and .64g for the DBE.

3.7.3.2.3 Computer Method of Static Analysis

For Class I systems analyzed by this method, an isometric sketch was made from the piping drawings with mass points located at every tee and at the intersection of straight pieces and elbows. In addition, straight pieces were divided into as many mass points as necessary to assure adequate distribution of loading between support points, with spans between mass points rarely exceeding ten foot lengths. This method accounted for the weight effects of valves as an additional load in the piping system for which stress values were calculated and restraints were placed as required. The valve loads were treated as concentrated loads acting on the centerline of the piping. A thermal analysis was then performed and after examining thermal displacement throughout the system, single or multi-directional restraints were preliminarily located and the static analysis was performed.

The static analysis program used the displacement method in which the equations of equilibrium at the junctions were solved simultaneously to obtain junction displacements, after which reactions at every point in the system were calculated. Input data for the program included weight per foot of pipe, fluid and insulation, weight of concentrated loads such as valves, static coefficients for the horizontal and vertical directions, and the location and direction of restraints. The computer output included forces, moments, and stresses (as per the Pressure Piping Code, USAS B31.1) at all points, displacements at all points, and forces on restraints and anchors.

The program multiplied the weight per foot of piping by the static coefficient and used this adjusted weight unit to determine the virtual weight of each piece of pipe or concentrated load. Half the weight of each member was lumped at the end points and concentrated loads added as applicable. The program then calculated two sets of reactions for the system with loadings applied simultaneously first in one horizontal direction (X) and the vertical direction (Y), then in the other horizontal direction (Z) and the vertical direction (Y).

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For each system, the results of the two combined horizontal-vertical analyses were evaluated and if the stress values were found excessive, restraints were added or relocated and the system reanalyzed both statically and thermally. Only in extreme cases where thermal considerations would not allow the use of positive restraints were hydraulic snubbers specified.

3.7.3.2.4 Simplified Method of Static Analysis

This method was used for the static analysis of relatively cold systems where thermal effects, although not ignored, were not a critical factor in the placement of restraints for seismic protection. The design basis was that restraints and supports be placed at close enough intervals so that seismic stresses would not exceed 5000 psi for the OBE and 10,000 psi for the DBE.

For every size and schedule of pipes, filled with water and empty, and for the appropriate g loadings, the maximum permissible span of straight, uniform pipe was established. These spans were determined on the conservative basis that supports act as pinned end joints for a uniformly loaded, simply supported beam rather than for a continuous beam. It is recognized that in the usual configuration of piping for large systems there are many deviations from the basic straight, uniform run of piping that must be considered in determining permissible span lengths. The effects of concentrated loads, branch connections, changes in pipe size, stress intensification factors, changes of direction, offsets, and various combinations of these effects were studied in detail. Correction factors were determined for each of these deviations which resulted in shortening the permissible span between restraint points for maintaining stress values within the preset limits.

3.7.3.2.5 Systems Analyzed by Computer Method and Simplified Method

Seismic response for Class I piping systems (except reactor coolant) was originally determined by static analysis. Systems that could not be analyzed by the simplified method were analyzed by the computer method. In response to IE Bulletin No. 79-14, some systems have been analyzed by a dynamic computer method. Systems were analyzed per the following categories:

- a) Computer Analysis
 - 1) Piping 2 in. and larger with temperature in excess of 212°F
 - 2) Piping with temperature less than 212°F with significant movement due to expansion of equipment at connection nozzles
 - 3) Piping with temperature less than 212°F located in the upper region of the reactor containment structure, and

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- 4) Piping with temperature less than 212°F connected to piping of higher temperature that could not be logically analyzed except as part of the main system.

b) Simplified Analysis

For the most part, systems analyzed by this method were those of lesser anticipated stress levels consisting of piping 212°F or less and with relatively low seismic loadings. Systems, other than those analyzed by the computer method, 1.50 in. and over were analyzed by the simplified method.

Piping 1.25 in. and smaller that were field run (drains, vents, test lines, and instrumentation lines) were seismically protected by specifying the maximum spacing between restraints such that stress values for straight runs of pipe would not exceed 10,000 psi for the 0.2g ground acceleration earthquake.

Although thermal expansion stress values are not identified in this report, it should be noted that, in all cases, thermal expansion stresses are within the S_a (or S_e) allowable of the Code for Pressure Piping, USAS B31.1; and in all cases, the sum of stresses due to internal pressure, weight and seismic loading for the OBE is within $1.2 S_h$ (allowable stress in hot condition).

c) Dynamic Computer Analysis

Piping may also be analyzed by a computerized dynamic analysis. The dynamic analysis more closely models actual seismic response of the piping system. Information pertaining to dynamic analysis methods is supplied in Section 3.7.3.2.9.

3.7.3.2.6 Seismic Amplification Factors and Building Frequency Factors

As part of the resonance evaluation, seismic acceleration loading amplification factors were developed such that an analysis could be performed to determine the effects on piping should the piping be in resonance with the building. The amplification factor (K) was determined as follows:

$$K = \frac{b}{a}$$

the Class I piping is where: b = peak acceleration at the floor where
located from floor response spectra (see Table 3.7.3-4).

a = Acceleration used in the piping analysis - Based on 0.1g for OBE.

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For the evaluation, the amplification factor for horizontal accelerations ranged as follows:

a) Reactor Containment Structure

1.81 at E1 226
1.79 at E1 401

b) Reactor Inner Structure

1.81 at E1 226
0.67 at E1 275

c) Reactor Auxiliary Building, and

1.81 at E1 226
2.74 at E1 262

d) Turbine Building - Class I Bay

2.20 at E1 226
9.00 at E1 262

In addition, the fundamental frequency of each structure was found for the purpose of determining the potential for resonance between piping and the structures.

3.7.3.2.7 Piping Evaluation

3.7.3.2.7.1 Systems Analyzed by Computer Method

The procedure used for evaluating the stress values for computer analyzed systems was as follows:

- a) For each system the various pipe sizes and wall thicknesses were listed in a table and identified by point numbers on the isometric sketches used for the analysis
- b) The internal pressure and weight stresses were taken from the original evaluation and listed
- c) The total allowable primary stress equal to $1.8 S_m$ or yield was determined and the pressure and weight stresses were subtracted from the total allowable, the remainder thus being the stress value

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available for seismic loading

- d) The K value for each point listed was determined by dividing the maximum acceleration of the appropriate floor response spectrum by the acceleration value used in the analysis. This ratio was then multiplied by the 1.3 modal participation factor
- e) The higher of either the static X-Y or Y-Z stresses was multiplied by 1.3K, which represents the maximum potential stress value should the piping be in resonance with the supporting structure, and
- f) The stress available for seismic loading (Item c) was divided by the stress value for potential resonance (Item e), resulting in a "design evaluation factor". The "design evaluation factor" is a measure of design conservatism and indicates that if the factor is larger than 1.0 the piping does meet evaluation criteria.

Work sheets were compiled as per the above described procedure for every pipe size for each of the systems analyzed by the computer method. Piping systems, except for that piping contained in the Class I turbine bay, were analyzed by the procedure and showed stress values which met acceptance stress criterion.

In the Class I turbine bay, a frequency evaluation was performed in lieu of a stress evaluation. The frequency evaluation consisted of determining the natural frequency of the piping (i.e., fundamental frequency) and comparing this value to the building's fundamental ranges. The frequency analysis was performed by a computer method which provides the fundamental frequency of the system and normalized mode shape.

This evaluation showed that of the Class I systems located above ground level the following piping systems had fundamental frequencies which fell within the building frequency band:

- a) Steam Driven auxiliary feedwater pump Discharge to Main Feed System
- b) Feedwater Regulator Valve Bypass #1, #2, #3, S/G

Restraints were added to these systems to change their fundamental frequency such that it fell outside the building fundamental frequency band.

3.7.3.2.7.2 Systems Analyzed by the Simplified Method

As described above, the design basis for systems analyzed by the simplified method was that restraints and supports were placed at close enough intervals

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so that seismic stresses would not exceed 5000 psi for the OBE and 10,000 psi for the DBE. An evaluation was performed for this method in a manner similar to that described in Section 3.2.3.2.7.1.

3.7.3.2.8 Valve Evaluation - Effects Of Valve Operators

For systems originally analyzed by the computer method, the stress levels at all applicable valve locations were increased as described in Reference 3.7.3-1 to account for the effect of the offset mass of the valve operators. Thus, by including the total stress (i.e., stress due to valve operators and normal piping stresses) in the evaluation described in Section 3.7.3.2.7, the potential effect of resonance was determined.

For systems analyzed by the simplified method, an overall evaluation was performed utilizing the information from Reference 3.7.3-1. For this evaluation the worst stress condition (i.e., for a 2 in. valve) was assumed to apply in all cases.

As a result of the investigation of the effect on Class I piping due to valve operator offset weight (Reference 3.7.3-1) it was concluded that supports were required for the operators on valves less than 2 in. diameter. Therefore, supports were provided to meet the following criteria:

- a) Supports were provided at the valve body or operator in such a manner that the offset weight effect is essentially eliminated
- b) The support accepts all valve operator load when considering the appropriate seismic loading at the valve location.

3.7.3.2.9 Analysis Methods Used During IE Bulletin 79-14 Closeout

During the resolution of NRC IE Bulletin 79-14, an OBE static and a DBE dynamic analysis was performed on the safety-related piping systems. The OBE static analysis followed the procedure outlined in Section 3.7.3.2.7. The DBE dynamic analysis incorporated the following techniques:

- a) The DBE response spectrum curves (.5 percent damping) were broadened plus and minus ten percent.
- b) The inclusion of closely spaced modes followed the guidelines of NRC Regulatory Guide 1.92.
- c) The cutoff frequency was 33 Hertz.

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- d) The participation of mass in the rigid range was included.
- e) A 3D earthquake was formed using the SRSS method.
- f) The vertical response spectrum was taken as 2/3 of the building ground response curve.
- g) In some cases multi-level excitation (different response spectra for different restraints) was used.

The use of dynamic seismic analysis instead of the original static seismic analysis more closely models the actual seismic response of the pipe so the loads are more representative. The dynamic analysis methods used for the IE Bulletin Program represent techniques accepted by the NRC and generally used in the nuclear industry.

3.7.3.3 Class I Equipment

The purpose of this section is to identify the procedure used in the seismic design of Class I equipment to assure that the seismic requirements were met (See Section 3.10 for an alternate method of seismic design).

3.7.3.3.1 Seismic Criteria

Seismic requirements and design adequacy were determined as follows:

- a) The horizontal seismic accelerations used were equal to or greater than the accelerations that occur at the equipment location (i.e., at the proper building elevation) as determined from the building dynamic analysis for the DBE
- b) The vertical seismic accelerations used were 2/3 of the value selected for horizontal acceleration
- c) The vertical and horizontal accelerations were assumed to act simultaneously
- d) The relative stiffness of the equipment and its support was evaluated based on past experience with similar types of equipment. If the equipment was relatively rigid with a fundamental frequency above 15-20 cps then seismic design was performed by using a seismic g loading applied at the center of gravity of the equipment. This g loading corresponded to the combined mode g loading at the elevation of the building on which the equipment was supported. If the equipment was

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flexible, then a rationed g is applied at the center of gravity of the equipment. This g loading included potential response of the equipment to building motion and the effects of both building and equipment damping.

3.7.3.3.2 Seismic Evaluation

Class I equipment (flexible and rigid) have been evaluated to assure functional adequacy when considering potential equipment resonance with the building during earthquake conditions.

Electrical racks, panels, controls boards, etc. fall in the category of flexible equipment. This equipment is located in the Auxiliary Building at or below Elevation 258 ft. From Figures 2.1.0-10 and 2.1.0-11 of Reference 3.7.3-1, the peak acceleration that a one-degree-of-freedom system in resonance with the building would experience is at Elevation 258 ft. This peak acceleration is 2.0g for 0.2g ground acceleration with 0.5 percent of critical damping. For the minimum damping anticipated in this type of equipment, i.e., 1 percent of critical value, the peak acceleration is reduced to about 1.6g. This is the maximum equivalent static load that should be used to account for both floor acceleration and response spectra distortion.

When the equipment supports were designed, the equivalent static load was selected by accounting only for the floor acceleration. This means that a load of $(0.30g/0.20g) \times 0.69g = 1.05g$ was selected for equipment at or below Elevation 258 ft. The design stresses were 2/3 of the material yield, e.g., 24,000 psi. Hence, the correct load of 1.6g would cause a maximum stress of $24,000 \times (1.6g/1.05g) = 36,500$ psi. The ultimate stress of this type of material is of the order of 70,000 psi. This gives a margin of 33,500 psi between the ultimate capacity and the maximum expected stresses.

For equipment considered as relatively rigid (i.e. having a fundamental frequency above 15-20 cps) seismic design was performed using a seismic g loading applied at the equipment center of gravity. This is discussed in Section 3.7.3.3.1 above.

In light of the above, it can be concluded that Class I equipment will maintain both structural and functional integrity for the earthquake loadings associated with 0.2g ground acceleration.

3.7.3.3.3 Fan Coolers

The seismic analysis of the fan cooler was conducted in two parts.

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- a) Analysis of the structural steel enclosure of the cooling units to include the effect of supported equipment
- b) Analysis of the fan-motor system and their foundation.

The fan cooler enclosure is shown in plan view in Figure 3.7.3-2. The maximum hypothetical earthquake seismic load stresses were determined on the basis of a 0.67g horizontal acceleration and 0.45g vertical acceleration taken simultaneously with inertial loading assumed uniformly distributed along the supporting member.

These values exceed the maximum of the ground response spectrum for 2 percent damping defined for the site and represent the combined mode total horizontal acceleration of the supporting operating deck.

Two seismic analyses were performed as shown in Figure 3.7.3-3. The resultant maximum stresses for dead-load-plus-seismic for each frame member are shown in Table 3.7.3-5.

The maximum combined stress of 18.0 ksi was determined at the beam to column joint of Frame 1. This value represents a safety factor of two for combined dead-plus-seismic with respect to outermost fiber local yield of the structure.

The fan-motor and supporting structure system have been analyzed dynamically as a lumped parameter system. The model used is shown in Figure 3.7.3-4. The only vibrational mode having a frequency below 40 cps is vertical vibration of the anchorage system.

This frequency is approximately 5 cps and is the result of spring mounting of the unit to eliminate operational vibration. This containment internal structure was not analyzed for vertical modes of vibration but experience on other containment internal structures indicates fundamental vertical frequency modes well in excess of 5 cps. Hence no potential resonance with the structure was considered.

This frequency response to the vertical component of the ground response spectrum for 2 percent damping resulted in a 0.3 loading. The seismic input to the fan cooler system was therefore 0.67g horizontal and 0.45g vertical as verified by dynamic analysis.

The failure modes considered for the motor unit are excess deflection of the rotor shaft (which results in rubbing against the housing) and bearing failure. The failure modes for the fan are failure of the fan shaft support

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bearings and deflection of the fan housing and fan to cause binding. In addition, the potential for shear and overturning failure of the motor-fan assembly at the foundation anchorage was investigated. The results of these failure mode analyses are shown in Table 3.7.3-6. Details of the foundation attachment are shown in Figure 3.7.3-5. In addition, the potential for shear and overturning failure of the motor-fan assembly at the foundation anchorage was investigated.

Based on the seismic analysis performed as outlined above it may be concluded the fan cooler units in the containment are adequately designed to resist the seismic loading defined for the site.

3.7.3.3.4 Tanks

The condensate storage, refueling water, primary water storage, and diesel oil storage tanks which are ground supported were designed as Class I Structures using the following seismic criteria:

- a) For a ground acceleration of 0.1g horizontally coincident with a vertical acceleration of .067g, stress limits when combined with normal dead and live load stresses were taken as 1.33 times allowable stress, and
- b) For a ground acceleration of 0.2g horizontally coincident with a vertical acceleration of 0.133, the allowable stress limits when combined with normal dead and live load stresses were taken as 95 percent of yield in tension and 90 percent of yield in compression and shear.

The tanks were dynamically analyzed using values from Housner's "Dynamic Behavior of Water Tanks." A program was developed based on Atomic Energy Commission (AEC) bulletin TID-7024 "Nuclear Reactor and Earthquake - Chapter VI - Dynamic Pressure on Fluid Containment" which includes Housner's formula for dynamic behavior of water tanks. The analysis took into account wave depth, spectrum velocity, seismic and static load. It considered the tank as 5 separate rings with stresses computed at each ring height.

During the resolution of USI A-40, "Seismic Design Criteria," the method of analysis of above ground, flexible, vertical tanks was identified as a topic requiring technical resolution. USI A-40 is resolved in Standard Review Plan (SRP) Sections 2.5.2, 3.7.1, 3.7.2, and 3.7.3 (Revision 2, August 1989). The guidelines related to seismic analysis of the above-ground vertical tanks are included in SRP Section 3.7.3.11.14. As part of the resolution of USI A-40, tanks at nuclear power plants were required to have confirmatory checks to

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ensure that the safety-related, above-ground, vertical tanks are adequately designed. H. B. Robinson committed to make confirmatory checks using the procedures developed by SQUG, under the resolution of the USI A-46 program. The implementation of criteria and procedures described in GIP-2, supplemented by the staff evaluations described in Supplement No. 1 to Generic Letter 87-02, for large, flat-bottom, cylindrical, vertical tanks which are needed for safe shutdown and refueling water storage in PWRs, is considered an acceptable method for resolving the seismic issues related to these types of tanks for both USI A-46 and USI A-40, as it applies to USI A-46 plants. The replacement of existing tanks and heat exchangers and the design and construction of new tanks and heat exchangers is to follow the guidelines provided in GIP-3 [3.7.3-5], except for new flat-bottom vertical tanks. New flat-bottom v vertical tanks must meet additional attributes given in Implementation Guidelines for Seismic Qualifications of New and Replacement Equipment/Parts (NARE) Using Generic Implementation Procedure (GIP) [3.7.3-6].

3.7.3.4 Lateral Core Seismic Analysis

Reference 3.7.3-3 (proprietary) describes the seismic analysis of the Cycle 4 core (Exxon Fuel). The objective of the analysis was to determine fuel rod stresses, guide tube stresses, and interactive grid spacer loads during a lateral core motion seismic event. Fuel rod integrity, core coolable geometry, and the ability to insert control rods, must be maintained during this event.

The results of the analysis show that fuel rod, grid spacer, and guide tube integrity is maintained during a 0.4g lateral seismic event. Maximum guide tube, fuel rod, and grid spacer stresses occur in the fuel assemblies adjacent to the core boundary.

A dynamic test of the fuel bundle, a static load-deflection grid spacer test and dynamic spacer tests were performed to determine input fuel assembly dynamic properties. These also provide stress-strain information on the guide tubes, spacers, and fuel rods which verify the structural adequacy of the fuel assembly during a lateral seismic event.

3.7.3.5 High-Density Spent Fuel Racks

The purpose of the seismic and stress analysis was to analyze the high-density spent fuel rack module under various loading conditions. The racks were evaluated for both operating basis earthquake (OBE) and safe shutdown earthquake (SSE) conditions and meet Seismic Category I requirements (Reference 3.7.3-4). A detailed stress analysis was performed to verify the acceptability of the critical load components and paths under normal and

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faulted conditions. The racks, which rest freely on the pool floor, were evaluated to ensure that under various loading conditions they do not impact each other, nor do they impact the pool walls.

The dynamic response of the fuel rack assembly during a seismic event is the condition which produces the governing loads and stresses on the building structure. The dynamic response, internal stresses, and loads were obtained from a seismic analysis which was performed in two phases. The first phase was a time history analysis on a simplified nonlinear finite element model. The second phase was a response spectrum analysis of a detailed rack assembly finite element model. The damping values used in the seismic analysis are two percent damping for OBE and four percent damping for SSE as specified in NRC Regulatory Guide 1.61.

The simplified nonlinear finite element model was used to determine the fuel rack response for full, partially filled, and empty fuel assembly loading conditions. This nonlinear model has the structural characteristics of a submerged rack assembly. The nonlinearities of the fuel rack assembly which are accounted for in the model are due to changes in the gap between the fuel cell and the fuel assembly, the boundary conditions of the fuel rack support locations and energy losses at the support locations.

The WECAN computer program was used to determine the nonlinear time history response of the fuel assembly/fuel rack system. The effective fuel mass, fuel assembly to cell impact loads, and overall rack response was obtained from the nonlinear time history results.

The detail model was a three-dimensional finite element representative of a rack assembly consisting of discrete three-dimensional beams interconnected at a finite number of nodal points. The results of the nonlinear time history model were incorporated in the detail model. Since the detail model did not account for the nonlinear effect of a fuel assembly impacting the cell and the hydrodynamic restoring force, the internal loads and stresses for the rack assembly obtained from this model were corrected by load correction factors. The load correction factor was derived from the nonlinear model results and was applied to the components in the stress analysis. The responses of the model from accelerations in three directions were combined by the SRSS method in the stress analysis. The loads in the four major components (support plate/leveling pad assembly, bottom grid, top grid, and fuel cell) were examined, and the maximum loaded section of each of these components was found. These maximum loads from the detail model were used in the stress analysis to obtain the stresses within the rack assembly, as discussed in Section 3.8.4.1.2.

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TABLE 3.7.3-1

COMPARISON OF PRIMARY COOLANT LOOP
PIPE STRESSES (SEISMIC ONLY)

LOCATION	STRESSES (ksi)	
	ORIGINAL SEISMIC	REVISED* SEISMIC
RPV Outlet	0.1	0.1
SG Inlet Elbow	0.2	0.2
SG Inlet	0.2	0.2
SG Outlet	1.6	1.6
SG Outlet Elbow	2.1	2.1
Crossover Leg Elbow (SG Side)	0.8	0.8
RCP Inlet	3.1	3.0
RCP Outlet	2.9	2.9
Cold Leg Elbow	1.1	1.2
RPV Inlet	1.1	1.1
Allowable Stress: 13 ksi		

*Based on IE Bulletin 79-07

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TABLE 3.7.3-2

COMPARISON OF PRIMARY COOLANT LOOP
SUPPORT LOADS (SEISMIC ONLY)

Note: Loads generated in this table are from one of two 2-Directional Shocks Only.

		(kip)			(in-kip)		
Support	Analysis	Fx	Fy	Fz	Mx	My	Mz
**SG Lower:	Original:	0	106	115	5069	2064	2253
	Revised:	0	123	116	5087	2052	2234
* Support stress: 4.0 ksi							
Yield stress: 40 ksi							
**SG Upper	Original:	4	0	204	0	0	0
	Revised:	4	0	204	0	0	0
* Support stress: 4.0 ksi							
Yield stress: 40 ksi							
RCP:	Original:	31	83	131	29	974	680
	Revised:	30	83	131	29	971	715
* Support stress: 4.8 ksi							
Yield stress: 40 ksi							
RPV:	Original:	26	33	3	2447	870	4672
	Revised:	26	33	3	2422	862	4690
* Support stress: 2.0 ksi							
Yield stress: 40 ksi							

* Deadweight and seismic loads combined.

**The steam generator lower assemblies were replaced in 1984 with the new weight of the steam generators (dry weight = 585,200 lbs.) being 96% of the weight of the old steam generators (dry weight = 610,000 lbs.). Therefore, the values for the coolant loop support loads provided in Table 3.7.3-2 above are conservative with respect to the replacement of the lower assemblies.

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TABLE 3.7.3-3

COMPARISON OF PRIMARY COOLANT LOOP
EQUIPMENT NOZZLE LOADS (SEISMIC ONLY)

Nozzle	Analysis	(kip)		(in-kip)			
		Fx	Fy	Fx	Mx	My	Mx
RPVON	Original:	30.30	1.10	1.79	21.66	109.6	70.5
	Revised:	30.33	1.12	1.82	22.7	111.3	72.0
* Revised Analysis Total							
Stress:		15.6 ksi					
Allowable Stress:			20.04 ksi				
SGIN	Original:	30.21	1.04	2.11	39.3	135.1	438.6
	Revised:	30.22	1.04	2.15	40.3	135.0	439.1
* Revised Analysis Total							
Stress:		14.9 ksi					
Allowable Stress:			22.44 ksi				
SGON	Original:	30.66	44.46	66.39	4305.0	265.7	1312.9
	Revised:	30.59	44.34	66.38	4303.9	274.8	1308.7
* Revised Analysis Total							
Stress:		15.4 ksi					
Allowable Stress:			22.44 ksi				
RCPIN	Original:	30.82	44.55	62.05	6034.1	914.2	3114.1
	Revised:	30.75	44.43	62.04	6017.9	912.2	3107.4
* Revised Analysis Total							
Stress:		15.6 ksi					
Allowable Stress:			22.44 ksi				
RCPON	Original:	4.85	31.20	2.75	3458.8	96.0	2831.7
	Revised:	4.85	31.18	2.76	3480.7	94.0	2808.6
* Revised Analysis Total							
Stress:		17.7 ksi					
Allowable Stress:			22.44 ksi				

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TABLE 3.7.3-3 (Cont'd)

COMPARISON OF PRIMARY COOLANT LOOP
EQUIPMENT NOZZLE LOADS (SEISMIC ONLY)

Nozzle	Analysis	(kip)		(in-kip)			
		Fx	Fy	Fx	Mx	My	Mx
RPVIN	Original:	7.0	31.5	2.68	399.3	222.7	2089.5
	Revised:	7.0	31.5	2.87	422.7	219.5	2109.6

* Revised Analysis Total

Stress: 15.3 ksi

Allowable Stress: 20.04 ksi

* Pressure, deadweight, and seismic loads combined

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TABLE 3.7.3-4

FLOOR RESPONSE ACCELERATIONS

Acceleration time history functions producing ground response curves used.

The response accelerations are for a normalized 0.2g earthquake and .50 percent piping damping.

Description	Maximum g Load	Building Damping
Auxiliary Building *		5%
Ground E1 226'	0.87	
E1 246.0'	1.51	
E1 262.0'	2.19	
Turbine Building - Class I Bay *		2%
Ground E1 224'	1.06	
E1 242.0'	6.67	
E1 262.0'	8.52	
E1 279.5'	12.90	
Reactor Containment		5%
Ground E1 226'	0.870	
E1 289'	1.34	
E1 352'	2.38	
E1 401'	3.40	
Reactor Containment Inner Structure		5%
Ground E1 226'	0.870	
E1 232.5'	0.89	
E1 248.0'	0.91	
E1 272.5'	0.96	
E1 296.6'	2.62	

*Analyses for both directions of the building were performed and only maximum values are reported.

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TABLE 3.7.3-5

SEISMIC STRESSES AND LOADS
ON FAN COOLERS

Frame	Beam					Column					(Vertical)	↓ (Vertical)	↑
	Pmax Location	Vmax	Mmax Pmax	Jmax	Member	Location	Pmax	Vmax	Mmax	Jmax			
1	0.2	6.0	29.0	18.0	2(6[18.0])	Joint	10.2 -0.6	27.7	9.0	2(9[25.4])	Joint	-3.4	
2		SMALL		<8.7	6WVF20	Joint	SMALL		<8.7	8WVF40	Joint	-2.0	
3	0.1	4.2	19.4	<11.9	10[28.3]	Joint	4.5 -3.2	19.4	<11.9	9[25.4 or 13[31.8]	Joint	-4.0	
4	0.1	4.2	19.4	<11.9	2(9[25.4])	Joint	4.5 -3.2	19.4	<11.9	2(9[25.4])	Joint	-3.3	
5		SMALL			6[18.0]			SMALL		8WVF40		-2.6	
6		SMALL			6WVF25			SMALL		8WVF40		-2.8	

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TABLE 3.7.3-6

SEISMIC ANALYSIS OF FAN MOTOR SYSTEM

Failure Mode	Description	Calculated Behavior	Limit Behavior	Factor of Safety
1	Deflection of the Motor Rotor and Shaft to cause rubbing against housing	.0063 in	0.30 in	4.8
2	Radial Load on Motor Bearing	1210 lb	22300 lb	18.4
3	Thrust Load on Motor Bearing (Parallel to Shaft) Including potential input load in accelerating through 0.1 in gap	2121 lb	10000 lb	4.7
4	Combined Deflection of Fan Wheel and of Fan Housing to cause rubbing	0.190 in	1.625 in	8.6
5	Radial Load on Fan Bearings	2100 lb	106,000 lb	50.5
6	Thrust Load on Fan Bearings	480 lb	2000 lb	4.2
7	Failure of Anchor bolts connecting upper support frame to lower support frame Shear Tension (overturning)	1.33 ksi 0.60 ksi	21.6 ksi 36.0 ksi	16.3 60.0
8	Failure of Welds connecting lower support frame to floor plate embedments Shear Overturning	1.97 ksi 0.32 ksi	21.6 ksi 21.6 ksi	11.0 68.0

Note: All calculations base on: horizontal seismic acceleration of 0.45g
vertical seismic acceleration of 0.30g
Dead Load

3.7.4 SEISMIC INSTRUMENTATION

One strong motion accelerograph (monitor) is located adjacent to the Containment Building and on the containment foundation mat. The other is located south of the Unit 2 waste retention basin, which places it in an area relatively free of interference reflections. Both monitors have a starter in the vertical, longitudinal, and transverse directions with a setpoint of 0.01 g.

Routine maintenance of these instruments is provided by the Instrumentation and Control Group of the plant operations organization in accordance with the supplier's instruction manual. Each instrument is equipped with test and calibrate switches enabling a check of its proper operation.

The strong motion accelerographs sound an alarm in the control room when they actuate. On receipt of an alarm, the control room operator would assess plant parameters, i.e., flows, temperatures, pressures, and reactivity, to detect any change in conditions. A check of control rod operability would also be performed. The control room observation check would be followed by a walk-through inspection to detect any signs of visual damage. If plant conditions remained normal, operation would be resumed upon approval of the plant superintendent. Someone will be dispatched by the control room operator to check the monitors and retrieve the recorded accelerograms in the event of a trigger.

If the recorded accelerograms indicated values below the level of the OBE of 0.1g horizontal, 0.067g vertical then no further action would be taken.

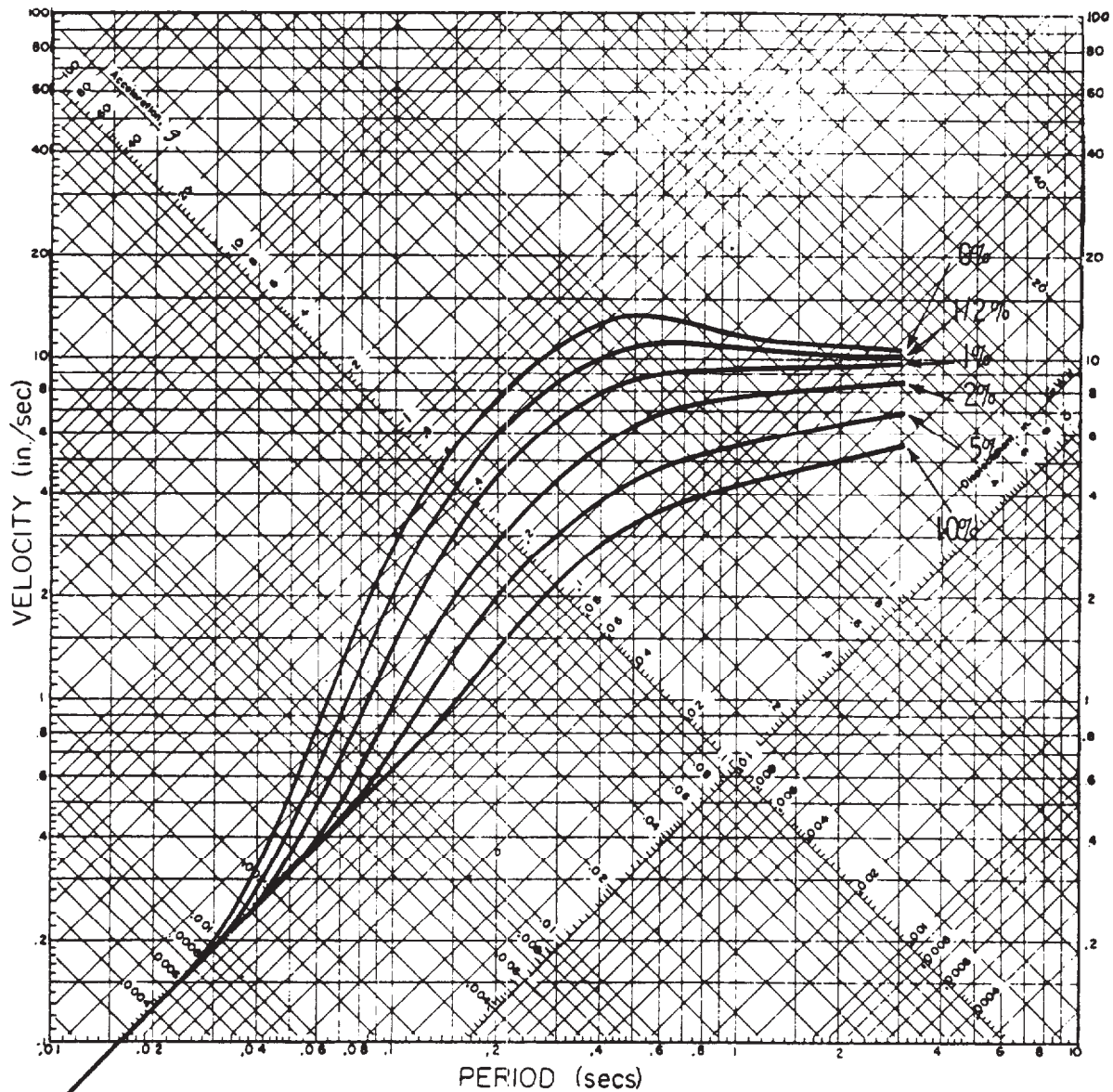
When the strong-motion accelerograph indicates that the operating basis earthquake has been exceeded, the reactor shall be shut down and shall remain shut down until inspection of the facility shows that no damage has been incurred which would jeopardize safe operation of the facility or until such damage is repaired.

The reactor facility was designed such, that, for ground motion less than the operating basis earthquake, those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public will remain functional. Any ground motion in excess of this results in an uncertainty as to the extent of the damage which must be resolved before continued operation can be considered safe. The requirement for shutdown and inspection after an earthquake exceeding the OBE is consistent with the requirements of Appendix A, 10 CFR 100.

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REFERENCES: SECTION 3.7

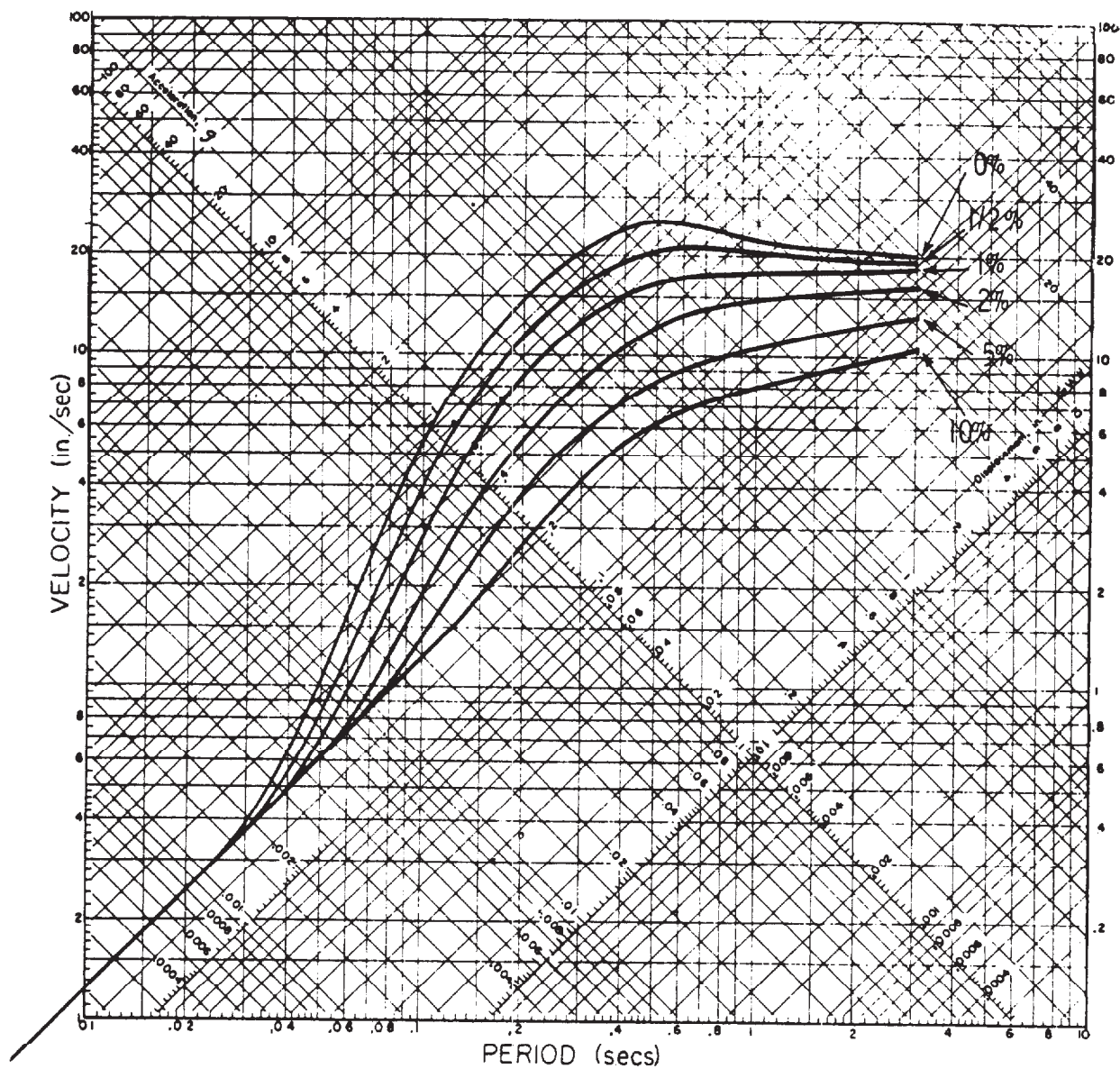
- 3.7.2-1 R. W. Clough, Earthquake Analysis by Response Spectrum Superposition, Seismological Society Bulletin, No. 3 Vol. 52, July 1962.
- 3.7.3-1 H. B. Robinson Unit 2, Additional information concerning Seismic Analysis of Class I Piping and Equipment, Docket 50261-35, June 1970.
- 3.7.3-2 Documentation of Selected Westinghouse Structural Analysis Computer Codes, WCAP-8252 Rev 1, May 1977.
- 3.7.3-3 Lateral Core Seismic Analysis for Exxon Nuclear 15x15 Reload Fuel for Westinghouse PWR's, XN-75-52 (Proprietary) September 1975.
- 3.7.3-4 Attachment to Letter NO-80-1759, Dated December 1, 1980, E. E. Utley (CP&L) to S. A. Varga (NRC), Request for Licensing Amendment - Spent Fuel Storage Expansion.
- 3.7.3-5 Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment, Revision 3. Prepared by SQUG and sent to the NRC by letter dated July 31, 1995.
- 3.7.3-6 Implementation Guidelines for Seismic Qualification of new and Replacement Equipment/Parts (NARE) Using Generic Implementation Procedure (GIP), Revision 4, dated July, 2000.



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10%G DESIGN SPECTRUM

FIGURE
3.7.2 - 1

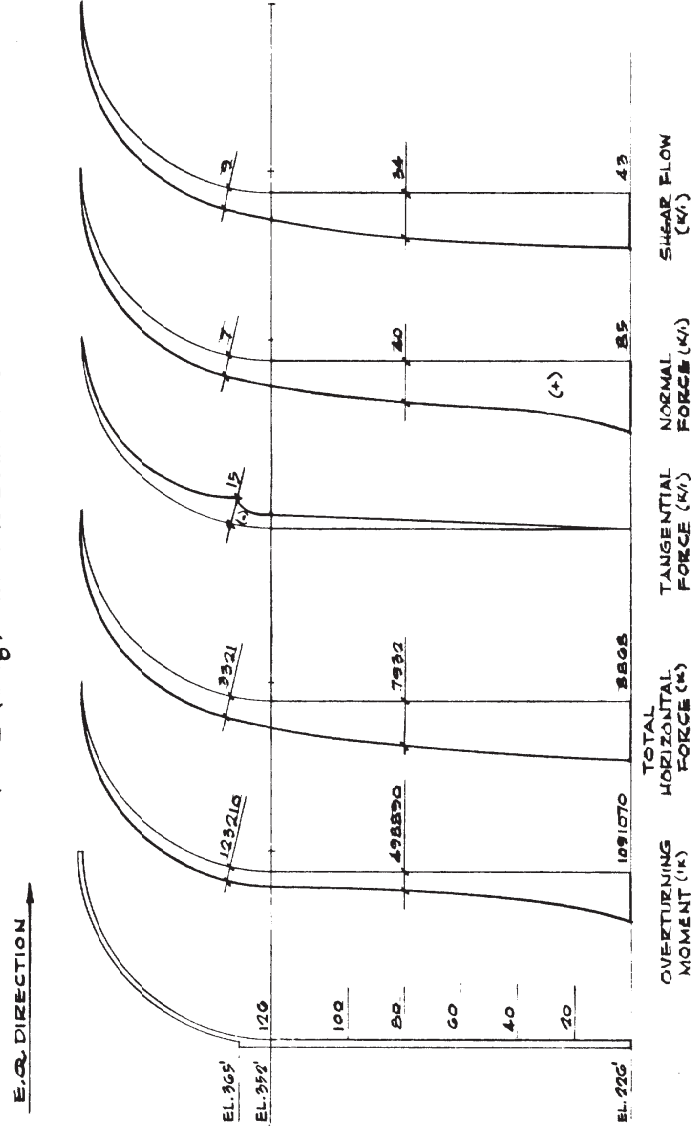


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20%G RESPONSE SPECTRUM

FIGURE
3.7.2 - 2

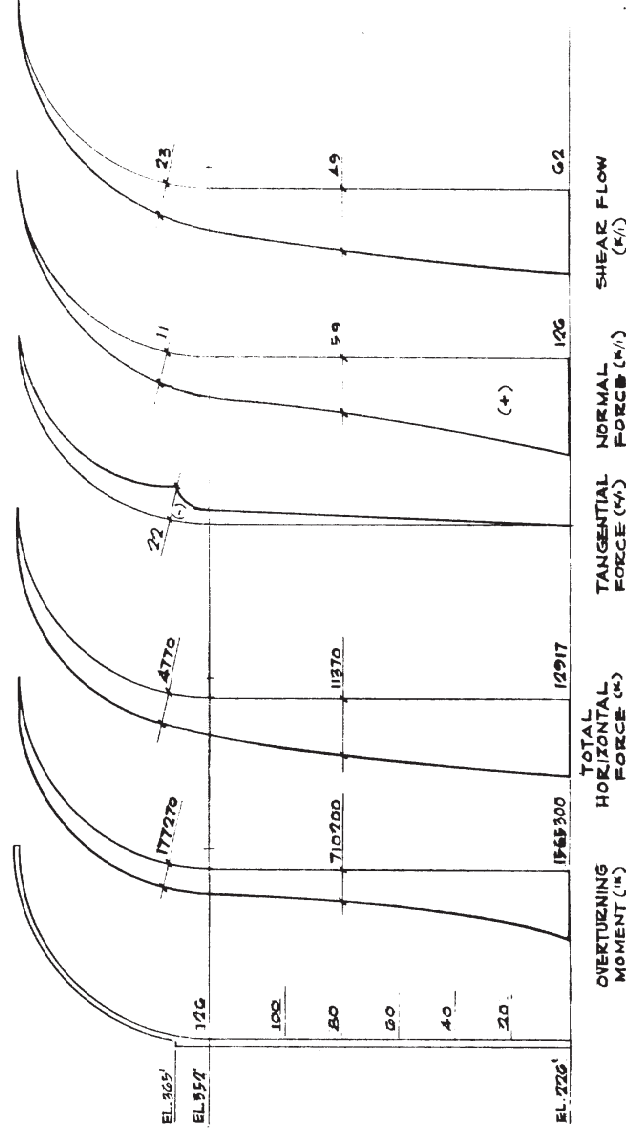
1.0 E (0.1g) WITH 2% DAMPING



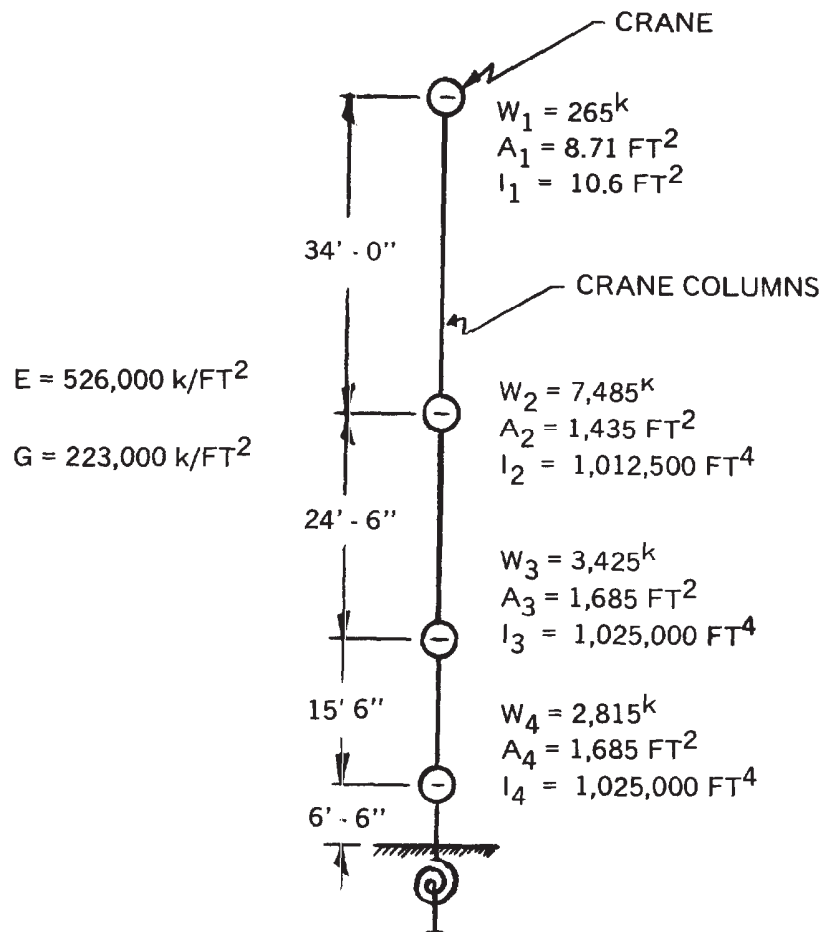
NOTE:
 (+) INDICATES TENSION
 (-) INDICATES COMPRESSION

1.0E'(0.2g) WITH 5% DAMPING

E-Q. DIRECTION →

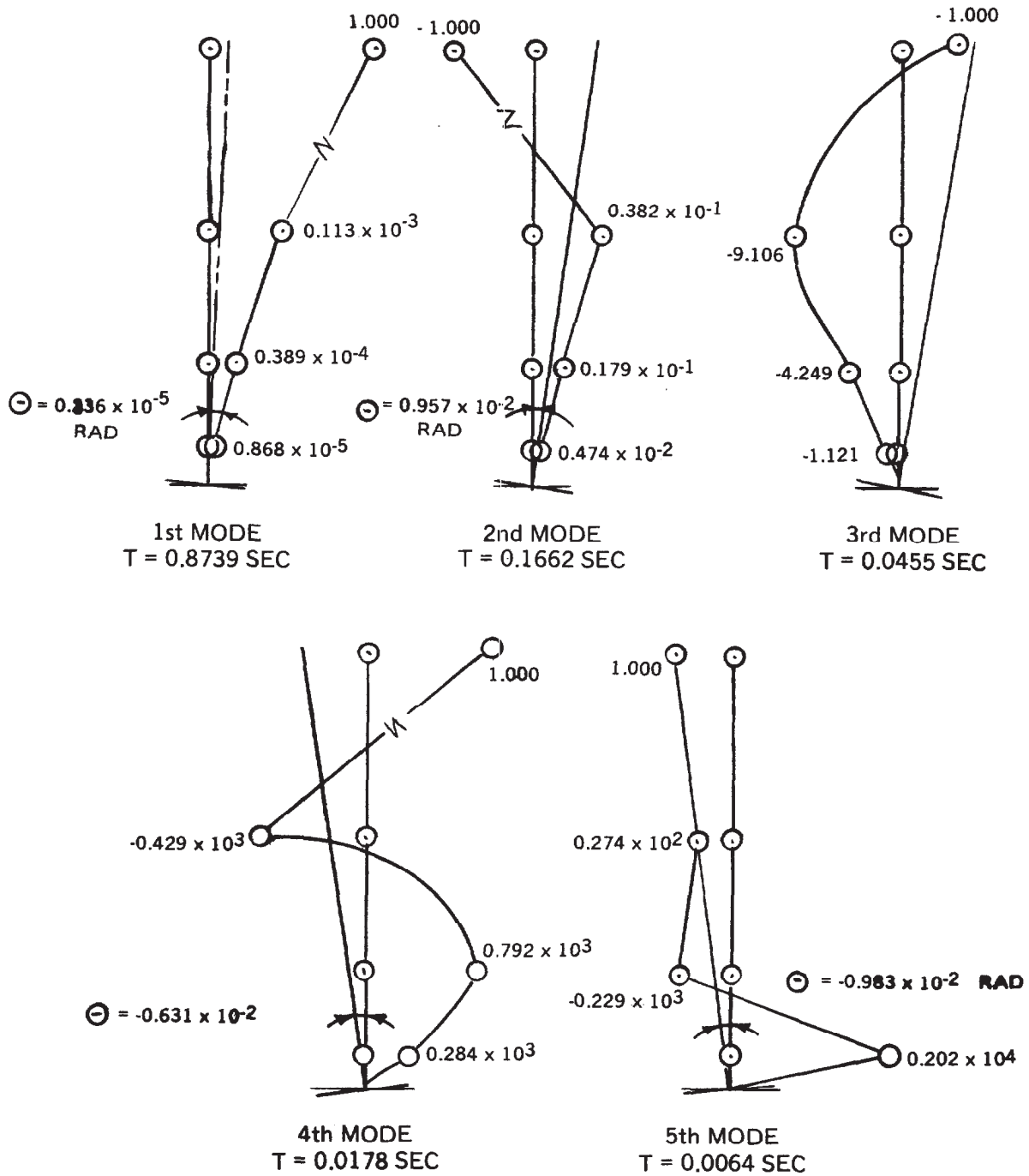


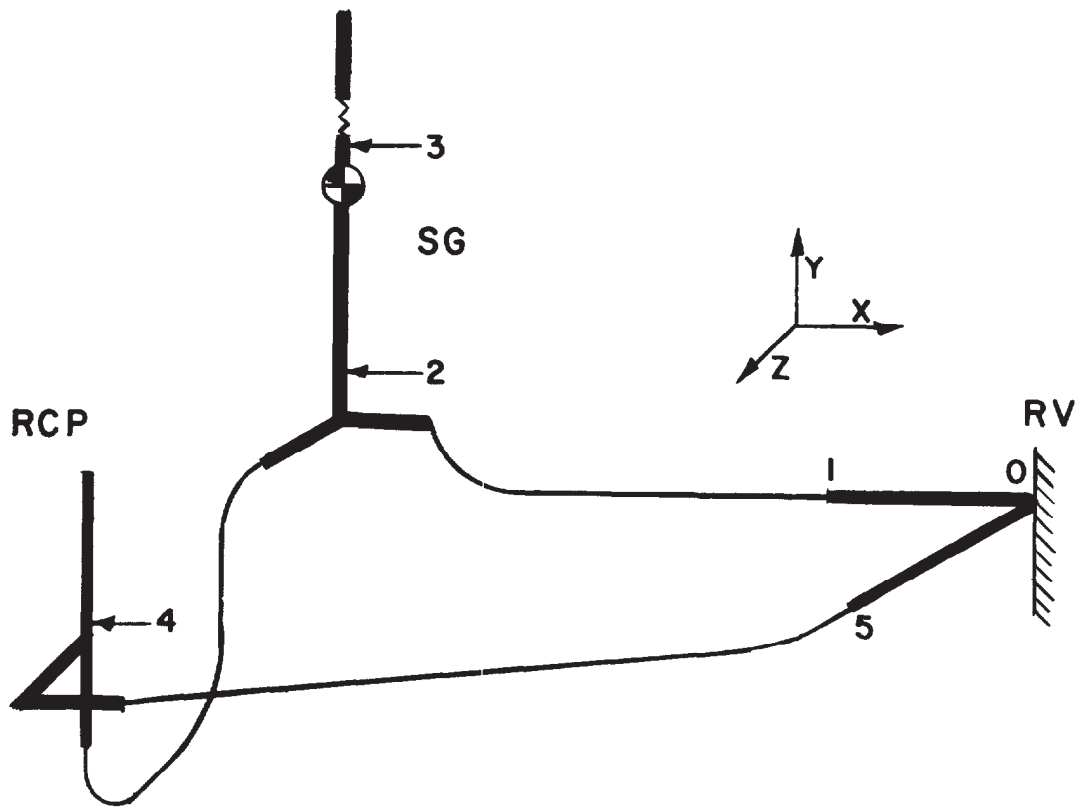
NOTE:
 (+) INDICATES TENSION
 (-) INDICATES COMPRESSION



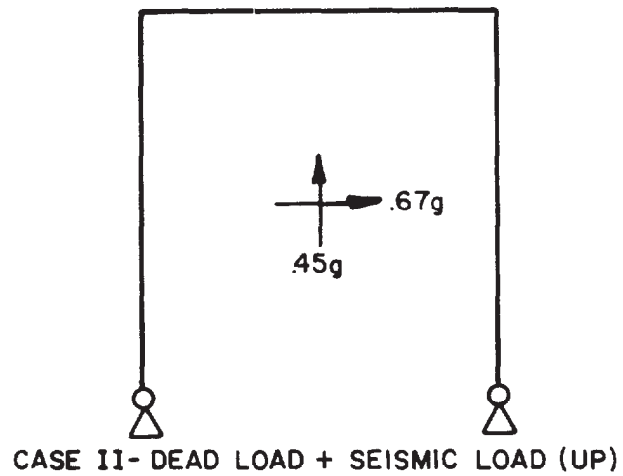
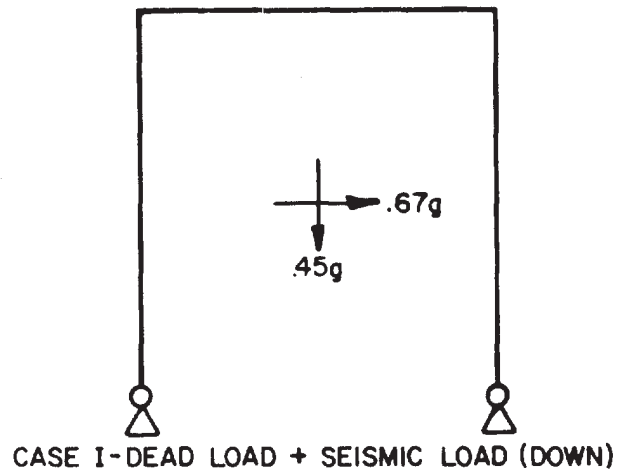
$$K_s = 187 \times 10^8 \text{ FT-k/RAD}$$

$$I_0 = 12.4 \times 10^6 \text{ k-SEC}^2 \cdot \text{FT}$$

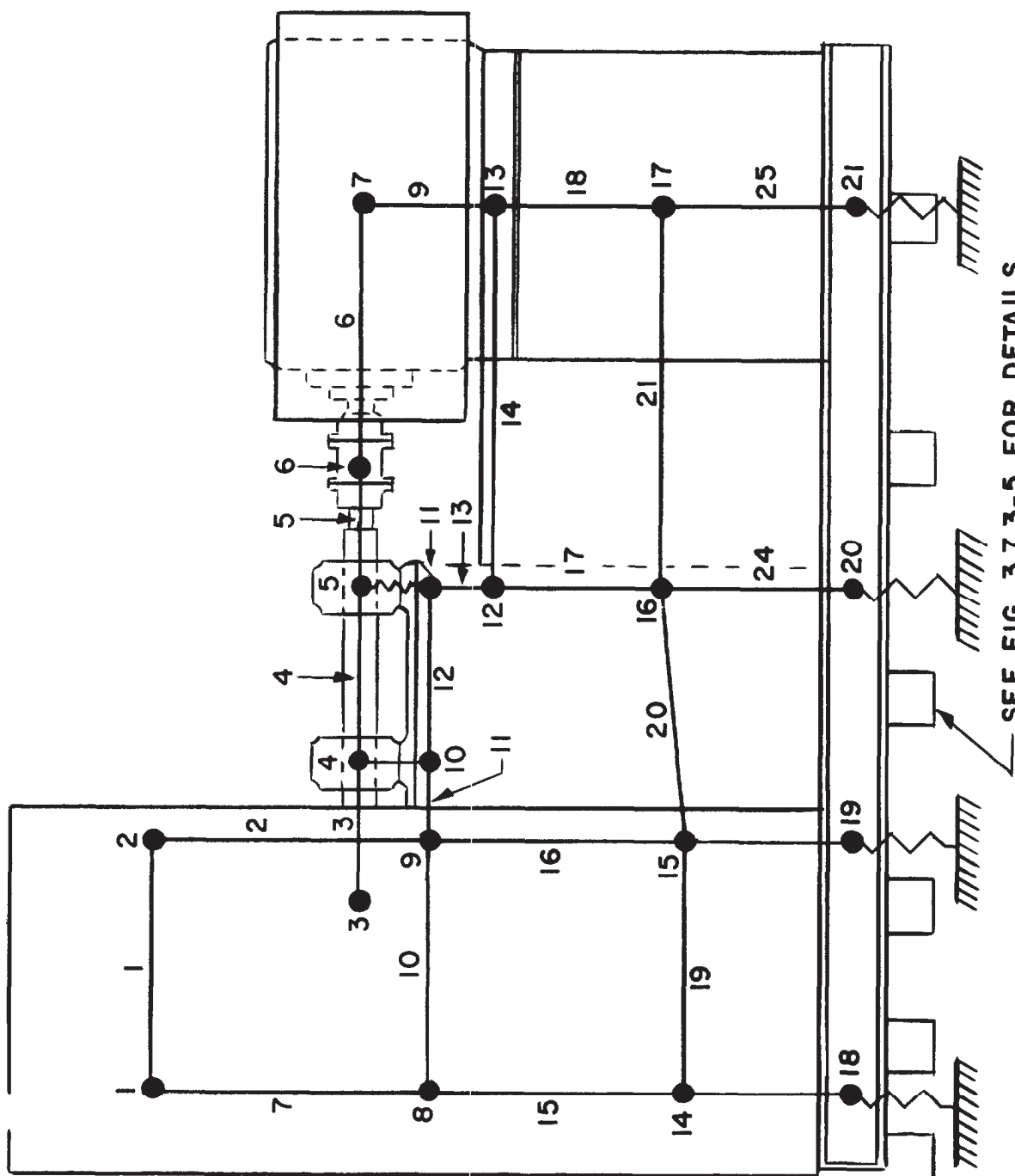




<p>H. B. ROBINSON UNIT 2</p> <p>Carolina Power & Light Company</p> <p>UPDATED FINAL SAFETY ANALYSIS REPORT</p>	<p>REACTOR COOLANT LOOP ORIENTATION FOR SEISMIC COMPUTER MODEL</p>	<p>FIGURE</p> <p>3.7.3 - 1</p>
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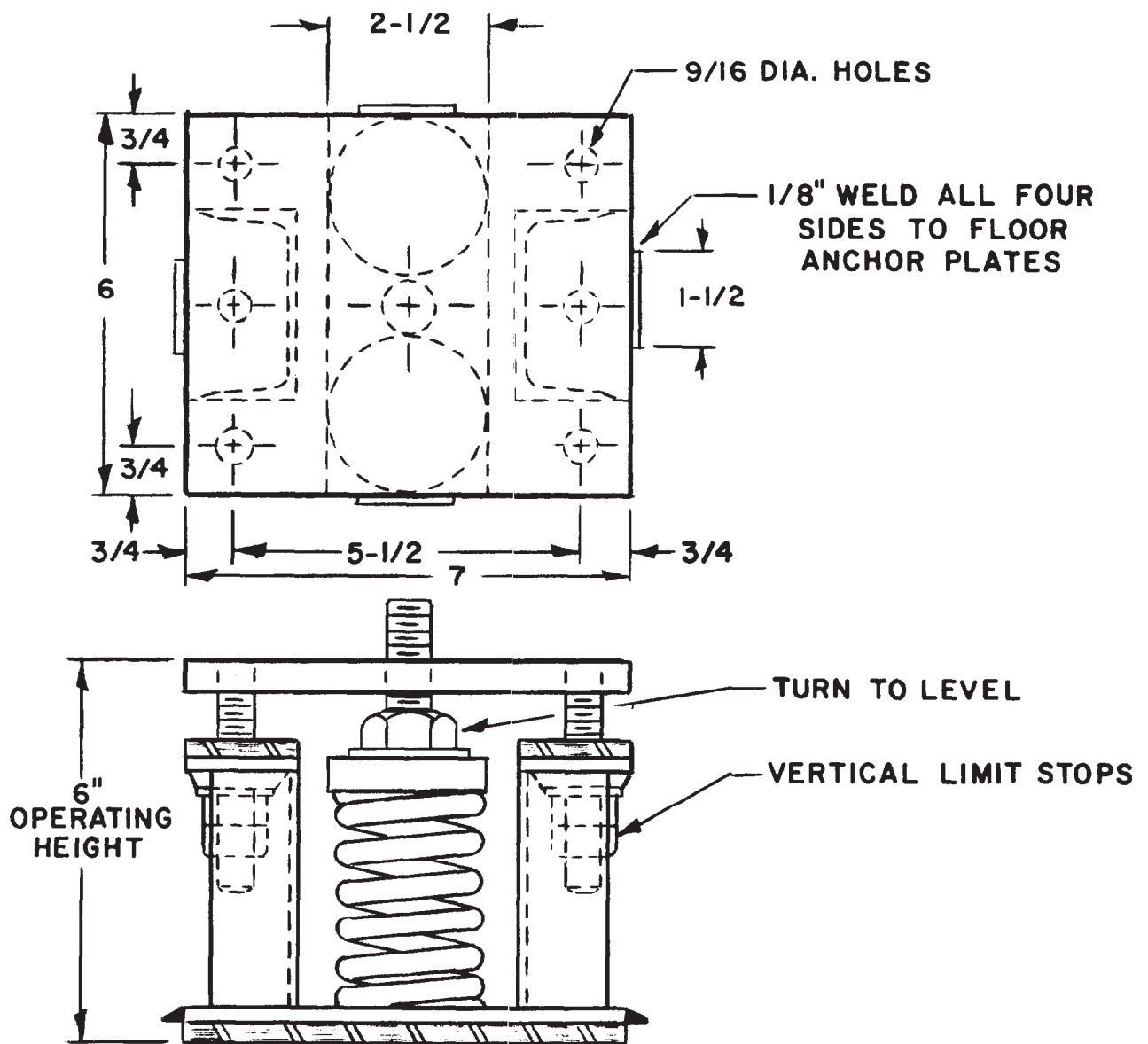
AMENDMENT 3



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LUMP PARAMETER MASS MODEL OF FAN COOLER SYSTEM

FIGURE
3.7.3 - 4



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FOUNDATION MOUNTING ATTACHMENT

FIGURE
3.7.3 - 5

3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 CONCRETE CONTAINMENT

3.8.1.1 Description of the Containment

3.8.1.1.1 General Description

The reactor containment structure is a steel lined concrete shell in the form of a vertical right cylinder with a hemispherical dome and a flat base supported by means of piles. The containment structure is designed for an accident pressure based upon the pressure transients as shown in Section 15.6. The containment structure is designed to contain radioactive material which might be released from the core following a loss-of-coolant accident as described in Section 6.2.1.

The structure consists of side walls measuring 126 ft from the liner on the base to the springline of the dome and an inside diameter of 130 ft. The containment free volume is 1,950,000 ft³. The side walls of the cylinder and dome are 3 ft 6 in. and 2 ft 6 in. thick, respectively. The inside radius of the dome is equal to the inside radius of the cylinder, i.e., the discontinuity at the springline due to the change in thickness is on the outer surface. The base consists of a 10 ft thick structural concrete slab. The base liner is installed on top of the structural slab and covered with two feet of concrete.

The basic structural elements considered in the design of the containment structure are the piles, base slab, side walls, and dome acting as essentially one structure under all loading conditions. The bottom plates of the liner are laid loose on the foundation slab and are anchored only at the hangways for the crane wall and primary shield. In the vertical walls and dome, the liner is anchored to the concrete shell by means of "KSM" shaped anchor studs fusion welded to the liner plate so that it forms an integral part of the entire composite structure under all loadings. The cylindrical portion of the liner is insulated. The dome of the containment is reinforced concrete. The cylinder walls are concrete-reinforced circumferentially and prestressed vertically. The base slab is reinforced concrete.

3.8.1.1.2 Base Slab

The base slab as shown on Figures 3.8.1-1 and 3.8.1-2 is 144 ft diameter circular reinforced concrete slab 10 ft in thickness. At the center it is penetrated by the reactor sump which extends 16 ft below the slab and is designed to hang from the slab. The base slab is designed to be supported by 923 steel pipe piles which supply restraint to it both vertically and horizontally and are anchored to it where required to provide restraint for uplift (See Figure 3.8.1-3). The base slab is reinforced with a radial, circumferential pattern on the top surface and a rectangular grid of reinforcing steel on the bottom which fits between the piles as shown on Figures 3.8.1-4 through 3.8.1-7.

Vertical stirrups are provided in areas of high shear. Design of the base slab has been in accordance with applicable portions of ACI 318-63 Part IV-B.

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3.8.1.1.3 Cylinder

The cylinder is 130 ft inside diameter, 126 ft from the top of the base slab to the springline of the dome and 3 ft 6 in. thick. Figures 3.8.1-8 and 3.8.1-9 show the masonry dimensions and outline. Figures 3.8.1-10 through 3.8.1-13 show the reinforcing steel at the base of the cylinder.

Both the cylinder and dome are designed to resist the membrane stresses imposed by the accident condition and local shears and moments resulting from secondary loads caused by discontinuities.

The cylinder is designed with hoop reinforcing steel of ASTM A408 Intermediate Grade and vertical high strength steel bars post tensioned to a value greater than the net dome uplift due to accident. Vertical reinforcing steel is provided to resist bending and thermal stresses. All vertical reinforcing steel is ASTM A408 Intermediate Grade.

Concrete in the cylinder is 4000 and 5000 psi compressive strength. The inside of the cylinder is insulated to limit the temperature rise on the liner due to an accident.

3.8.1.1.4 Reinforcing Steel

Reinforcing steel in the dome of the containment vessel is deformed billet steel bars conforming to ASTM Designation A432-65. This steel has a minimum yield strength of 60,000 psi, a minimum tensile strength of 90,000 psi, and a minimum elongation of 7 percent in an 8 in. specimen.

Reinforcing steel in the cylindrical portion of the containment vessel and in the base slab is deformed billet steel bars conforming to ASTM A15 or A408 - Intermediate Grade. This steel has a minimum yield strength of 40,000 psi, a minimum tensile strength of 70,000 psi, and a minimum elongation of 7 percent in an 8 in. specimen.

Splices in reinforcing bars have been butt connected by means of Cadweld splices designed to develop the minimum ultimate tensile strength of the bar.

Each heat of reinforcing steel has been tested to determine the physical properties and the chemical content by the mill. In addition, random sampling and testing of the physical properties have been done by the constructor.

The splices used to join reinforcing bars have been tested to assure their capability to develop the minimum ultimate strength of the reinforcing steel. The test program required visual inspection and cutting out, at random, completed splices and testing to determine breaking strength and point and type of break.

Due to the potential cracking of the concrete, it was assumed that the bond between the concrete and the reinforcing had not fully developed.

At the bottom of the containment wall, a bond length for the reinforcing of twice the required length was used.

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Where the dome reinforcing is anchored in the top of the wall an anchorage detail consisting of an anchor plate backed up by a cadweld sleeve is provided at the end of each bar. The plate and the sleeve are designed to develop the full ultimate strength of the bar. As for all other cadwelds in the structure, 2 percent of the production cadwelds of this anchorage detail were laboratory tested for strength.

3.8.1.1.5 Containment liner

The containment liner is designed to serve as a leakproof membrane and is not relied upon for the structural integrity of the containment except for resisting tangential shears in the dome. It is anchored to the concrete by means of "KSM" shaped steel studs. The liner is not anchored to the concrete base slab hence does not act compositely with it. It was laid loose on the base slab and the butt weld backing strips were set in grooves in the base slab. After welding, the distortions in the liner were considered too great and a neat cement grout was flowed beneath it to fill the voids. A bond breaker, form oil, was flowed first on the base slab to prevent the liner from acting compositely with the slab.

Stress conditions in the liner under all conditions of design have been analyzed to assure that the principle stresses do not exceed the yield or buckling stresses as provided in design stress criteria. Fatigue, accident, and operational loads are discussed in Section 3.8.1.4. Sample buckling calculations are shown indicating maximum loading conditions that prevail. A discussion on the liner anchors and transfer on their loads into the concrete has been submitted in the Containment Design Report (Reference 3.8.1-1).

The loading condition which produces maximum biaxial compression in the liner is that of winter operation combined with 1.0 times the hypothetical earthquake. Under this condition, the allowable buckling stress is not exceeded.

The steel liner and its welded seam joints are covered by carbon steel channels with pressurizing connections. These seam weld channels can be used to determine the leak-tightness of the liner seam welds.

3.8.1.1.6 Containment penetrations

Penetrations through the containment reinforced concrete pressure barrier for pipe, electrical conductors, ducts, and access hatches are typically of the double barrier type. In general, a penetration consists of a sleeve embedded in the concrete wall and welded to the containment liner. The weld to the liner is shrouded by a channel which was used to demonstrate the integrity of the penetration-to-liner weld joint. The pipe, electrical conductor cartridge, or duct passes through the embedded sleeve and the ends of the resulting annulus are closed off, either by welded end plates, bolted flanges, or a combination of these. Provisions are made for differential expansion and misalignment between pipe or cartridge, and sleeve. Pressurizing connections are provided to demonstrate the integrity of the penetration assemblies.

An exception to this are electrical penetrations C1, C2, C3, C5, C9, C10, D9, E1, E5, E10 and F1. These penetrations have double pressure barrier protection in their header plate and therefore an endplate is required at one end only. See Section 3.8.1.2 for codes and standards applicability.

3.8.1.1.6.1 Electrical penetrations

Cartridge type penetrations are used for all electrical conductors passing through the containment, with the exception of the penetration at sleeve numbers C9, C10, D9, E5 and E10 in the north cable vault, and sleeve numbers C1, C2, C3, C5, E1 and F1 in the south cable vault which are of the capsule type design. The penetration cartridge is a hollow cylinder closed on both ends, through which the conductors pass. This cartridge is provided with a pressure connection to allow continuous or intermittent pressurization of the penetration. Figure 3.8.1-14 shows a typical cartridge type electrical penetration. Figure 3.8.1-14a shows a typical capsule design electrical penetration. The method used to seal the joint between the cartridge end plate and the conductor depends upon the type of cable involved. In general, there are four types used:

1. Type 1 - High voltage power, 4160 volts
2. Type 2 - Power, control, and instrumentation, 600 volts and below
3. Type 3 - Thermocouple leads
4. Type 4 - Coaxial and triaxial cables

Type 1 penetrations are rubber insulated copper rods. These insulated rods will pass through a leak tight gland fitting threaded into each end plate of the cartridge. Either alumina insulating bushings or fused glass seals may be used to provide the double barrier.

Type 2 penetrations are single or multiconductor mineral insulated cable with a metallic sheath. This cable will pass through a leak tight gland threaded into each end of the penetration cartridge. The ends of the mineral insulated cable are potted with epoxy resin. Copper rod conductors with fused glass seals in the cartridge and plates is an alternate which may be used.

Type 3 penetrations are the same as Type 2 except the conductors will be thermocouple material. The sealing methods are the same as for the Type 2 penetrations.

Type 4 penetrations are used for coaxial and triaxial cables. In addition to the leak tight gland fittings in the cartridge end plate, a plug and receptacle connection provides a double barrier to leakage through the cable itself. An alternate method uses fused glass seals in the cartridge end plates and fused glass seals between the conductors of the coaxial or triaxial cable.

In the capsule penetration design, a single stainless plate is machined with the required quantity of feed-through ports which are interconnected by peripherally machined gun drills which creates a manifold system for pressure monitoring. Feed-throughs are assembled through the plate and sealed in place with a patented metal compression fitting assembly which creates seal zones at the front and backside of the plate, while allowing for a chamber to form between the seal zones to accommodate leakage monitoring.

The capsule series penetration is designed for a weldment interface to the containment nozzle. The weldment interface is by a transition ring,

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factory welded to the penetration header plate, and field welded to the containment nozzle.

The penetration sleeves to accommodate the electrical penetration assembly cartridges are 10 in., Schedule 80 carbon steel pipe, except where otherwise noted. For the electrical penetrations C1, C2, C3, C5, C9, C10, D9, E1, E5, E10, and F1, the header plate and conductors are pressurized. There are 51 electrical penetrations.

3.8.1.1.6.2 Piping penetrations

Double barrier piping penetrations are typically provided for piping passing through the containment. The pipe is centered in the embedded sleeve which is welded to the liner, except for small pipes where several pipes may pass through the same penetration sleeve.

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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 vapor barrier will not be breeched due to a hypothesized pipe rupture.

Typically, end plates are welded to the pipe at both ends of the sleeve. Several pipes may pass through the same embedded sleeve to minimize the number of penetrations required. In this case, each pipe is welded to both end plates. A connection to the penetration sleeve is provided to allow continuous or intermittent pressurization of the compartment formed between the piping and the embedded sleeve. In the case of piping carrying hot fluid, the pipe is insulated. The RHR supply pipe is insulated to keep the concrete surrounding the embedded sleeve below 200°F. Figure 3.8.1-15 shows typical hot and cold pipe penetrations. There are 46 containment penetrations sleeves for pipes. Pipes are anchored to the structural steel girders as close as possible to the inside of the wall or to the crane wall. Loads due to pipe ruptures within the containment or due to thermal stresses are not transferred to the liner.

An exception to this is the steam generator blowdown penetrations and two safety injection penetrations (RHR penetrations S-14 and S-15). The end plate is welded directly to the sleeve. The sleeve is welded to the liner reinforcement plate. Piping loads are transmitted to the concrete wall, except for torsion loads which are carried by the liner plate. However, the torsion loads are below the liner allowable stress.

Two piping penetrations are provided in the containment sump area.

3.8.1.1.6.3 Equipment and personnel access hatches

An equipment hatch is provided which is fabricated from welded steel and furnished with a double-gasketed flange and bolted dished door. Equipment up to a diameter of approximately 18 ft can be transferred into and out of containment via this hatch. The hatch barrel is embedded in the containment wall and welded to the liner and is a portion of the structural frame embedded in the wall. Provision is made to pressurize the space between the double gaskets of the door flanges and the weld seam channels at the liner joint, hatch flanges, and dished door. Pressure is relieved from the double gasket spaces prior to opening the door.

The personnel hatch is a double door, hydraulically-latched, welded steel assembly. It is attached to the structural frame embedded in the wall of which the frame barrel forms the central portion of the lock. A quick-acting type, equalizing valve connects the personnel hatch with the interior of the containment vessel for the purposes of equalizing pressure in the two systems when entering or leaving the containment.

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The personnel hatch doors are interlocked to prevent both being opened simultaneously and to ensure that one door is completely closed before the opposite door can be opened. Indicating lights and annunciators situated in the control room indicate the door operational status.

Provision is made to permit bypassing the door interlocking system to allow doors to be left open during plant cold shutdown. Each door lock hinge is designed to be capable of independent three-dimensional adjustment to assist proper seating. An Emergency Lighting and Communication System operating from an external emergency supply is provided in the lock interior. Emergency access to either the inner door, from the containment interior; or to the outer door, from outside, is possible by the use of special door unlatching tools.

3.8.1.1.6.4 Fuel Transfer Penetration

A fuel transfer penetration is provided for fuel movement between the refueling transfer canal in the reactor containment and the spent fuel pit. The penetration consists of a 20 in. stainless steel pipe installed inside a 24 in. pipe (See Figure 3.8.1-16). The inner pipe acts as the transfer tube and is fitted with a double-gasketed blind flange in the refueling canal and a standard gate valve in the spent fuel pit. This arrangement prevents leakage through the transfer tube in the event of an accident. The outer pipe is welded to the containment liner and provision is made by use of a special seal ring for testing all welds essential to the integrity of the penetration. Bellows expansion joints are provided on the pipes to compensate for any differential movement between the two pipes or other structures.

3.8.1.1.6.5 Containment Supply and Exhaust Purge Ducts

The ventilation system purge ducts are each equipped with two quick-acting tight-sealing butterfly valves for isolation purposes. The valves are manually opened for containment purging, but are automatically actuated to the closed position upon a safety injection signal or high containment radiation level signal.

3.8.1.1.7 Containment Dome

The dome is a hemispherical dome 130 ft inside diameter and 2 ft 6 in. thick reinforced concrete. The difference in cylinder and dome thickness is effected on the outside surface, the transition between thicknesses being accomplished 13 ft above the springline of the dome at the anchor surface of the cylinder prestressing steel tendons. The general masonry outline of the dome is shown on Figure 3.8.1-9. The inside of the dome is insulated from the springline to a point above the anchor surface of the cylinder prestressing steel tendons. The outer surface of the dome is covered with a membrane roof to provide weather protection.

The dome is reinforced with a radial - circumferential pattern of ASTM A432 (60,000 psi yield strength) reinforcing steel. At two lines where the stresses in the radial reinforcing steel is one-half of the theoretical reinforcing steel stress due to the convergence of the bars the radial reinforcing steel is anchored to structural steel assemblies which transfer the load to one-half the number of reinforcing bars anchored to the other side

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of the structural steel assembly. The radial reinforcing steel is replaced with a structural steel plate "skull-cap" at the zenith of the dome and the radial reinforcing steel is anchored to it by means of Cadweld splices. Figures 3.8.1-17 through 3.8.1-19 show the dome reinforcing layout and the structural transfer assemblies and "skull-cap".

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All radial shear in the dome near the springline is assumed to be resisted by reinforcing steel diagonal bars designed in accordance with ACI 318-63 Part IV-B. All tangential shear in the dome is assumed to be resisted by the steel liner plate.

3.8.1.1.8 Prestressing Steel Tendon Design

The prestressing system chosen for post-tensioning the Robinson containment structure in the vertical direction, consists of 1 3/8 in. diameter high strength steel bars supplied by Stressteel Corporation closely grouped into tendons consisting of six bars per tendon.

These tendons are placed within heavy wall 6 in. galvanized steel pipe sheaths.

Tendons are on the centerline of the wall and are spaced approximately every 3 ft around the periphery of the containment. This concept of grouping a number of high strength bars is not new to the industry, a seven bar group having been used in a dam in South America. It utilizes standard concepts and components proven adequate through 15 years of experience in the United States, and more experience using the similar Lee-McCall system in Europe.

The bottom anchorage is a steel plate with six threaded holes into which are screwed the steel bars. The thread used is a tapered, cut thread designed and proven to develop the minimum guaranteed ultimate tensile strength of the bar. Complete seating of the thread is necessary to develop the full tensile strength of the bar and this is easily accomplished and inspected since the thread is fully engaged when no threads are showing at the inner face of the plate.

Threaded holes in the bottom bearing plate have been sealed from the bottom by a steel plug with a binding compound between the plug and the bar. The entire bottom anchorage was designed and tested to show no permanent physical distortion at the minimum ultimate tensile strength of the tendon which is a 25 percent greater load than the maximum load to which it will be subjected in the life of the structure.

Couplings consist of internally threaded sleeves into which the high strength steel bars are securely screwed. The same screw thread details are used as described for the bottom anchorage. The void space between the bars within the coupler body is filled with the same binding compound used to bind the threads to eliminate any possibility of corrosion.

The tendon coupling consists of a set of six individual bar couplers staggered in elevation. Each tendon has two couplers in its length, one at the construction joint in the cylinder wall at Elevation 250 ft which is a field assembled coupling and a second half way up the wall between the construction joint and the top anchorage which is a shop assembled coupling. To assure the integrity of the coupler, the threads are coated with an epoxy compound which binds the coupler sleeve to the bar and prevents the possibility of unthreading due to vibration during shipping or erection. Once tensioned, the friction within the coupler threads eliminates any tendency for unthreading.

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The top anchorage consists of a steel plate bearing on the concrete with three of the Stressteel bars anchored to this plate by means of Howlett Grip Nuts. A second and smaller plate bears on the top of these grip nuts and the remaining three Stressteel bars are anchored to the top plate by means of Howlett Grip Nuts. The Howlett Grip Nut is a modified positive action wedge anchor which has the advantages of a wedge anchor and the positive adjustment capability of a threaded anchor. It does not require an exact predetermination of bar length along with all of the fine shimming required at the top anchorage with such a predetermination. This concept of stacking the top anchorage details allows a closer grouping of bars than is ordinarily required with a bar system.

The steel sheath surrounding each tendon is made of 6 in. Schedule 40 galvanized steel pipe with threaded and flanged connections. The sheath is connected to the bottom anchorage plate by means of a threaded coupling and provides protection of the tendon both during and after construction.

The use of a heavy steel sheath is not the ordinary post-tensioning practice and provides a positive barrier to the propagation of cracks during testing extending to the tendon. It precludes the possibility of any corrosive environment being formed near the tendon due to environment external to the sheath.

The tendons were stressed by means of a hydraulic jack which stresses one bar of the six-bar group at a time. The tensioning sequence required that three bars in each tendon be stressed initially using two jacks placed 180° apart and each working around the containment in a clockwise direction. When all tendons had three bars stressed, the remaining three bars in each tendon were stressed.

Tendon stress was measured by jacking force and was checked by total tendon elongation as measured from a nominal 1000 lb jack pull to maximum jacking pull. Bar stress was verified by repressuring the jack until lift-off of the top anchorage nut occurred and the jacking pull necessary for this lift-off was the verification of the bar stress.

The prestressing steel tendons interrupted by the equipment hatch and personnel lock were terminated at the penetration and anchored to the structural steel frame around the penetration.

The design of the prestressing steel system allowed for a 2 percent loss of tendons due to breaking as an ultra-conservative figure. The system was designed as an unbonded system.

The anchorages and tendons have been tested to verify that they are capable of withstanding a cyclic type of loading within the range expected during earthquake.

A history and experience in the use of high strength alloy steel bars is included in the Preliminary Safety Analysis Report for HBR 2 Amendment No. 5 - Page VIII C(2) (e)-1.

The corrosion protection scheme used during construction was a nitrogen atmosphere, a system widely used for corrosion protection (Section 3.8.1.6).

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As an additional check, six extra removable bars have been placed in six of the lower tendon assemblies. These bars have been removed to monitor the effectiveness of the nitrogen atmosphere corrosion protection. All tendons were inspected visually from the top before coupling or grouting to check that corrosion had not occurred.

A study was performed for the Robinson containment to determine the appropriate value to be used for relaxation of the Stressteel bars. This study was based upon relaxation tests made on bars both in the United States and in the United Kingdom. Results of these tests are shown in Figures 3.8.1-20 through 3.8.1-28.

The following losses due to relaxation of the steel were concluded from these tests:

<u>Initial Stress as % of Ultimate</u>	<u>Relaxation Loss % of Initial Stress</u>
60	5
70	6
75	7
80	8

Reference is made to the Preliminary Safety Analysis Report for HBR 2, Amendment No. 5, for data on the performance of high strength steel bars under repeated loads, ductility of Stressteel bars, bar toughness, and elongation characteristics.

3.8.1.2 Applicable Codes, Standards, and Specifications

Approved specifications and codes designated by the responsible design engineer have been used as the minimum criteria for design and construction. In many cases these standards have been exceeded. A partial listing of these codes or specifications follows:

ACI 318 - 63	"Building Code Requirements for Reinforced Concrete"
ACI 613	"Recommended Practice for Selecting Proportions for Concrete"
ACI 614	"Recommended Practice for Measuring Mixing & Placing Concrete"
ACI	Manual of Concrete Inspection
AISC	"Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (April 17, 1963)"
API 620	Welded Steel Tanks for Oil Storage

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ASME Boiler and Pressure Vessel Code, Section III - Nuclear Vessels, Section VIII - Unfired Pressure Vessels, Section IX - Welding Qualifications - (Applicable Portions)

Containment design gives consideration to leakage testability, including necessary provisions to enable tests to comply with:

- a) ANS 7.62 Leakage Testing for Containment Structures for Nuclear Reactors (July 14, 1967)
- b) AEC Technical Safety Guide 7.5.1, "Containment Leakage Testing and Surveillance Requirements", (December 15, 1966).

All components, systems, and structures classified as Class I are designed in accordance with the following criteria:

- a) For the design earthquake ground acceleration of 0.1g horizontally coincident with a vertical acceleration of 0.067g, allowable stress limits were taken at 1.33 times allowable code stresses. Stress limitations for the containment structure are given in Section 2.2 of the Containment Design Report (Reference 3.8.1-1). Primary steady state stresses are maintained within the allowable 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, 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 0.133g acting in the vertical and 0.2g acting in the 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.

The liner has been constructed in accordance with API Code 620 "Welded Steel Tanks for Oil Storage" except for dimensional tolerances which are more stringent than both the API Code and the ASME Boiler and Pressure Vessel Code. Welding procedures and welders were qualified in conformance with Section IX of the ASME Boiler and Pressure Vessel Code. Post weld heat treatment, where required has been in accordance with the ASME Boiler and Pressure Vessel Code, Section III. Spot radiography was performed with the acceptance standards and procedures which complied with Section VIII of the ASME Boiler and Pressure Vessel Code. Testing, as construction proceeded, complied with American Standards Association ASA N6-2, "Safety Standards for Design Fabrication and Maintenance of Steel Containment Structures for Nuclear Power Reactors."

The liner is reinforced about all openings in accordance with the philosophy of ASME Section VIII Pressure Vessels Code (i.e., by replacing cutout area of liner plate).

Penetrations conform to the applicable sections of USAS N6.2-1965, "Safety Standard for the Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors." The personnel lock and the equipment access hatch conform to the requirements of ASME Section III Nuclear Vessels Code.

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The materials for penetrations, including the personnel and equipment access hatches together with the mechanical and electrical penetrations, are carbon steel, conforming with the requirements of the ASME Nuclear Vessels Code, and exhibit ductility and welding characteristics compatible with the main liner material. As required by the Nuclear Vessels Code, the penetration materials shall meet the necessary Charpy V-notch impact values obtained in accordance with ASTM Pressure Vessel Code Section III.

Electrical penetrations are designed and demonstrated by test to withstand, without loss of leaktightness, the containment post-accident environment and to meet the National Electric Code, IEEE - Proposed Guide for Electrical Penetration Assemblies in Containment Structures for Stationary Nuclear Power Reactors or subsequent issues of this standard, IEEE Electric Penetration Assemblies in Containment Structure for Nuclear Power Generating Stations.

3.8.1.3 Loads and Loading Combinations

The pressure retaining components of the containment structure are designed for the maximum probable earthquake ground motion of the site combined with the simultaneous loads of the design basis accident as follows:

- a) The liner is designed to ensure that no strains greater than the strain at the guaranteed minimum yield point occur at the factored loads
- b) The prestressed concrete is designed on the basis of a resultant concrete compression or zero tension due to primary membrane forces resulting from the factored loads.

The following loadings were considered in the design of the containment in addition to the pressure and temperature conditions described above:

- a) Structure dead load
- b) Live loads
- c) Internal test pressure
- d) Earthquake
- e) Wind
- f) Uplift due to buoyant forces
- g) Internal negative pressure, and
- h) Tornado.

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3.8.1.3.1 Loads

The following loads can act upon the containment structure creating stresses within the component parts:

- a) Dead load consists of the weight of the concrete wall, dome, liner insulation, base slab, and all internal concrete. Weights used for dead load calculations were as follows:
 - 1) Concrete 143 pcf as determined from laboratory tests
 - 2) Reinforcing Steel and Prestressing Steel 489 pcf using nominal cross-sectional areas of reinforcing as defined in ASTM for bar size and nominal cross-sectional areas of prestressing tendons
 - 3) Steel Liner 489 pcf using nominal cross-sectional area of liner
 - 4) Insulation 4 pcf PVC and/or 2 pcf Polyimide foam
- b) Live load consists of snow and construction loads on the dome and major components of equipment which are supported on the containment base slab. Snow and ice loads were applied uniformly to the top surface of the dome at an assumed value of 10 pounds per horizontal square foot (psf) which is equivalent to about 12 1/2 in. of snow. This is considered to be conservative since the slope of the dome tends to cause much of any snow which falls on it to slide off. Construction live load for the dome was assumed at 12 psf as specified by the Uniform Building Code.

Equipment loads were those specified by the manufacturers of the various pieces of equipment. Table 3.8.1-1 lists the weights of the major pieces of equipment.

- c) Internal pressure due to a loss-of-coolant accident cause containment stresses. This is a time dependent variable as discussed in Section 6.2.1.
- d) Thermal expansion stresses due to an internal temperature increase caused by a loss-of-coolant accident cause thermal loads. Thermal loads on the containment liner can be broken into two areas:
 - 1) Insulated cylinder
 - 2) Uninsulated dome

The cylinder liner does not begin to feel the thermal effects of the accident until after the containment pressure is brought well below the maximum accident value, since it is insulated, and this rise in temperature is considered in the design.

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The dome liner, being uninsulated, feels the thermal effects of the accident immediately and its temperature follows but lags the containment steam temperature.

The temperature of the liner is a time dependent variable for both portions.

The loads resulting from the thermal expansion of the liner after an accident are shown on Figure 3.8.1-29.

The accident condition will create only a very slight increase in the concrete temperature on the insulated portion of the wall, as illustrated by Figure 3.8.1-30.

The accident condition creates a high skin temperature on the concrete behind the uninsulated liner; however, after 10,000 seconds, when the containment temperature and pressure are rapidly decreasing, the total depth of the wall which has felt any increase in temperature is about 9 in. in the 2 ft 6 in. wall thickness. This is illustrated by Figure 3.8.1-31.

Because accident temperature rise in the concrete is minimal, increases in temperature of the concrete behind both the insulated and uninsulated liner were not considered.

- e) Operating thermal expansion stresses caused by normal thermal gradients across the containment wall and dome were analyzed. Three extreme conditions were considered.

For all three an "as constructed" temperature for the liner and concrete was assumed at 60°F. The three operating conditions analyzed were:

- 1) Summer operation:
 - (a) Operating temperature inside containment: 120°F
 - (b) Exterior sustained concrete temperature: 90°F
- 2) Winter operation (hot):
 - (a) Operating temperature inside containment: 120°F
 - (b) Exterior sustained concrete temperature: 20°F
- 3) Winter operation (cold):
 - (a) Operating temperature inside containment: 50°F
 - (b) Exterior sustained concrete temperature: 20°F

In all cases a straight line temperature gradient was used through the wall of the containment.

The loads resulting from the thermal operating conditions are shown on Figures 3.8.1-32 and 3.8.1-33.

- f) Uplift due to buoyant forces will be created by the displacement of ground water by the structure. The ground water elevation was assumed to be at grade level for conservatism. The buoyant force was computed as the weight of the displaced water assumed at a specific weight of 62.4 pcf.

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- g) Earthquake loading was predicated upon a design earthquake and a larger assumed hypothetical earthquake. These earthquakes have a horizontal ground acceleration of 0.10g and 0.20g, respectively. A vertical component two-thirds of the magnitude of the horizontal component was applied simultaneously (see Section 3.7).
- h) Wind loading was based upon the standard wind loading criteria as specified by the ASA in "American Standard Code Requirements for Minimum Design Loads in Buildings and Other Structures" (A58.1-1955). This code designates the site as being in a 25 psf zone; however, a 30 psf basic wind loading was used for conservatism. This is equivalent to a gusted wind velocity of 108 mph (see Section 3.3).
- i) Internal test pressure applied to verify the structural integrity of the vessel at 115 percent of the design pressure. This applied only under controlled conditions. For this structure the test pressure is 48.3 psig.
- j) Loading from an internal negative pressure of 2 psig was considered. The inadvertent initiation of containment spray flow with refueling borated water at 45°F temperature would produce a negative pressure in the containment of 2.96 psi at 75 percent initial relative humidity. The maximum allowable negative pressure is 3 psig, based on 2 psig times 1.5 load factor.
- k) Tornado - evaluated using a wind velocity of 300 mph at one atmosphere which is equivalent to a velocity pressure of 230 psf. An internal positive pressure of 14.7 psig was assumed to act simultaneously (see Section 3.3).

3.8.1.3.2 Load Combinations

The design is based upon limiting load factors which are used as the ratio by which loads are multiplied for design purposes to assure that the load/deformation behavior of the structure is one of elastic, small strain behavior at the design load. The load factor approach was used in this design as a means of making a rational evaluation of the isolated factors which must be considered in assuring an adequate safety margin for the structure. This approach permits the designer to place the greatest conservatism on those loads most subject to variation and which most directly control the overall safety of the structure. In the case of the containment structure, therefore, this approach places minimum emphasis on the fixed gravity loads and maximum emphasis on accident and earthquake or wind loads. The loads hereafter referred to as factored loads, utilized to determine the required limiting capacity of any structural element of the containment structure, were computed as follows:

- a) $C = 1.0D \pm 0.05D + 1.5P + 1.0(T + TL) + 1.0B$
- b) $C = 1.0D \pm 0.05D + 1.25P + 1.0(T' + TL') + 1.25E + 1.0B$

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c) $C = 1.0D + 0.05D + 1.0P + 1.0(T'' + TL'') + 1.0E' + 1.0B$

d) $C = 1.0D + 0.05D + 1.0P_T + 1.0(T_0 + TL_0) + 1.25W_T + 1.0B$

e) $C = 1.0D + 0.05D + 1.15P_D$

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where:	C	=	Required load capacity of section
	D	=	Dead load of structure and equipment and live loads
	P	=	Accident pressure load as shown on pressure-temperature transient curves
	T	=	Load due to maximum temperature gradient through the concrete shell and mat based upon temperatures associated with 1.5 times accident pressure
	TL	=	Load exerted by the liner based upon temperatures associated with 1.5 times accident pressure (including operating temperature thermal gradient)
	T'	=	Load due to maximum temperature gradient through the concrete shell and mat based upon temperatures associated with 1.25 times accident pressure (including operating temperature thermal gradient)
	TL'	=	Load exerted by the liner based upon temperatures associated with 1.25 times accident pressure (including operating temperature thermal gradient)
	E	=	Load resulting from either design earthquake or wind, whichever is greater
	T''	=	Load due to maximum temperature gradient through the concrete shell and mat based upon temperatures associated with the accident pressure
	TL''	=	Load exerted by the liner based upon temperatures associated with the accident pressure
	E'	=	Load resulting from assumed hypothetical earthquake
	P _T	=	Load due to equivalent internal pressure due to tornado
	T ₀	=	Load due to temperature gradient through the concrete shell based upon operating temperature
	TL ₀	=	Load exerted by liner based upon operating temperature
	W _T	=	Load resulting from tornado wind
	P _D	=	Design pressure
	B	=	Load resulting from buoyancy

Equation a) indicates that the containment has the capacity to withstand loadings at least 50 percent greater than those calculated for the postulated loss-of-coolant accident alone. Loads and deflections resulting from the analysis using Equation a) are shown in Figure 3.8.1-34.

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Equation b) indicates that the containment has the capacity to withstand loadings at least 25 percent greater than those calculated for the postulated loss-of-coolant accident with a coincident design earthquake. Loads and deflections resulting from the analysis using Equation b) are shown in Figure 3.8.1-35.

Equation c) indicates that the containment has the capacity to withstand loadings at least as great as those calculated for the postulated loss-of-coolant accident with a coincident assumed hypothetical earthquake, with no loss of function. Loads and deflections resulting from the analysis using Equation c) are shown in Figure 3.8.1-36. This equation is included to assure no loss of function for the containment resulting from the assumed hypothetical earthquake.

Equation d) indicates that the containment has the capacity to withstand loadings at least 25 percent greater than those postulated as resulting from a tornado. Loads and deflections resulting from the analysis using Equation d) are shown in Figure 3.8.1-37.

Equation e) indicates that the containment has the capacity to withstand loading at least 15 percent greater than those calculated for the design pressure alone as a test condition. Loads and deflections resulting from the analysis using Equation e) are shown in Figure 3.8.1-38.

If the loads resulting from the ASA wind load on any portion of the structure exceed those resulting from the earthquake load, the wind load "W" was used in lieu of "E" in the appropriate equation. The loads due to the buoyant force (B) are only used where they add to the net load on a component.

All structural components have been designed to withstand the most severe loading combination for the component.

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 318-63. 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 used are based in part upon the load factors specified in Part IV-B of ACI 318-63. Modifications are necessary to the ACI 318-63 factors (which were established for application to many different types of structures) to make them applicable to a containment structure. ACI 318-63 is a code established for buildings in which, during the life of the structure, the interior arrangements may be dramatically altered and even the occupancy and use completely changed. It therefore requires large load factors on dead load and live load.

The dead load for the containment structure can be computed very closely before construction and since the structure will serve only one function and use during its life, there is no chance of a major change in the dead load without extensive structural investigations. The same is true of the live load which, for the purposes of accident analysis, is actually the dead weight of the equipment and piping within the containment. Since the loads can be closely

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computed, the philosophy of the load factor approach, which is to place the greatest conservative on those loads which are most subject to variation, requires only a nominal load factor.

The load factor of 1.5 for the accident pressure and indirectly for the associated temperature is consistent with previous practice on similar containment vessels. The design pressure is based upon the simultaneous failure of several safeguard systems and is already conservative. The 50 percent margin is an additional conservatism and is similar to the margin provided by the ASME code.

The load factor for earthquake and wind is consistent with ACI 318-63 for the design earthquake.

The decrease in value for the load factors with increase in number or severity of design loads is consistent with the philosophy of the load factor approach; namely, that the probability of experiencing coincidental maximum loads, and their values being greater than computed, is somewhat less than the probability of a single maximum load occurring, and being greater than its computed value.

The load factor for buoyancy is justified as the value for this load can be closely calculated with little chance for error.

The load factor for tornado wind is in accordance with ACI 318-63 for wind loads. The load factor for tornado pressure is justified since the basic load represents a full vacuum which cannot be exceeded.

The load factor for thermal gradient through the concrete shell and load exerted by liner due to operating temperature during the incidence of a tornado (Equation d) is reasonable since the values used are already conservative and since the load combination does not occur simultaneously with an accident.

The load factors for test pressure (Equation e) are justified since this pressure is closely controlled and will not be exceeded.

The load factors for the combination of accident and assumed hypothetical earthquake (Equation c) demonstrates the capability of the structure to withstand this simultaneous combination of loads with no loss of function.

3.8.1.4 Design and Analysis Procedures

3.8.1.4.1 Mathematical Models Utilized

Basically three separate structural components have been analyzed, each in equilibrium with loads applied to it and with constraints occurring at the juncture of the structures. The three structures are:

- a) The 130 ft ID hemispherical dome
- b) The 130 ft ID cylinder
- c) The base slab (including piles)

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Mathematically, the dome and cylinder have been treated as thin-walled shell structures which, for internal pressure and thermal loads and as cantilever beam for earthquake loads, result in analysis of them as membranes. Since the thickness of the dome and cylinder is small in comparison with the radius of curvature (cylinder 1/18.5, dome 1/25.5) and there are no discontinuities such as sharp bends in the meridional curves, the stresses due to pressure and wind or earthquake can be calculated by assuming that they are uniformly distributed across the thickness.

The base slab was treated as an annulus supported on an elastic foundation. The basic assumptions are made for this type of structure:

- a) The foundation is elastic, implying that the settlement at any point is proportional to the pressure at that point
- b) The foundation modulus in tension is equal to that in compression. The foundation modulus is defined as the pressure which is required to produce a unit settlement.

These assumptions are valid for the pile foundation described in this report, (Section 3.8.5) which is designed to transmit uplift to the piles.

3.8.1.4.2 Base Slab Analysis

The base slab has been analyzed as an annulus supported on an elastic foundation. The slab, a round disc resting on a pile foundation, will deflect depending upon the various loads imposed. The elastic foundation theory accounts for these deflections. The foundation modulus used in the design analysis has been determined by load tests performed in the field. Slight inaccuracies in the determination of the foundation modulus are reflected in the calculated values of moment, shear, and pile load as a fourth root of the initial inaccuracy and therefore do not greatly affect the calculated value.

The analysis of the annulus divides it into a number of radial wedges and analyzes each of these wedges as a beam on an elastic foundation. The solution, as outlined in "Beams on Elastic Foundation," by M. Hetenyi, was utilized to obtain the radial moments, shears and deflections, and circumferential moments.

To assure the correctness of the annulus assumption, the sump beneath the reactor vessel has been so designed and constructed that it hangs from the base slab and offers no bending resistance to the base slab nor any deflection resistance but only loads the slab with its own dead weight.

3.8.1.4.3 Cylinder Analysis

The analysis of the cylinder is the superposition of membrane forces resulting from gravity, pressure and thermal loads, over-turning due to earthquake or wind, and shears due to earthquake or wind. The concrete is reinforced circumferentially using reinforcing steel hoops and is post-tensioned vertically. The analysis, therefore, assumes that the concrete is vertically cracked under load.

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The basic equation for analysis is:

$$D \frac{d^4 w}{dy^4} + \frac{Eh}{r^2} w = P_z \quad (1)$$

where: $\frac{Eh}{r^2} =$ the equivalent to the foundation modulus for a beam on an elastic foundation; here taken as the modulus of the hoop reinforcing steel (Reference 3.8.1-2)

Earthquake shears are computed in the cylinder wall by the theory of shear flow.

3.8.1.4.3.1 Membrane Design

The hoop reinforcing steel is designed to resist the hoop load due to internal pressure and the internal bending moment due to thermal gradients. The prestressing steel tendons are designed to resist the membrane uplift load due to the dome pull under internal pressure.

The liner is not relied upon for structural integrity in the cylinder but has been analyzed to assure that all strains, as the liner acts in composite with the concrete shell, do not jeopardize the integrity of the liner (i.e., no strain greater than the strain at the guaranteed yield point will occur at the factored loads).

3.8.1.4.3.2 Tangential Shear Design

The membrane stress on any horizontal plane where the concrete is relied upon to resist shear is maintained at a value such that the provisions concerning shear of the ACI Code Part IV-B are met under the factored load conditions by providing sufficient post-tensioning force.

3.8.1.4.3.3 Longitudinal Shear Design

The design of the cylinder assumes that the concrete is vertically cracked under internal pressure load.

For the cylindrical portion of the vessel, resistance to the vertical shears resulting from the earthquake or wind loading is developed in the circumferential reinforcing by dowel action across the vertical cracks. The resulting principal stresses in the reinforcement does not exceed 0.95 x yield stress as provided in the design stress criteria.

Design of the structure provides for a minimum spacing of cracks in concrete tension members of twice the concrete cover dimension. To develop the vertical shear by dowel action the dowels must develop a bearing stress on the concrete no greater than the ultimate bearing strength of the concrete. The dowels (hoop reinforcing steel) are designed so as not to cause local crushing of the concrete at a crack. In addition, the liner aids in resisting the vertical shear across the cracks; however, this is not accounted for in determining the capacity of the cracked section to resist the vertical shear.

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For reinforced concrete in direct tension, Broms has determined that the minimum crack spacing in the concrete will be equal to twice the thickness of the concrete cover measured from the center of the reinforcing bar (Reference 3.8.1-3). Broms found by experimental means that the crack spacing did not decrease beyond a steel stress of 20,000 - 30,000 psi for a cover of 1.25 to 3.0 in. and about 50,000 psi for a cover of 6.0 in.

For purposes of computation a cover of 4.0 in. has been used for the hoop reinforcing steel in the containment vessel. In actuality this is the cover dimension from the edge of the bar and not the center and is therefore a conservative assumption resulting in a smaller value of crack spacing. The crack spacing was assumed as twice this dimension or 8 in. This is further conservative since Broms found that for a deeper cover the critical steel stress at which the minimum crack spacing was reached was higher and the average steel stress will be about 30,000 psi which is probably somewhat below the critical stress for the cover used.

All of these conservative assumptions result in a larger number of cracks than will probably occur. The crack width will be the total elongation of the hoop reinforcing steel divided by the number of cracks. The important consequence of the assumption is the increase in the magnitude of bearing stress on the concrete by the dowel. By decreasing the crack spacing this stress increases; therefore, the smallest crack spacing should be used for a conservative solution for this stress.

The total vertical shear is assumed to be transferred across the vertical cracks by the hoop reinforcing bars acting as dowels. The vertical shear is the same unit shear stress as exists due to shear flow on the horizontal plane. This stress is summed and divided equally into the number of horizontal hoop reinforcing bars and the individual bar shear stress computed. Computations indicate that dowel shear stresses of about 5000 psi exist for the maximum hypothetical earthquake. A principal stress analysis of the steel dowels indicates that the maximum shearing stress will be about 16,000 psi resulting in a margin against shear failure of about 1.25 or an actual ϕ factor of 0.80. This is more conservative than the limiting factors established in the Design Stress Criteria.

Computations indicate a maximum bearing stress on the concrete due to dowel action of about 6300 psi as computed by beam on elastic foundation theory. The Report of Sub-Committee III - ACI Committee 325 published in the July 1956 Journal of the American Concrete Institute indicates that concrete bearing strength for a dowel of about 2 in. diameter is 1.75 times the compressive strength of the concrete. This results in a bearing strength of 7000 psi for the concrete to be used ($f_c = 4000$ psi) and an actual ϕ value of 0.90 - equal to the ϕ value for flexure stipulated in the Design Stress Criteria.

The following discussion presents further justification for the acceptability of the 6300 psi stress level. As noted in Table 6 of the ACI Committee 325 paper, a ratio of ultimate bearing stress to f_c of 1.78 was achieved for a 2 in. diameter dowel. For a 4000 psi concrete then an ultimate bearing strength of 7100 psi can be expected for a 2 in. dowel. This is all that this paper was utilized for. The allowable capacity of dowels and allowable bearing stresses are intended for a joint in which there is a continual repetition of loading since it is intended for pavement joints and

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is, therefore, not applicable to the use intended in the containment structure. They are further intended for use with a normal working strength design and are not compatible with the factored load design of the Robinson containment. Utilizing a ϕ factor of 0.90 for bearing, then an allowable bearing stress of 6400 psi is obtained as a guide which compares favorably with the 6300 psi value presented.

It should be noted here that all of the vertical shear will be assumed as transmitted by the dowels. In actuality, some of the shear will be transmitted by the concrete crack interfaces since cracks of only 0.01 in. are expected and the cracks will be ragged. Therefore, the stresses computed are conservatively high.

The capacity of the dowels to resist and transfer shear across a vertical crack was analyzed utilizing the data presented in the Report by Sub-committee III, ACI Committee 325 entitled "Structural Design Considerations for Pavement Joints" as published in the Journal of the American Concrete Institute, July, 1956. The data presented in this report are obtained mostly from a report by Henri Marcus (Reference 3.8.1-4). Marcus' data are based upon a series of dowel tests which he performed.

Computations have been made to demonstrate the actual participation of the liner in resisting longitudinal shear. This is a computation of the composite action of the liner and cracked concrete wall.

The philosophy of this type of analysis is outlined in the Preliminary Facility Description and Safety Analysis Report for H.B. Robinson Unit 2 - Amendment 7, page VIII A 12 (a) 1-3.

The composite section analysis shows that for the Robinson containment the wall will carry two-thirds of the longitudinal shear and the liner one third where vertical cracks occur.

Principal stress analyses show that with this distribution of longitudinal shear the maximum shearing stresses and principal stresses are below design stress limits.

3.8.1.4.3.4 Radial Shear Design

As can be seen in the Containment Loading Diagrams the major radial shears occur at the spring line and at the base of the cylinder. The tensile vertical membrane stress in the cylinder at the spring line is such that the minimum membrane stress in the concrete is 0 psi under the governing factored load condition and is net compression under the design basis load. The cylinder at the base will always have a net compression on the horizontal plane. The combined radial and lateral shears are resisted by the concrete and, where required, supplementary reinforcing steel. Design for these shears is based upon ACI 318-63, Part IV-B, requirements.

ACI 318-63, Part IV-B is applicable to this structure since in essence it is a cantilever beam when a vertical section of wall is viewed. A substantial amount of reinforcement is provided in the vertical direction in the areas where radial shear is of a large magnitude. The provisions in ACI 318-63 require tensile reinforcing. They further use as a basis of development a cracked section analysis. The areas subjected to large radial shear will be slightly cracked due to bending.

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The basic analysis of the combination of loads at the base of the wall is a highly complex problem which does not readily lend itself to theoretical analysis. The shear provisions in ACI 318-63 are empirical and as stated in the report of ACI Committee 326 which established these provisions:

"Sufficient tests data are available to indicate that the proposals of this report for the design of beams without web reinforcement, are adequate at any section along the length of a member, regardless of concrete strength used, the strength of the reinforcement, the manner of loading and supporting the beam, or the cross-section shape of the member involved."

The formulas in ACI 318-63 are somewhat conservative as they define a lower limit rather than a mean shear resistance.

ACI Committee 326 report further states, in regard to the design concept of providing reinforcing steel for the shear more than that resisted by the concrete, that "neither laboratories tests nor field observation of performance in service have indicated a lack of safety resulting from current design methods such as those of ACI 318-56." The design methods of AC 318-56 are similar to those of ACI 318-63 with minor modifications made for ultimate strength design.

In designing for shear in the cylinder the combination of radial and lateral shears was used in determining the nominal shear stress on a horizontal section. When this combined shear was factored into the equation in ACI 318-63, Part IV-B, along with applicable hoop tension and vertical loads and bending moment and found to be more than the allowable ACI limits, then the two components of shear, namely the radial and lateral vectors, were reinforced in proportion to their total values for only the excess shears above allowed ACI limits. The reinforcing consists of stirrup bars for the radial shear and, for the lateral shears, the hoop reinforcing steel which in the areas of high radial and lateral shears is subjected to very low hoop stresses and is therefore available to act as stirrup shear reinforcing since at the base of the wall, membrane tension does not exist.

In addition, a principal stress analysis has been pursued in areas of high radial and lateral shears. The results of this analysis were used to verify the adequacy of the reinforcing layout as acknowledged by ACI Committee 326, a principal stress analysis on a reinforced concrete member subjected to shear and bending is a somewhat unsure approach as several of the stresses acting on the member cannot be truly defined in magnitude. It should be noted here that the shear provisions of ACI 318-63, Part IV-B, are based on principal stress analysis backed-up by empirical data.

Each penetration of the containment creates a discontinuity around which stresses must be carried. Where the penetrations are relatively small, the horizontal reinforcing steel has been shifted slightly up or down and the vertical prestressing tendons have been shifted slightly to each side of the penetration placed between tendons.

At penetrations too large to pass reinforcing steel around by slight shifting of the bars, structural steel frames have been provided. The reinforcing steel is attached to these frames by means of Cadweld splices, the splice sleeves being welded to the frames.

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Penetrations requiring these sleeves and steel frames are:

- | | | |
|----|---------------------------------------------|-----------------|
| a) | Fuel Transfer Tube | 24 in. dia. |
| b) | Containment Supply and Exhaust Ducts | 48 in. dia. |
| c) | Closely spaced group of piping penetrations | 24 ft x 10 ft |
| d) | Equipment Hatch | 18 ft dia. |
| e) | Personnel Lock | 9 ft dia. |
| f) | Main Steam Lines | 3 @ 42 in. dia. |
| g) | Feedwater Lines | 3 @ 28 in. dia. |

The equipment hatch and personnel lock are too large to pass the prestressing tendons around. Therefore the tendons are anchored to the frames.

The frames are designed to resist all of the loads applied to them in accordance with the same criteria as the reinforcing steel.

3.8.1.4.3.5 Prestressing Anchor Zone Design

The cylinder model in the area of the prestressing tendon anchors has vertical cracks. To transfer the uplift from the dome reinforcing steel to the prestressing steel tendons diagonal reinforcing steel has been utilized as shown in Figure 3.8.1-39.

To resist the radial component of the dome reinforcing steel in this area, reinforcing steel stirrups have been employed. These are designed to transmit all of the radial component.

3.8.1.4.4 Dome Analysis

The analysis of the hemispherical dome has been performed by the super-position of membrane forces resulting from gravity, accident pressure and accident thermal loads. In addition, earthquake and wind loading create both direct and shear stresses in the dome and the operating temperature of the liner creates tension and compression. All of the combined direct stresses are developed in the reinforcing steel encased in the concrete. The liner of the dome is used to resist the seismic shear load. The basic analysis is based upon the equations presented in "Thin Shell Concrete Structures" by D. P. Billington.

3.8.1.4.5 Liner Analysis

Under the design accident temperature conditions the cylinder liner tends to expand. In the vertical direction this thermal expansion is resisted by the concrete wall resulting in a compressive stress in the liner. This stress would be beyond the yield point of the liner material if the liner were not insulated to reduce the design temperature to which it would be exposed. Sufficient insulation has been provided to maintain the liner stresses within design limits. The liner in the dome does not need insulation as its thermal expansion is not resisted by concrete but by a web of reinforcing steel which

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is in itself elongating due to the internal pressure. This elongation due to pressure relieves the liner compressive stress to the point where it is within allowable limits. The stress conditions in the liner under all conditions of design have been analyzed to assure that the principle stresses do not exceed the yield stresses as provided in the design stress criteria.

The design of the stud connectors is such that the elastic stability of the liner plate is ensured, the forces acting on the liner are transferred directly to the concrete, and the liner is prevented from being pulled away from the concrete under conditions of internal negative pressure within the containment.

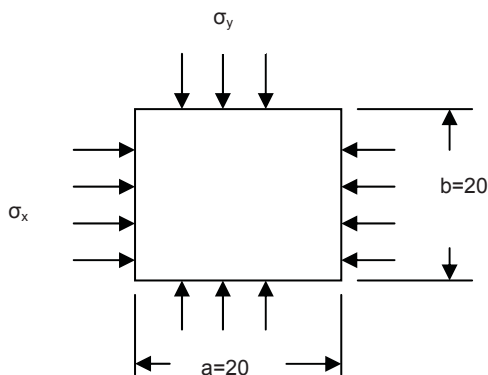
In arriving at the maximum spacing of the stud connectors to ensure elastic stability of the plate, the liner is considered to be a series of flat plates, simply supported at the edges, the size of the plate being the horizontal and vertical spacing of the studs. This is a conservative basis for design.

It is expected that the exact model for deformation of the liner is that of an axially loaded long cylinder with intermediate supports. The reasons for this are that under the operating conditions, the axial compressive strains are much greater than the circumferential compressive strains and because circumferential buckling is inhibited by the passive lateral support of the concrete wall. Therefore the deformed shape can be approximated by a vertical deformation wave which deforms away from the concrete between horizontal rows of anchors. The effect of hoop restraint is of some significance in resisting plate deformation away from the concrete; however, in the analysis, this beneficial effect was neglected.

While it is believed that this approach is reasonable and conservative, the size of the studs and their spacing will conform to an even more conservative approach which assumed simply supported straight plates.

A comparative analysis follows:

- a) Model A - The analysis of Model A is based on 20 in. square plates 3/8 in. thick, with biaxial uniform loads acting on all four edges (Reference 3.8.1-5).



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- 1) All edges simply supported

$$\sigma_{cr} = \frac{\pi^2 D}{a^2 h} = 9,450 \text{ psi}$$

For elastic stability to be assured,

$$\sigma_x + \sigma_y < 4 \sigma_{cr}, \text{ that is, } 37,800 \text{ psi}$$

The limiting case for biaxial compression as shown in answer to question VIII.B.(2) in Supplement 4 to the PSAR is $\sigma_x = -23,310$ psi and $\sigma_y = -9350$ psi. Then $23,310 + 9350 = 32,660$ psi which is $4 \sigma_{cr}$.

- 2) All edges are clamped

$$\sigma_{cr} = 5.30 \frac{\pi^2 D}{a^2 H} = 50,500 \text{ psi}$$

where: D = flexural rigidity

a = length of side = 20"

h = thickness of plate = 3/8"

- b) Model B - In Model B a fixed ended column, 20 in. long, 1 in. wide, and 3/8 in. thick is considered simulating the local deformation of an axial loaded cylinder, as discussed above.

$$I = .0044 \text{ in}^4$$

$$P_{cr} = \frac{4 EI \pi^2}{\text{liter}^2} = \frac{4 (30 \times 10^6) (.0044) (3.14)^2}{20^2} = 13,000$$

$$\sigma_{cr} = \frac{13,000}{\text{liter} \times .375} = 34,600 \text{ psi}$$

It is readily seen that Model A 1) is the most conservative, and this was used as the basis for design.

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The liner has been reinforced around penetrations using the ASME Pressure Vessel Code as a guide. This is a conservative approach since the liner contributes only a small amount to the gross rigidity of the cylinder and will strain in essentially the same fashion and to the same degree as the reinforcing steel in the wall. Since the penetrations have been designed for in the reinforcing by placing steel frames around them which will strain essentially the same as the unpenetrated wall the stresses in the liner will actually not be increased greatly at the penetrations.

3.8.1.4.6 Discontinuity Analysis

Discontinuity stresses occur at changes in section or direction of the containment shell.

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.

The juncture of the cylinder to the dome is a point of discontinuity since, under the internal pressure and temperature design conditions, the cylinder will tend to increase in diameter differently than the dome. To compute the unrestrained dimensional changes, the dome and cylinder are considered to be steel membranes equivalent in area to the reinforcing steel since the concrete is cracked vertically in the cylinder and in two directions in the dome, and only the reinforcing resists all hoop tension. The moments and shears are computed by equating the deformations of the cylinder and hemispherical dome at the point of juncture and solving for deflections, moments, and shears in the cylinder and dome.

In the solution of the continuity equation the cylinder is assumed to have no horizontal cracks since it is prestressed vertically. This is a conservative simplifying assumption since small horizontal cracks due to secondary discontinuity moments in the cylinder tend to relieve moment and shear. These discontinuity stresses are significant for a short distance from the point of discontinuity in the structure. Therefore, the stresses are considered local.

The juncture of the cylinder to the base slab is a point of discontinuity. In the analysis, the juncture is considered to be fixed to a mat of infinite rigidity. The cylinder at this point cannot expand or rotate under the internal pressure and temperature load conditions, hence, shear and moment is introduced into the cylinder walls. The analysis to obtain the moments and shears in the cylinder walls due to these loads at the joint is done in the same manner as for the juncture of the cylinder to dome.

At the large diameter penetration, such as the equipment hatch and personnel lock, structural steel frames have been embedded in the concrete. To these frames the horizontal and vertical reinforcing steel has been anchored and in the case of the equipment hatch and personnel lock the vertical prestressing tendons are also anchored to them. These frames are designed to resist all of the load applied to them from the members anchored to them in accordance with the same criteria as the reinforcing steel.

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The frames were analyzed as four-sided rigid frames with loads applied to all four sides. The effect of the equipment hatch and personnel lock on the containment cylinder wall was checked by a finite element analysis performed by the Franklin Institute. This analysis indicated that the structural frames around these two penetrations are slightly stiffer than the wall section but result in no stresses beyond the design stress limits.

Stiffness method based on flat plate finite element representation was developed for the subject analysis by Dr. Z. Zudans of the Franklin Institute Research Laboratory, (FIRL), Philadelphia, Pennsylvania. Individual elements consisted of several layers of orthotropic elastic material bounded by stiffened or nonstiffened edges. Mechanical surface loadings and arbitrary temperature distributions through the element thickness, as well as concentrated loadings, were included. Solution method was based on the usual assumptions of thin shell theory.

3.8.1.4.6.1 Introduction

The finite element method for finding stresses in plate structures was first treated by Hrennikoff (References 3.8.1-2 and 3.8.1-6). With the advent of computers this method was explored by Clough, Yettram and Husain, McCormick, Turner, Clough, Martin and Topp, Lunder, Zienkiewicz, Argyris, and others (References 3.8.1-7 through 3.8.1-15).

The method used for the H. B. Robinson Containment Vessel analysis was based on a four-cornered flat panel. Shell theory utilized in the analysis was the first order reference theory outlined in detail by Zudans (Reference 3.8.1-16). The structure was subdivided into a finite number of four-cornered elements. The stiffness of each of the elements was lumped at its four corners. Each of the corners of the finite element has five degrees of freedom: three displacements (axial, circumferential, radial) and two rotations (axial, circumferential). Equilibrium equations of the corners were written and the solution gave the displacements for each of the corners. These in turn are used to evaluate the stresses within the panel. In the following, a brief description of the analysis method, as well as the practical capabilities and the limitations of the resulting computer program is described.

3.8.1.4.6.2 Method of Analysis

Stiffness method of analysis was based on the solution of joint equilibrium equations written in terms of joint displacements. In order to generate the system equations, stiffness matrices of individual panels were required. These were computed by utilizing the Principle of Virtual Work (Reference 3.8.1-16).

(Note: symbols used in Equation (1) of Section 3.8.1.4.3 (Cylinder Analysis) are those of Reference 3.8.1-16, additional symbols are defined when they first appear in this description.)

Physical components of the joint displacements are assumed in the following form:

$$d = T(x^1, x^2) \underset{\sim}{a} \quad (2)$$

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where: $\begin{pmatrix} U \\ V \\ W \end{pmatrix}$ are the displacements in x^1 , x^2 and x^3 directions and θ_1, θ_2 are the rotations about x^1 and x^2 axes, respectively (Figure 5-62, Reference 3.8.1-16).

$\begin{pmatrix} \theta_1 \\ \theta_2 \end{pmatrix}$

T is a (5 x 20) matrix of polynomials in (x^1, x^2) describing the displacement form assumed in the panel, and a is a vector (20 x 1) of the constants determining the polynomial coefficients. The latter are found in terms of the panel corner displacements D by inverting a matrix consisting of four blocks of T evaluated at four panel corners. This results in the displacement vector expressed in terms of panel corner displacements.

$$\tilde{d} = T \tilde{t}^{-1} \tilde{D} \quad (3)$$

where \tilde{t}^{-1} is the above mentioned (20 x 20) inverse and D is the corner displacement vector (20 x 1).

The displacement vector, \tilde{D} , is further utilized to define the strains in the shell.

The joint force equation is:

$$\tilde{F} = K \tilde{D} + \tilde{P} + \tilde{H} \quad (4)$$

In Equation (4), F is (20 x 1) force vector corresponding to panel joint displacements, K is (20 x 20) panel stiffness matrix, P is (20 x 1) load vector, and H is (20 x 1) effective thermal load vector.

Presence of the stiffeners along the edges of the panel adds to each of the terms of \tilde{F}

Respective contributions are again evaluated by the use of the Principle of Virtual Work and are based on the shell stiffener theory developed in Reference 3.8.1-16.

Addition of \tilde{F} for each of the joints, as contributed by the panels attached to the joint, results in

the overall system of equilibrium equations for all joints of the system. This system of equations is then modified to account for the externally prescribed constraints on the displacements or on the forces (boundary conditions). Solution of the system is then performed by utilizing the tridiagonal character of this system. The tridiagonal character of the system is essential in order to achieve the required accuracy in the system solution.

Once the resulting displacement vector is found the stresses at the desired points are evaluated from the following equation:

$$\tilde{\sigma} = Z R T \tilde{t}^{-1} \tilde{D} \tilde{\sigma}^T \quad (5)$$

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In Equation (5), σ is (3 x 1) stress vector (σ_{11} , σ_{12} , σ_{22}), Z is (3 x 6) matrix relating strains to stresses, R is (6 x 5) matrix of differential operators relating strains to displacements and σ^T the stress modification vector (3 x 1) to account for the local temperature deviation from the linear through the wall thickness distribution.

3.8.1.4.6.3 Computer Program, Resulting Capabilities, Limitations

The computer program developed for the present analysis utilized greatly the FIRL three-dimensional structural analysis program. It consisted of four major links designed to perform the following operations:

Link 1 - Evaluates the stiffness matrices, load influence coefficients and other quantities required for the following links. It essentially operates for each of the individual panels as well as sets up the system topology required for proper bookkeeping when generating the system equilibrium equations.

Link 2 - Forms the system equations, modifies the same to suit the prescribed boundary conditions and records these on tape for auxiliary storage.

Link 3 - Breaks up the system in diagonal and tridiagonal blocks, inverts the modified diagonal blocks in a forward sweep, performs the backward sweep and generates the displacement solution vector. All matrix operations are performed in double precision and on (5 x 5) submatrices. Diagonal blocks A_{kk} are inverted by first performing the following operation

$$A_{kk} = S^T W S \quad (6)$$

where S is the upper triangular matrix and W a diagonal matrix. Inversion of the diagonal blocks is then performed by inverting the right-hand side of Equation (6). All operations in this part are kept in double precision in order to achieve the required accuracy. Last operation in this link consists in utilizing the solution and generating the joint equilibrium residuals. These residuals, compared to the joint load vector, were utilized to evaluate the achieved solution accuracy.

Link 4 - It is designed to evaluate the stresses (including the computation of principal stress) at specified points, print displacements and stresses in a convenient output format.

The computer program was first designed for an IBM 7094, 32K computer. The size limitations therefore were dictated by the size of this computer. After a sufficient number of test runs had been performed and the computer program had been fully debugged, it was recompiled on GE 635 with 120K core memory, increasing the critical dimensions to meet the requirements of a more sophisticated mathematical model (finer subdivision), without undue loss of the operational speed.

Capabilities of the program are as follows:

- a) Panels can consist of up to ten variable thickness layers each having its own orthotropic elastic properties
- b) Boundaries of each of the panels can be either stiffened or not stiffened

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- c) Alternative passage allows to prescribe panel in-plane and bending stiffnesses as input quantities.

This was incorporated to allow the possibility of evaluating the effects of concrete cracking and reinforcing steel configuration that would not fit the layer concept. There are five components of distributed panel surface loading that can be prescribed as well as an arbitrary temperature distribution through the wall thickness. It was anticipated that by proper specification of negative average panel temperatures the effects on concrete shrinkage and creep could be evaluated at any given time (but not as continuous functions of time), when the amount of the postulated shrinkage and creep was known. Similarly, prescription of positive tendon temperatures could allow for tendon relaxation. Concentrated loads could be specified at any of the joints in any of the five directions associated with the five degrees of freedom. The shape of the panel boundary between any two joints was defined by a corner point and one intermediate point coordinate. Total of 20 joints could be placed on line (floor in the sense of level above the base). This limitation hinged on the largest size diagonal block (in double precision) to be inverted in the core memory. Transition to a larger capacity computer allowed for an increase of at least by a factor of three (3) or more, (as dictated by the computational accuracy). Five simultaneous loading conditions could be handled.

Program debugging was based on comparing the results of this program with those of the known classical shell solutions as well as the comparison to some test data. There is a great number of shell problem solutions available at the FIRL and elsewhere in the literature.

3.8.1.4.7 Creep and Shrinkage in Post-Tensioned Reinforced Concrete Wall

The predicted loss of prestressing and tendon relaxation reflect the combined effect of creep and shrinkage in the post-tensioned, reinforced concrete containment wall. The results were:

a)	Initial Prestress	120 ksi (0.75 f_u)
b)	Final Prestress (Avg)	107 ksi (0.67 f_u)
c)	Tendon Relaxation Loss	7% of 120 ksi
d)	Modulus of Elasticity of Tendon	31 x 10 ⁶ psi

These values were obtained utilizing methods developed for analytical procedures proposed in References 3.8.1-17 through 3.8.1-25. For these calculations the thickness, height, and average temperature of the wall along the actual concrete mix as developed by laboratory trial mixes, the concrete strength and modulus of elasticity, age of concrete at time of prestressing, and amount of reinforcing steel were all considered.

The average creep and shrinkage strain within the wall thus obtained is 0.15 x 10⁻³ in./in. This results in a stress of 4350 psi (compression) in the vertical reinforcement, liner and large penetration frames. The latter are assumed to be strained as much as the wall's vertical steel.

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An error in the estimation in this figure results in a direct ratio change in the compression in the liner, vertical reinforcing and frames. Thusly, an error of 10 percent low for this value would result in an increase in these stresses of 435 psi (neglecting the interaction with relaxation of the prestressing tendons).

The following information was developed for the plant license renewal application and is included here to document the relative magnitudes of the changes in various factors affecting the prestressing loss, and remaining prestressing force levels during the initial license renewal period.

Description	Initial Value (April 1970) (lb/in ²)	Initial Value (May 1970) (lb/in ²)	Value after 60 Years (lb/in ²)
Prestress losses due to concrete shrinkage	N/A	N/A	6,000
Prestress losses due to concrete creep	N/A	N/A	9,471
Prestress losses due to tendon relaxation	N/A	N/A	10,200
Prestress losses due to elastic shortening	2104*	0	0
Total Tendon Prestress Loss	2104*	0	25,671
Available Tendon Prestress	120,000	120,000	94,329
Minimum Required Prestress**	91,726	91,726	91,726

*This loss due to tendon elastic shortening after initial tensioning in April 1970 but was regained in May 1970 when the tendons were re-tensioned.

**2% tendon breakage is conservatively factored into the Minimum Required Prestress value. The 2% tendon breakage from ACI is primarily associated with wire rope tendons; the RNP tendons are 1-3/8" diameter steel bars. Therefore, breakage would have been immediately identified during the tensioning process.

The containment exterior wall concrete was stressed to approximately 526 psi when the tendons were tensioned. This is a relatively low level for concrete with a 4000 psi design strength; the portion of the exterior wall at the tendon load block actually has a 5000 psi design strength. Because of these factors, the creep rate and total creep are estimated to be low.

The tendons were initially tensioned in late March/early April 1970, and then re-tensioned in May 1970. The last exterior wall concrete lifts were placed in December 1969 and the first lifts were placed in November 1968. Because of the delay in tendon tensioning from final concrete set to actually tensioning, the creep rate and total creep are estimated to be lower.

The tendons are considered to be post-tensioned because they were not loaded until after the concrete was placed. This allows a portion of the shrinkage to occur before tendon tensioning. The longer time between concrete placement and tendon tensioning, the more concrete hydration takes place. Therefore, the rate and total creep are reduced.

3.8.1.4.8 Performance Capability Margin

The containment structure is designed based upon 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 at design loads. 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.

The uncertainties of structural behavior have been accounted for in the selection of the methods of analyses and the results of the analyses can be shown to be conservative. Such uncertainties have not been accounted for by using margins on load assumptions. The methods of design and analysis for the Robinson containment structure have been chosen to be in accordance with this philosophy as follows:

- a) Dome: The dome was analyzed using thin shell theory which for this thickness of dome to diameter ratio yields an accurate representation of dome behavior. It is a method of analysis sufficiently simple in solution to require no simplifying assumptions, particularly in the critical design load conditions which create tensions in the membrane. The use of the dome liner only to resist shears due to wind or earthquake is a conservative assumption. With no internal pressure loading, the concrete of the dome will participate in resisting shear to a large extent. With the internal pressure loading and the concrete cracked, the concrete and reinforcing steel will still resist shear, but no credit is taken for this shear resistance.
- b) Cylinder: The basic cylinder analysis uses thin shell theory which for this diameter to thickness ratio yields an accurate representation of cylinder behavior.
- c) Base Slab: The use of beam on elastic foundation theory for solution of the base slab is an accurate representation of the actual slab. The slab is set on a series of piles (springs). The value of the modulus of subgrade reaction (spring constant) has been confirmed from test data obtained in the field.
- d) Junctions: Loads at the juncture of the dome to the cylinder and the cylinder to the base slab have been computed by solution of the continuity equation established for these junctures. In these solutions the assumption is made that the cylinder concrete is not cracked horizontally. This is a conservative simplifying assumption since small horizontal cracks due to the secondary discontinuity moments tend to relieve the moment and shear.

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- e) Penetrations: The stresses around the equipment and personnel hatches have been checked using a finite element approach resulting in a conservative design as a result of the selection of the initial loading and material properties.

In summary all elements of the containment have been analyzed using either a method of analysis which yields a true and accurate answer or using simplifying assumptions which result in a conservative answer for the particular stresses being computed.

- f) Proof Test: To provide further assurance for the adequacy of the structural design a proof test to 115 percent of the design pressure is made. This test compares the actual deformation of the structure with the predicted deformation based upon the design assumptions.

In considering the selection of the proof test pressure, the desired end results of the test must be considered. In the case of a containment structure constructed of concrete in which instrumentation will be provided to verify the reaction of the structure to the loads, it is necessary to meet three criteria at the proof pressure without causing any damage to the containment structure:

- 1) The liner should be stressed to levels in tension equal to, or greater than, the stress levels computed for the design basis accident condition.
- 2) The structure should be stressed to such a level that it reacts essentially the same as it would react with the design basis accident condition (neglecting earthquake forces). In the case of a concrete containment structure which depends upon reinforcing steel for its strength, it is necessary to have a test pressure sufficiently high that the concrete will crack, bringing into play the reinforcing steel. Considering concrete to have a tensile strength equal to 0.1 f', the Robinson containment concrete will crack at a pressure of about 34 psi. This is a pressure equal to 81 percent of the design pressure.
- 3) When instrumentation is to be used to measure deflections and deformations, the test pressure must be sufficiently high that the precision of the measuring devices is not a significant percentage of the magnitude of the measurement taken. With a test pressure equal to or slightly larger than the design pressure, a radial growth of about 1/2 in. in radius is expected in the cylinder. This can be accurately measured with precise theodolites using calibrated targets.

A proof test pressure which meets all three criteria is adequate for the purposes intended for the test. In the case of the Robinson concrete containment structure, a test pressure of 81 percent of design pressure would be the minimum which could be considered, since this is the lowest pressure at which the structural reaction of the containment will follow the design assumptions. At 115 percent of design pressure, all liner stresses are at, or above, the design basis accident stresses in tension. It would be correct and justifiable to decrease the test pressure to design pressure, since this would decrease liner stresses by a very small amount and the stresses would remain at, or above, the stress levels under design basis.

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3.8.1.5 Structural Acceptance Criteria

3.8.1.5.1 Design Stress Limits

Using the factored loads as defined in Section 3.8.1.3, the various components will have the required load capacity if the stresses in them do not exceed the yield strengths of the materials used.

To provide for the possibility that small, adverse variations in dimensions and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in a net under capacity of the component, the yield strengths of the individual materials were reduced by a reduction factor " ϕ ".

The design of the containment structure assumed a normal elastic distribution for stress across the cross section of the structural members rather than the stress block distribution assumed in ACI 318-63 Part IVB. As such, the factor ϕ was applied to f'_c at the extreme fiber in compression and all stresses reduced accordingly within the stress diagram.

The factors were established for the design on the basis of the function of the component and the effect on its net capacity of the variations enumerated in Section 3.8.1.3. For this design the following reduction factors " ϕ " were used:

a)	Structural components	
	Tension members	0.95
	Flexural members	0.90
	Shear and diagonal tension	0.85
	Bond	0.85
	Concrete Compression	0.90
b)	Liner	
	Tension	0.95
	Compression	0.90
	Shear	0.90
c)	Piles	0.75 for uplift, compression and shear of pile-soil structure

The use of the reduction factor " ϕ " is in accordance with the philosophy of ACI 318-63. The interpretation was altered to apply it to the yield strengths of the materials rather than the loads and component capacities as a design convenience yielding the same results. The reduction factors used were in accordance with ACI 318-63 with the exception of tension members of structural components and liner values. ACI 318-63 makes no provision for these values. The value for tension members of structural components was arrived at as slightly higher than flexural members since slight mislocations have little or no effect on direct tension stresses, whereas they have a greater effect on flexural stresses. The liner values were based upon the tension member of structural components for liner in tension, the flexural member for compression, and a slight increase in the value of shear and diagonal tension for shear since, for the liner, shear strength is not sensitive to the tensile strength of the concrete as in the structural components.

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3.8.1.5.2 Allowable Stresses for Large Steel Frames

The allowable stresses for the large steel frames are discussed in the Containment Design Report (Reference 3.8.1-1). The coefficients of creep and shrink are discussed in Section 3.8.1.4. Larger coefficients will result in larger strains and therefore increased stresses; however, the coefficients chosen are felt to be conservative. The age at time of prestressing will be considerably greater than assumed in the computation of this coefficient, and therefore the calculated creep coefficient is greater than that which will occur.

The grade of steel selected for the large frames was chosen on the basis of strain compatibility with the reinforcing steel under design loads, weldability, and quality control desired on the steel. The barrels of the equipment hatch frame and personnel lock frame are an extension of the liner and must therefore have the same nil ductility transition temperature (NDTT) requirements as the liner.

Both the equipment hatch and personnel lock frames were stress-relieved after completion of fabrication including welding of Cadweld sleeves.

Cadwelds were visually inspected and test splices made on a "sister splice" consisting of two sleeves welded back-to-back to a plate of steel identical to the flange steel, and these sister splices tested to failure.

3.8.1.5.3 Stresses in Prestressing Tendon Anchor Hardware and Concrete Under the Bearing Plates

The concrete under the bearing plates of the anchors was analyzed utilizing Guyon's "Prestressed Concrete" pp. 133 and 159 (Reference 3.8.1-26).

Analysis by means of a "SOLIDS" computer program indicated that, for the case of a cylindrical hole in the concrete, Guyon's results are more conservative than the "SOLIDS" results.

A typical stress calculation and reinforcing design for the concrete beneath a bearing plate is shown in Figures 3.8.1-40 and 3.8.1-41.

The stress condition in the prestressing anchor hardware is very complex. Therefore, a prototype of the anchor was tested to 110 percent of the minimum ultimate tensile strength of the tendon, and showed no yielding other than of minor magnitude.

3.8.1.5.4 Concrete Containment Design Considerations

The justification for the selected allowable maximum compression in concrete for the case of two or tri-dimensional stress distribution, when one or two of the three principal stresses are tensile stresses, is as follows.

The design model for the containment structure wall was a cylinder which was cracked vertically and therefore had no capability of resisting stress in the concrete in a hoop direction. The resulting vertical elements of concrete were analyzed as reinforced concrete beams using the uniaxial compression factor f'_c with normal design and analysis methods.

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In support of such a design model, the following is quoted from W. Rockenhauser's lecture for the Seminar on Prestressed Concrete Nuclear Reactor Structures (Reference 3.8.1-27).

"It is common design practice for ordinary biaxially stressed structures to use the full allowable concrete compressive stress . . . when the stress in a transverse direction is tension. Under such conditions concrete cracks will develop which will reduce the tensile stress fields in the concrete. The compressive stresses are carried by "compression columns" which cannot buckle because of the confinement provided by adjacent concrete. Lateral restraint may also be provided where bonded reinforcement is used."

"In triaxially stressed structures where tension occurs, a similar condition exists. Uniaxial tension with biaxial compression produces "compression planes" which are loaded biaxially in compression. Biaxial tension with uniaxial compression produces "compression columns" which sustain full uniaxial compressive loads providing confinement is provided".

The concept of compression columns does not completely apply here since the vertical beams do not have compression across their complete cross-section under all design conditions and hence do not require confinement as a column would.

As a by-product of the finite element analysis for the personnel lock frame a three-dimensional stress analysis was performed on the containment wall. The analysis accounted for the tensile strength of concrete by removing or altering the stiffness of the concrete when it was found to have a principle stress in excess of defined tensile strength $6\sqrt{f'_c}$. By such an analysis, the cracking of the concrete is simulated, and a final true stress pattern is developed which shows the principal stresses to be compression or very low tension.

3.8.1.6 Materials, Quality Assurance, and Special Construction Techniques

3.8.1.6.1 Materials

Basically eight materials have been used for construction of the containment structure for which materials specifications are discussed. These are:

- a) Concrete
- b) Reinforcing Steel
- c) Prestressing Steel System
- d) Plate Steel Penetration Frames
- e) Liner
- f) Equipment Hatch and Personnel Lock
- g) Insulation
- h) Pipe Pile

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3.8.1.6.1.1 Concrete

Concrete was specified to be of two classes:

- a) - 4000 psi 28 day cylinder compressive strength
- b) - 5000 psi 28 day cylinder compressive strength

Concrete mixes were designed by Froehling & Robertson, an independent testing laboratory, to meet a test mix strength 15 percent greater than the design requirements. All testing of concrete samples has been performed by Froehling and Robertson. Evaluation of compression tests has been in accordance with ACI 214-65.

All concrete was designed for a 4 in. slump and 3 percent entrained air.

The concrete is a dense, durable mixture of sound coarse aggregate, fine aggregate, cement, water and admixtures which are added to improve the quality and workability of the fluid concrete during placement and to retard the set of the concrete.

Aggregates conform to "Standard Specifications for Concrete Aggregate" ASTM Designation C33 (latest revision at the time of construction was used). Fine aggregate consists of sharp, hard, strong, and durable sand free from adherent coating, clay loam, alkali organic material, or other deleterious substances. Cement was Type II as specified in "Standard Specifications for Portland Cement" ASTM Designation C150 (latest revision at the time of construction was used).

Water was free from any injurious amounts of acid, alkali, salts, oil, sediment, or organic matter.

Admixtures used were in conformance with "Specification for Chemical Admixtures for Concrete" ASTM Designation C494 (latest revision at the time of construction was used). PDA Dual Purpose and PDA 25XL have been the admixtures used. These are lignosulfonates. Admixtures did not contain chlorides. The concrete has a minimum density after curing and drying of 143 psf.

Concrete samples were tested in accordance with the following ASTM Standards:

- a) ASTM C172 - Method of Sampling Fresh Concrete
- b) ASTM C31 - Method of Making and Curing Concrete Compression and Flexure Test Specimen in Field
- c) ASTM C39 - Method of Test for Compressive Strength of Moulded Concrete Cylinders

The number of test cylinders made under various conditions is shown in Table 3.8.1-2.

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The criteria for chemical analysis of the cement, water, and admixture used in the structure were as follows:

- a) Cement used in the general structures shall meet the requirements for Portland Cement in the following specification:

Portland Cement - - Type II
Federal Specification - SS-C-192g
ASTM Designation - C-150-58

1)	Chemical Composition	Percent
	Silicon Dioxide (SiO_2)	23.4
	Aluminum Oxide (Al_2O_3)	5.0
	Ferric Oxide (Fe_2O_3)	3.3
	Magnesium Oxide (MgO)	1.2
	Sulfur Trioxide (SO_3)	1.9 (0.27 Percent Na_2O)
	Loss on Ignition	0.7
	Insoluble Residue	0.32
	Tricalcium Silicate	44
	Tricalcium Aluminate	7.7
2)	Specific Surface: (Blaine)	3720 cm^2/g
3)	Soundness: Autoclave Expansion	0.02 Percent
4)	Time of Setting:	
	Initial Set	2 hr 50 min (Gillmore)
	Final Set	5 hr 40 min (Gillmore)
5)	Compressive Strength Tests:	
	3 days	2390 lb per sq in.
	7 days	3640 lb per sq in.
6)	Air Entrainment:	
	Percent by Volume - 9.4	

- b) Water used in the concrete shall be periodically analyzed by independent laboratories for the chloride content, with a specified limit of 100 ppm, in accordance with ASTM-D-512, and to certify that it is free from injurious amounts of oil, acid, alkali, organic matter, and other deleterious substances and considered to be chemically satisfactory.
- c) Admixture used in concrete for general structures shall be PDA25XL, PDA25DP, and Protex as manufactured by Protex Industries, Denver, Colorado. The first two are lignosulfonates and the latter is a resin solution - all three contain no chlorides.

3.8.1.6.1.2 Reinforcing Steel

Reinforcing steel in the dome of the containment vessel is deformed billet steel bars conforming to ASTM Designation A432-65. This steel has a minimum yield strength of 60,000 psi, a minimum tensile strength of 90,000 psi, and a minimum elongation of 7 percent in an 8 in. specimen.

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Reinforcing steel in the cylindrical portion of the containment vessel and in the base slab is deformed billet steel bars conforming to ASTM A15 or A408 - Intermediate Grade. This steel has a minimum yield strength of 40,000 psi, a minimum tensile strength of 70,000 psi, and a minimum elongation of 7 percent in an 8 in. specimen.

The base slab reinforcing originally planned to be A432 (60,000 psi yield) reinforcing was changed early in the design stages to A408 Intermediate Grade (40,000 psi yield). This was done to take some economic advantage of the requirements associated with the strain compatibility between the liner on the base slab and the bonded reinforcing in the slab.

The original liner design utilized a completely bonded liner on the slab, and the liner strength, rather than reinforcing strength, determined the amount of rebar. Therefore, there was no justification for the high strength steel rebar, and A408 was substituted. The liner on the base slab was changed to an unbonded design after the reinforcing was detailed. The only basic change involved in the lower grade rebar is a greater amount than would have been required with high strength bars.

All reinforcing steel and frames which form an extension of the reinforcing steel are encased completely within the highly alkaline environment of the concrete wall and dome and are, therefore, protected from corrosion.

3.8.1.6.1.3 Prestressing Steel System

The tendons are high strength steel bars 1 3/8 in. in diameter, fabricated from steel conforming to ASTM Specifications A322 and A29. During manufacture, each bar was cold stretched to a minimum of 87 1/2 percent of the guaranteed minimum ultimate strength: The following are the minimum physical properties:

- a) Breaking Strength - 238,000 lb/bar (160,000 psi)
- b) Yield Strength - @ 0.7 percent extension 208,000 lb/bar (140,000 psi)
- c) Approximate Modulus of Elasticity - 30,000,000 psi
- d) Elongation in 20 diameters after rupture - 4.0 percent
- e) Reduction in Area - 20 percent

Figure 3.8.1-42 is a typical load strain curve for a 1 3/8 in. diameter high strength steel bar from the lower tendon sections after cold working to 87-1/2 percent of the guaranteed minimum ultimate tensile strength. The fact that each bar must be stretched to cold work is in essence a positive proof test that each bar will sustain a maximum load in excess of that to which it will ever be subjected during the service life of the containment structure (about 80 percent of the guaranteed minimum ultimate strength of the bar).

There is no requirement on the prestressing steel tendons for NDTT. The change in stress in a prestressing tendon is of the order of 5 percent due to the 1.5p load condition. This increase in load occurs in the order of 10 seconds. The accident load cannot then be considered to be a shock load in nature; therefore, the NDTT is of little significance. Further, a structure

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such as the H. B. Robinson containment, which is composed of multi-redundant independent structural elements—in this case reinforcing steel and prestressing steel tendons—is not susceptible to "zipper action" type of failure whereby a single failure could propagate the NDTT, therefore, is of little interest for the structure with the exception of the liner which has NDTT requirements imposed upon it.

The steel pipe which encases the bars is ASTM Designation A53, 6 in. Schedule 40 galvanized, threaded and coupled pipe. This pipe has a nominal wall thickness of 0.280 in.

The following is a discussion of the tendon tests, material specifications, and corrosion protections for the Prestressing Steel System:

a) Tendon Tests

- 1) Bars - Each bar has been proof tested to 87-1/2 percent of its guaranteed minimum strength which is 9 percent greater than the maximum tensile stress to which it will be subjected at the time of tensioning. Anchors both top and bottom have had full scale prototype tests made on them to the minimum ultimate tensile strength of the tendon. These tests have been performed not to prove the individual components of hardware which are all well proven by experience to be adequate, but to prove and assure that the assemblage as designed and assembled does not have some unknown and unexpected weakness.
- 2) Coupling and Grip Nuts - One sample was taken from each 200 produced and the sample tested to the guaranteed minimum ultimate tensile strength of the bar.
- 3) Grip Nut Assemblies - As stated in Section 3.8.1, anchorage and tendons were tested to verify that they can withstand a cyclic type of loading expected during an earthquake. The results of this test conducted in November, 1966, are shown on Table 3.8.1-3 (Table B, Amendment No. 5 to the PSAR).

Another test was performed at Lehigh University on an assembly including a 1-3/8 in. Howlett grip nut on January 16, 1967. In this test the stress range was from 0.6 f's to 0.66 f's or a stress range of 10,000 psi. The test went 800,000 cycles with no indication of failure. The range was then increased to 0.6 f's to 0.7 f's or a stress range of 16,000 psi. This second test on the same assembly withstood another 400,000 cycles. The test report appears on Table 3.8.1-4. Subsequent to this test, several more fatigue tests have been made on grip nut assemblies. These test reports are shown on Tables 3.8.1-5 and 3.8.1-6.

During the testing of the lower anchor plate by Stressteel at their plant, one of the six bars, each anchored by a Howlett grip nut, was cycled 200 times between 70 and 75 percent of ultimate. No distress was observed.

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The stress range from operating conditions to accident and earthquake conditions is listed in the following table. These values are computed assuming a fully bonded tendon which, while not the tendon design, could occur and is a conservative assumption. Examining the previously mentioned tests, it is concluded that the Howlett grip nut is adequate under this range of stress change.

<u>Position On Wall</u>	<u>Load in Wall for 0.2g Earthquake</u>	<u>Approximate Change in Stress in Prestressing Tendon for 0.2g Earthquake</u>	<u>Approximate Change in Stress in Prestressing Tendon for 1.5P Accident Load</u>
Springline	± 18 k/lf	± 350 psi	+ 5000 psi
Base of Wall	± 127 k/lf	± 2500 psi	+ 5000 psi

- b) The following specifications and factory control requirements were imposed in the manufacture of the bar:
- 1) ASTM A322 Specification for Hot Rolled Alloy Steel Bars.
 - 2) ASTM A29 Specification for General Requirements for Hot Rolled and Cold Finished Carbon and Alloy Steel Bars.
 - 3) Four sets of stress-strain data were taken on each heat of steel after cold stretching.
 - 4) Each bar was certified to have been cold stretched to a minimum load of 208,000 lb which is 87-1/2 percent of the guaranteed minimum ultimate tensile strength of the bar.
 - 5) A full chemical analysis of each heat of steel used was made.
 - 6) All threads were cut on automatic thread cutting machinery and each thread inspected with Go and No Go gauges and the thread then coated with a protective coating of plastic if it was not to be coupled in the factory.
 - 7) In addition to the above tests made by the supplier, a sample of each heat of steel used was tested by an independent testing laboratory retained by the engineers to verify the physical strength properties of the steel.

c) Corrosion Protection

Ungalvanized prestressing steel tendons have an extensive history of successful application. The problem of corrosion was met and successfully solved by the use of Portland cement grout put in intimate contact with the steel. The alkalinity of the environment thus provided prevented corrosion of the steel tendon.

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Corrosion protection was afforded the coupling by a highly alkaline environment provided by the Portland Cement grout which completely surrounds each individual bar coupling. The space between the two bars in the middle of the coupling sleeve is filled with the same compound used to bind the threads to eliminate the air pocket and thereby the possibility of any corrosion at that point. A form of cathodic protection is provided by the galvanized steel pipe sheath which prevents any possibility of galvanic corrosion on the entire tendon.

Corrosion protection of the entire tendon during construction was accomplished by means of assembling all sections of the tendon at the factory (with the exception of the field-assembled coupling), placing the tendon within the steel sheath, capping the sheath at both ends, and introducing Vapor Phase Inhibitor (Type 250) crystals as manufactured by Shell Oil Company. The atmosphere within the steel sheath was then slightly pressurized and made inert with dry nitrogen until the tendon was grouted.

The threads exposed for some time before the coupling was made up at the construction joint were protected by dipping them into plastic which was peeled off when the coupling was made up. This protects the threads from corrosion and physical damage.

Cathodic protection for either reinforcing steel in the containment structure, or post stressing tendons, was not considered necessary at this plant. The basis for this is that bare steel embedded in properly compounded structural concrete (reinforcing steel) or sand cement group (post stressing tendons) is immune to the effects of galvanic corrosion under normal conditions because of the protective value of concrete on steel. Conditions at the construction site are considered to be ideal for this type of protection as sand, aggregate, and water available for concrete manufacture are such that a high grade of concrete or cement-sand grout is assured with quality control and inspection surveillance.

The construction for post tensioning steel tendons involving the enclosure of the grouted tendons in a sealed galvanized steel pipe ensured against corrosion to an even greater extent through the absence of oxygen, a degree of cathodic protection afforded by the galvanizing on the interior of the containment pipe, and shielding afforded by the containment pipe against entry of current from outside sources.

Concrete has been used successfully for many years as a protective covering for steel. The protection is afforded the steel by the uniform alkaline environment plus a polarization phenomenon which, with properly compounded concrete or grout, causes the steel to stifle either collection or discharge of direct current under galvanic corrosion conditions. The mechanism is discussed below.

The polarization effect on steel in concrete grouting has been discussed by E. H. Thalmann in his article on "Cathodic Protection in the Design of Power Plants," (Reference 3.8.1-28). In this paper it was pointed out that the long time effect of cathodic protection on steel in concrete was a potential of about -1.1 volts with respect to a Cu/CuSO_4 reference electrode. Since the protection potential of a zinc-steel cathodic protection system (such as that provided by galvanizing) is about -1.05 volts to Cu/CuSO_4 , (Reference 3.8.1-29), there should be a protective rather than a deleterious effect on the system.

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Assuming that the grouting is properly compounded and applied, there should be an initial polarization potential of between -1.1 and -1.05 volts, and no corrosion will be possible. As the grout continues to cure and its electrical resistivity increases, the current flow from the galvanizing will be diminished. This will result in some reduction of the polarization potential on the steel.

Under cathodic protection such as that afforded the tendons by the galvanized pipe sheath, the source of the polarization potential on the steel is the current flow from the galvanizing. Under such conditions, the potential of the steel may approach but will never exceed that of the zinc. This is because the polarization potential on the steel must be maintained by current flow from the zinc; should the current flow from the zinc be interrupted, the steel polarization would tend to drop. Because of this cause-and-result relationship, the back electromotive force on the steel can never reach the point where current discharge from the steel, with resulting corrosion, will occur.

The study disclosed further that if concrete contains impurities (particularly chlorides), the polarization effect was reduced greatly to the extent that corrosion of steel could proceed. These discoveries led to proper formulation of concrete for construction projects to ensure freedom from corrosion on reinforcing steel and when used for corrosion protection for such structures as buried piping in corrosive soil.

In public water systems, cement linings are used successfully for the control of corrosion on mains subject to severe corrosion.

In New Orleans, where the soils are classified by the Bureau of Standards as among the most corrosive in the United States, cement grout has been used for many years as the only means of corrosion protection for ferrous/low pressure gas service lines by New Orleans Public Service (Reference 3.8.1-30). The only corrosion failures encountered have been at locations where the cement covering has been removed by accidental damage. The reference, written in 1956, cites over 50 years successful use of this means of protection.

Stray earth currents from major direct current generating sources can, under certain conditions, develop sufficient voltage to exceed the protective range of concrete and force the concrete-encased steel to either collect or discharge current. No such sources (such as direct current railroads or mining operations) are known to have any effect at the proposed construction site. During plant construction, direct current welding generators can result in a temporary hazard if the machines are connected improperly. This was recognized and appropriate cautions followed on this project. Figure 3.8.1-43 outlines the general principles followed to avoid this difficulty.

Although concrete encasement is considered to be the only corrosion protection necessary for the steel in the containment structure (even if low resistivity corrosive soils existed), the presence of high resistivity soils at the construction site (see Section 3.8.5) operate to minimize further any possible corrosive effect in that, should stray direct current conditions ever develop, the high contact resistance between the structure and earth will minimize current pickup or discharge.

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In addition to the protection provided for the post tensioning tendons by the cement grout, the tendons are protected further by the shielding effect of the tendon containment pipe and by cathodic protection afforded the tendons by the galvanizing on the interior of the containment pipe.

The containment pipe shields the tendons in two ways:

- a) The more important effect is the positive exclusion of moisture and oxygen from the grout column and tendons. Should there be any tendency for corrosion to occur, utilizing oxygen in the process, any trapped oxygen within the containment pipe would be consumed and corrosion would cease. This, then, simply reinforces the normal protective function of the grout as discussed earlier. Further, as the moisture in the cement grout continues to combine with time and is not replenished from outside sources, the electrical resistivity of the grout will increase to high values and will act as a further deterrent to corrosion current flow.
- b) The second shielding effect is associated with the interception of any stray direct currents should they ever exist. The steel containment pipe would collect or discharge such current and prevent flow to or from the tendons in the grout column. Although the containment pipe would perform this function, the possibility of its having to do so is remote since stray direct current effects, should they ever become a problem, would normally be apparent on below-grade structures.

Although galvanized steel was selected for the containment pipe for other reasons (to prevent external rusting during construction prior to concrete emplacement), the galvanizing on the interior of the pipe provides a degree of cathodic protection to the tendons. This is because the zinc coated interior pipe surface acts as a large cathodic protection anode which tends to supply cathodic protection current to the tendons throughout the length of the column. The polarization effect of the grout-encased tendons prevents significant current discharge from the zinc such that long galvanizing life is expected.

Laboratory evaluation of the cathodic protection effect of galvanizing on grout-encased steel has been made using the grout mix proposed for use in the tendon containment pipes.

A test similar to the above was performed using anode quality zinc strip rather than galvanizing. The degree of protective potential attained is greater since the zinc strip is a purer zinc than that used for galvanizing. The degree of protection afforded by the galvanizing, however, is entirely adequate for protection of the tendons.

Assuming that it were needed, the cathodic protection afforded the tendons by the galvanized containment pipe represents the ideal level of protection since it automatically operates below the hydrogen overvoltage potential at which free hydrogen is evolved at the surface of the protected tendons.

The use of an impressed current cathodic protection system for the post stressing tendons, using an external source of direct current power, would be hazardous since, through misadjustment, such a system can operate above the hydrogen overvoltage potential.

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Possible effects of such an operation include hydrogen embrittlement of the tendons and deterioration of the bond between the tendons and grout. Such a system should be avoided since it would be difficult to check the effect on the tendons encased in the grout and damage could be incurred before an adverse condition is discovered.

3.8.1.6.1.4 Plate Steel Penetration Frames

Three categories of frames have been specified for use in the containment structure:

- a) Small Penetration Frames - covering frames in the cylinder except the equipment hatch and personnel lock frames
- b) Large Penetration Frames - covering the equipment hatch and personnel lock
- c) Dome Reinforcing Plates and Reinforcing Tie Assemblies - covering frames in the dome.

Each of these categories was specified and purchased separately. Material specifications for them were as follows:

- a) Small Penetration Frames - all steel plate and bolts ASTM-A36 (36,000 psi yield strength)
- b) Large Penetration Frames - steel sleeves ASTM A516 Grade 60 firebox quality, to ASTM A300 with 15 ft lb longitudinal V-notch charpies at -40°F. (32,000 psi yield strength) - steel web plates - ASTM A516 Grade 70 firebox quality (38,000 psi yield strength) with ultrasonic testing per ASTM A435 with 100 percent scan
- c) Dome Reinforcing Plates and Reinforcing Tie Assemblies - Reinforcing tie assemblies - ASTM A36 - Reinforcing plates - ASTM A572 Grade 60 (60,000 psi yield strength).

3.8.1.6.1.5 Liner

Specifications for liner materials are:

- a) Liner plate ASTM A442 Grade 60 or ASTM A516 Grade 60 with Charpy V-notch impact tests in accordance with ASME Pressure Vessel Code Section III at 20°F
- b) Penetration sleeves ASTM A333 Grade 1, ASTM A333 Grade 6, ASTM A304, or ASTM A516 Grade 60 to ASTM A300 with Charpy V-notch impact tests in accordance with ASME Pressure Vessel Code Section III at -40°F
- c) Forgings ASTM A350
- d) Structural steel accessories ASTM A36.
- e) The new Conax Penetration extension sleeve material is ASTM 333 Grade 6 with a Charpy V-notch impact values at -40°F.

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The liner plate was protected against corrosion as follows: The entire inside surface of the dome and walls was sandblasted. The plate from el. 228' to approximately el. 352' was coated with a zinc rich primer and modified alkyd topcoat (Keeler and long 6820/7230 system). The plate above el. 352' was coated with a zinc filled inorganic primer such as Carbozinc 11 and Phenoline 305 phenolic epoxy topcoat. The coated surfaces of the liner plate are protected by insulation and sheathing up to el. 367'10". All painted surfaces were inspected after erection and any damaged areas reprimed before finishing. The face of the liner plate in contact with the concrete has no primer or paint applied; the intimate contact with the concrete provides corrosion protection. Maintenance coatings are specified and applied equal to or better than the original coatings.

3.8.1.6.1.6 Equipment Hatch and Personnel Lock

Specifications for the equipment hatch and personnel lock materials are:

- a) Barrels and covers ASTM A516 Grade 60 firebox quality to ASTM A300 with Charpy V-notch impact tests in accordance with ASTM Pressure Vessel Code Section III at maximum temperature of 0°F.
- b) Equipment hatch flanges ASTM A516 Grade 55 firebox quality to ASTM A300 with Charpy V-notch impact tests in accordance with ASME Pressure Vessel Code Section III at maximum temperature of 0°F.

3.8.1.6.1.7 Insulation

Containment liner insulation consists of 44 in. x 84 in. x 1 1/4 in. thick, 4 lb/ft³ density cross-linked PVC foam and/or 2 lb/ft³ density Polyimide foam with an outer covering of 0.019 in. thick stainless steel. Panels are erected with the 44 in. dimension vertical and the 84 in. dimension horizontal.

3.8.1.6.1.8 Pipe Pile (See Section 3.8.5)

3.8.1.6.2 Quality Assurance

3.8.1.6.2.1 General Program

Quality of both materials and construction of the containment vessel was assured by a continuous program of quality control and inspection by Ebasco Services and Westinghouse Atomic Power Division (WAPD). Ebasco Services Incorporated was the Engineer-Constructor and, as such, had direct responsibility for inspection and quality control. An Ebasco quality control group not reporting to field production management was directly responsible for implementing and administering the inspection and quality control programs. Ebasco maintained a staff of factory inspectors who inspected and verified the quality of all manufactured products used in the construction of the containment. Ebasco was responsible for assuring quality of the materials

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used in the manufacture of the concrete and for the proper placement, erection, construction procedures and practices in the actual construction of the containment. Working under the direction of the quality control field engineer was a staff of qualified engineers and inspectors. Design engineers thoroughly familiar with the design were at the site as required during the critical phases of containment erection to ensure conformance to design standards. The testing of all steel products and concrete was done by a qualified independent testing laboratory. Radiographic inspection of welds was done under the supervision of the Nondestructive Testing Laboratory of

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Ebasco. Copies of all quality control reports were forwarded to CP&L, WAPD, and the design engineering department of Ebasco. It was the direct responsibility of the Ebasco engineering department to approve the quality control reports; however, as a parallel effort, WAPD and CP&L reviewed these reports for compliance with quality control procedures. In addition, WAPD reviewed and approved all radiographs, whether conducted in the field or in the shop of the supplier. WAPD furnished for CP&L review copies of nondestructive tests conducted by Westinghouse or its suppliers.

3.8.1.6.2.2 Prestressed Steel Tendons

The manufacturer's quality control audited by periodic visits of Ebasco Quality Compliance Representatives to the manufacturer's plant combined with the inspection of the material upon arrival at the HBR site provided assurance that prestressing bars whose ultimate strength of at least 160,000 psi were received. A description of the quality control maintained in manufacturing the prestressed bars is given in the Containment Design Report, Reference 3.8.1-1 as follows:

- a) The hot-rolled bars were purchased in accordance with ASTM A-29 and ASTM A-322. The bars were AISI 5160, with its chemistry modified as permitted by Section 3 of ASTM A-322-64. Mill test reports (chemical and physical properties) for each heat of bar material used were submitted with each shipment. Processing the bars consisted of cold working, furnace heating, threading, and assembling. Heat identification of all bars was maintained at all times after their receipt. In addition, all bars received for the HBR project were stored in a pre-established area of the shop.
- b) Cold working involved pulling a bar to a predetermined minimum force of 208,000 lb. This load was determined by a machine gage and light bulb which indicated when the required hydraulic pressure was obtained. The function of the gage and the light was to indicate that the prescribed minimum elongation for each bar was met.
- c) Heating the bars at 700°F for four hours improves the yield strength and ductility. The heating cycle was documented by using temperature recorders. After furnace heating, the bars were cut to length, threaded, and assembled into tendons. The bars were identified at each stage of the process.
- d) Four unprocessed bars from each heat of material to be used were randomly selected upon receiving them from the bar manufacturer and sent to an independent laboratory for tensile testing. After a minimum of 50 percent of a heat of bars was cold stretched and stress relieved, four random samples were selected and tensile tested by an independent laboratory. The heat of steel tested was acceptable if the minimum required properties were obtained.
- e) The couplers which are AISI 4130 steel require that a mill test report for each heat used in the manufacturing of the couplers must be submitted for approval and that the threads be magnetic particle inspected. From each heat of material used, one coupler was randomly selected from every 200 pieces manufactured and tested, with threaded bars to determine if the coupler would develop a minimum ultimate tensile strength of 238 kip. The batch of couplers from which the tested sample came were rejected if required minimum ultimate strength was not met. A minimum of two samples were tested from each batch. All couplers were color coded.

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- f) The grip nuts were machined from open hearth quality hot rolled AISI 8620 steel. A mill test report for each heat of material used was required. Each grip nut was magnetic particle inspected and hardness tested. From each manufacturer's batch, in accordance with Ebasco specifications, one grip nut was randomly selected from every 200 pieces and tensile tested with a bar to 238 kip. The batch of grip nuts from which the tested sample came was rejected if the required minimum ultimate strength was not met. A minimum of two samples were tested from each batch. The anchorage plates, which are a hot-rolled AISI 1040 steel, required that a mill test report for each heat of material used be submitted for approval. The drilled and tapped holes were visually inspected.
- g) The following is a list of documents which assured quality control coverage of all phases of tendon manufacture:
 - 1) Mill shipping list
 - 2) Bar receiving record
 - 3) Bar stretching record
 - 4) Stress relieving record
 - 5) Temperature control charts of stress relieving furnace
 - 6) Inspection report - upper tendon assemblies
 - 7) Physical properties - test reports.

Quality assurance measures that were implemented to assure that the tendon bars were in a satisfactory physical condition with respect to corrosion when encased in grout consisted of flushing the tendons with an alcohol - VPI solution and pumping and pressuring them with dry nitrogen gas. They were then kept under a dry nitrogen atmosphere until prestressing and grouting operations started. A dry nitrogen atmosphere is a well proven and long accepted method of providing a corrosion-free product when stored over long periods of time. As a result, the tendons were judged to be in a condition acceptable for grouting.

The condition of the tendons was checked by the use of tell-tale bars put into the tendons at the time of placing and left in until prestressing operations began, at which time they were removed and inspected for corrosion.

Minor corrosion which occurred on the lower tendon assemblies early in the construction phase was quickly stopped and an investigation at that time concluded that the tendons did not suffer any loss of strength.

3.8.1.6.3 Special Construction Techniques

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3.8.1.6.3.1 Construction Tests

The following construction tests were conducted:

- a) Bottom Liner Plates - All liner plate welds were tested for leaktightness by a vacuum box. After completion of a successful vacuum box leak test, the welds were covered by channels. A strength test of channel welds was then performed by applying 48.3 psig air pressure to the channels in the zone for a period of 15 minutes.

In addition, the zone of channels was held at the 42 psig air pressure using a Freon-air mixture for a period of at least two hours, with no more than 0.5 psi drop in pressure. Compensation for change in ambient air temperature was made as necessary.

- b) Vertical Cylindrical Walls and Dome - Liner plate seam welds in the cylindrical walls and dome were spot radiographed as follows: For each 50 lineal ft (maximum) of welding by each welder a 12 in. (minimum) radiograph was made.

ASME Pressure Vessel Code, Section VIII, specifies one percent of spot radiography, but two percent spot radiography was used as an aid to quality control on full penetration butt welds.

ASA N6-2 states that for parts of containment structures where the steel plate serves primarily as a membrane liner to reduce leakage, radiography of seam weld would not be required. However, in the interests of conservatism, and in accordance with Quality Control Procedures, the liner was spot radiographed two percent. The bottom three rings were magnetic particle tested.

In those instances where back-up plates were used with liner welds and radiography was unsatisfactory, the welds were magnetic particle tested at the intersection of the liner plates for a distance of one foot in each direction. This resulted in approximately seven percent magnetic particle testing of liner welds. In addition all back up plate liner welds were 100 percent vacuum box tested.

The inspection for liner attachment studs was as described below. The first two studs welded to each plate, after being allowed to cool, were bent approximately 34° by striking the stud with a hammer. If failure occurred in the welds of either stud, the welding procedure or technique was corrected and two successive studs successfully welded and tested before further studs were attached to the liner. All studs welds were visually inspected, and studs on which a full 360° weld was not obtained were repaired by adding a 1/8 in. fillet weld in place of the insufficient weld, using the shielded metal arc process with low hydrogen welding electrodes.

Before welding a new stud where a defective one was removed, the area was ground flush and smooth.

Any stud which did not show a full 360° weld or any stud which had been repaired by welding was struck with a hammer and bent 15°. For studs showing less than 350° weld prior to repair, the direction of bending was opposite to the lack of weld. The procedures outlined above for the first two studs on each plate were then followed.

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The penetration welds were spot radiographed. After successful spot radiograph, the welds were covered by steel angles, channels, or pipe sections welded to the liner and the assemblage strength tested to 48.3 psig for a period of 15 minutes. On successful conclusion of the strength test, halogen sniffer tests were performed on all penetration welds at 42 psig pressure. The detection of any amount of halogen indicated a leak requiring weld repairs and retesting. The zone under pressure was held under 42 psig for a period of at least 2 hr with zero drop in pressure.

- c) Penetrations - Strength and leak tests of individual penetration internals, closures, and sleeve weld channels were performed in a similar manner to the above, with all leaks repaired and retested until no further leaks were found.

Shop leak testing procedures were conducted in accordance with ASA N6.1965. A proof test was applied to each penetration which pressurized the necessary areas to 48.3 psig. This pressure was maintained for a sufficient time to allow soap bubble and Freon sniff tests of all welds and mating surfaces. Any leaks found were repaired and retested, with this procedure repeated until no leaks existed.

- d) Onsite Testing of Materials - Concrete was tested in accordance with ASTM C 172, C 31, C 192, C 39, and C 231. Standard slump tests were made in accordance with ASTM C 143. Aggregate was tested in accordance with ASTM C33. Water tests were in accordance with ASTM D 512.

Reinforcing steel tensile strength tests were on full bars as a by-product of tests on splices. Random sampling tests of sample heat bars were by full scale tensile testing.

One tensile strength test was conducted on a representative sample of tendon material for the prestressing steel from each heat of steel used. (Prestressing is a full scale tension test beyond the stress value for the factored load condition).

Each mechanical splicing crew received instructions from a representative of the supplier and was required to prepare several acceptable sample mechanical splices for each bar size.

All Cadweld splices were visually inspected to assure that sound filler metal was present at both ends of the sleeve and in the tap hole. This inspection was made on the same day that the splice was made by an inspector working for the Quality Control Engineer. An inspection manual was issued to guide the inspector in his judgment of a satisfactory splice. Any splices in doubt as to integrity were cut out and replaced.

The Cadweld splices made by each crew were randomly sampled by cutting out the splice and a length of bar at each end of the splice and tension testing to failure in an independent testing laboratory. The schedule for testing and sampling was approximately as follows for each crew:

- a) 4 out of the first 100 splices for each bar size and grade
- b) 3 out of the next 100 splices, and

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- c) 2 out of the next and subsequent units of 100 splices.

A continual review of the test data was performed and if any of the splices failed to develop 90 percent of the minimum ultimate strength of the bar or if two splices in sequence failed to develop the full minimum tensile strength, the production of that crew was halted until an investigation was made to determine the cause and extent of the defective splices, and corrections were made to assure that further splices met the specification strength. Before returning to production, the crew was required to make several sample splices and qualify as a new crew if their splicing procedures were found inadequate by the investigation. It is believed that the most probable cause for poor Cadweld splices was the performance of the splicing crew. All material was subjected to quality control inspection procedures in the shop before shipment, which gave a high degree of reliability to the materials and components used.

3.8.1.6.3.2 Grouting of Prestressing Steel Tendons

The following are applicable to grouting of prestressing steel tendons:

- a) Construction Procedure - A STRESSTEEL MGP-3L6 Mixer and grout pump unit with two 50 ft lengths of hose were used. The equipment was designed to mix and pump grout continuously. The mixing tanks were marked for the prescribed volume of water and provisions were made for rapidly refilling the tanks when they were empty. After the measured quantity of water was in a mixing tank, cement was placed in mixing tanks by placing bags over the tank screen, breaking the bag, and sifting cement through the screen. Admixture was added from pre-measured packages by hand after all the cement was placed. The materials were mixed and continued to run for a minimum of 5 min after the admixture addition was made before discharging into the agitator tank. Based on test samples, water temperature was adjusted to give a temperature of the mixed grout of 50° - 90°F.

The grout pump outlet hose was connected with a threaded fitting to the 1 1/2 in. grout pipe for the tendon to be grouted. As soon as a batch of grout was finished mixing and discharged into the agitator tank, the pump was activated and pumping started. Pumping was carried out continuously, as grout was available from the mixers, until grout covered all hardware at the top of the tendon. The inlet valve from the grout pump was then closed, the hose removed and connected to the grout pipe on the next tendon to be grouted. All water and other foreign material (if present) was removed from the top of grout, and grout allowed to freely expand. After expansion stopped, grout required to fill stand pipe was added and cap screwed on.

Agreeing with the latest procedures of the prestressing industry, it was decided that flushing the tendon with lime water was not necessary.

Prior to placing concrete in the mat and lower wall, the first stage or lower portion of the tendons containing the 1 1/2 in. diameter grout pipe was thoroughly probed with a plumber's snake and then purged with an alcohol and VPI solution.

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The tendon sheaths consist of schedule 40 pipe having a wall thickness of 0.280 in. After adding the second stage or upper portion of the tendons, these tendon sheaths were kept under a constant positive pressure nitrogen atmosphere, which was sufficient to prevent grout leaking in if the pipes leaked. They were also inspected just prior to concreting and no damage was reported. The fact that the sheaths were capable of maintaining the gas pressure within them provides assurance that they could not have allowed cement mortar to leak into them during concreting and cause a grout blockage.

Segregation was avoided in long vertical ducts by using a grout mixture which contained a nitrogen-producing expansive agent (Intracrete, by Sika Chemical Co.) and a gelling agent. Tests demonstrated that grout with this admixture was homogeneous, did not bleed, and did not leave voids or any type of segregation.

- b) Grout Materials and Properties - The grout materials used were specified as follows:
- 1) 1 sack of Type III Portland Cement meeting ASTM Specification C-150-68
 - 2) 1 lb of admixture. Admixture used in grout was "Intracrete-Gel" as manufactured by Sika Chemical Corporation, Lyndhurst, New Jersey. The amounts of nitrates, aluminum, chlorides, and hydrogen in this admixture were undetectable. The maximum amount of nitrogen generated by this admixture was 14 grams in 100 liters of grout
 - 3) 6 gal of potable water. Water used in grout was periodically analyzed by independent laboratories for the chloride content, with a specified limit of 100 ppm, in accordance with ASTM-D-512, and was certified free from injurious amounts of oil, acid, alkali, organic matter, and other deleterious substances and considered chemically satisfactory
 - 4) As a measure of fluidity, a flow cone efflux time was specified as 75 seconds maximum, and
 - 5) Actual expansion in the first two tendons that were grouted in the field was measured and correlated to 50 in. high graduated test cylinders from the same batch.
- c) Corrosion Effects of Grouting - The major imperfection of the grouting was the possible existence of voids or "lenses" which might occur as the result of a temporary interruption in the filling process or shrinkage during setting.

For these lenses to function as corrosion cells it would be necessary that they contain both air and water. There may have been some uncombined alkaline water which had not yet been absorbed into the drying grout, but the gas present in the void would most likely be nitrogen from two possible sources: the nitrogen resulting from the curing action of the grout or the nitrogen remaining from the inert atmosphere used to preserve the tendons from corrosion.

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There should have been no oxygen present since the grouting procedure of filling from the bottom-up displaced the nitrogen out of the top of the tendon. The tendon design is such that once the grout was within the sheath no air could leak in. However, assuming the most severe condition where small amounts of both oxygen and water were present, the corrosion cell thus formed would last only until the oxygen was used up. Since there was no chance of oxygen replacement, the oxygen would be used up and the corrosion stopped. During this period the galvanized lining of the sheath furnished a measure of cathodic protection to the tendons.

d) Testing

- 1) Contraction and Expansion of Grout - Six tests were performed in 3 in. diameter transparent plastic tubes 11 ft high, with two 3/4 in. bars and couplers. Two tests were performed in 6 in. diameter transparent plastic tubes 30 ft high. The two latter tests had six bars, couplers, and a top anchorage. They were modeled to simulate the actual structure, except for the height of the test.

The total expansion measured in these tests was approximately 5 percent; and observation, after the grout had set, showed no contraction or formation of voids of any nature.

- 2) Fluidity and/or Viscosity of Grout - Fluidity of the grout was evaluated using the flow cone rate, but this reading was recorded more from an academic point of view than as a control per se. It was agreed prior to performing these tests that the water-to-cement ratio would be held at 6 gal per sack of cement for all testing. During the 11 ft high tests, the ambient temperature was about 68°F, the grout temperature after mixing was 75°F, and the flow cone reading was 23 seconds. During the full-size 30 ft high tests, the ambient temperature was about 80°F and the grout temperature 94°F. The grout in the first test model of these full-size tests had a flow cone reading of 60 seconds. It was so thick that it would not flow easily from the mixer to the hopper. Even under these adverse conditions, no difficulty was experienced in pumping the grout.

Tests made in the field prior to grouting established the pumpability of the grout mix for the ambient temperatures existing at the time.

e) Quality Assurance Procedures - Additional quality assurance procedures were specified as follows:

- 1) In the event of short delays (over 1/2 hr) the grout in the pump was dumped and the equipment washed out.
- 2) Grouting equipment was thoroughly cleaned at the end of each working day.
- 3) Prior to pumping, the density of grout was measured to determine quality of mix.
- 4) Ambient and material temperature was recorded.

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- 5) Flow cone efflux time was recorded.
- 6) Time of mixing, agitating, and grouting was recorded.
- 7) Expansion was observed and recorded.

3.8.1.7 Testing and In-Service Surveillance Requirements

3.8.1.7.1 Structural Integrity Pressure Test

A pressure test was made on the completed building using air at 48.3 psig. This pressure was maintained on the building for a period of one hour. During this test, measurements and observations were made to verify the adequacy of the structure design.

Figures 3.8.1-44 through 3.8.1-48 illustrate the comparison between deflections and stresses for the design accident condition and the test pressure condition.

The general acceptance criterion for the pressure test is identified in the Containment Design Report (Reference 3.8.1-1).

A structural test program of the containment shell was carried out after construction was completed in order to provide additional assurances of the adequacy of the structural design. The structural tests were made by pressurizing the containment to 115 percent of design pressure (48.3 psig) for a period of one hour and measuring the movement of the structure at selected locations as described below. A detailed plan for testing (including predicted and tolerable deflections) was prepared.

The structural performance was checked by comparing the actual deflections under test with the predicted values. The areas of greatest interest were:

- a) The cylinder wall at points where it is in pure membrane stress
- b) The cylinder wall at points of discontinuity caused by junction to another structural component
 - 1) Springline
 - 2) Base
- c) The cylinder wall in the areas of the equipment hatch and personnel lock
- d) The base slab at the perimeter, and
- e) The reactor sump structure.

To determine the degree of agreement between predicted and computed performance in these areas, an instrumentation program was developed with the aim of being able to trace the deflection pattern of the outside surface of the wall measured in a vertical line at four quarter points on the cylinder. In addition, the longitudinal and circumferential growth of the cylinder from base to the spring line was measured at these quarter points. Radial movement of the outside surface of the wall was measured on four lines radiating away from the equipment hatch and personnel lock. Vertical movement of the base slab adjacent to the wall was measured at the quarter points.

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To match the type of instrumentation to the magnitude of measurement, calibrated targets attached to the wall and high magnification theodolites were used to measure radial movement at the springline and upper portions of the wall where movements of about 1/2 in. were expected. Theodolites are capable of a precision of ± 0.01 in. when used in such situations. This was sufficient for this magnitude of measurement.

Measurements at the base of the wall and around the two large penetrations were made by means of dial gauges capable of measuring to .001 in.

Longitudinal and circumferential growth of the wall was measured by means of electrical strain gauges attached to the face of the liner to the insulation immediately prior to testing in an area subjected only to membrane forces. Three gauges in each direction were used to reduce the difficulty of interpretation. Unbonded and unstressed gauges were placed alongside the bonded gauges to allow for thermal effects.

Dial gauges were used to determine the movement of the base slab.

To provide further information on the basic hypotheses, meridional and tangential strains were measured on the dome liner near the peak where pure membrane strains occur. These strains were measured along the same quarter points as strains gauges. To reduce the difficulty of reading interpretation, three redundant gauges with three dummy gauges (non-strain reading) for thermal compensation were placed for each measurement desired.

The reactor sump structure walls were instrumented to measure physical movement at several elevations above the sump bottom slab by means of pin-supported bars with redundant and dummy strain gauges attached.

To ascertain the degree of conservatism in the assumption on vertical crack spacing in the cylinder wall, a program to measure the average, minimum, and maximum spacing was pursued at three elevations:

- a) Base
- b) Area of membrane stress only, and
- c) Elevation of top tendon anchorage.

These cracks were measured at the quarter points for a circumferential distance of 10 ft using stress sensitive paint to make any cracking more discernible.

The crack pattern in the area of the large penetrations were visually checked to confirm consistency with the predicted tension stress pattern.

The expected magnitudes of readings and the tolerable differences between predicted and measured were developed. These tolerable differences were dependent upon several factors - among them being:

- a) The magnitude of the measurement
- b) The amount of conservatism factored into the predicted value, and

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- c) The type of instrumentation.

In the event that any measurements were outside the tolerable differences, an analysis was made to determine the reasons for and the implications of the difference.

The measurement program conducted concurrently with the pressure test was basically that described in VIII E.(1) (b), Fourth Supplement to PSAR, modified as follows:

- a) Linear variable differential transducers (LVDT) were employed instead of dial gauges where radial displacements of the wall and base, up to a height of about 10 ft., were measured. The LVDT and associated instrumentation have a resolution of better than 0.001 in. and an accuracy of 0.002 to 0.005 in.
- b) Radial displacements of the wall at quarter points 10 ft or more above the base were measured by means of transits sighted on scales mounted on these points.
- c) The vertical movement of the top of the wall was measured with suspended invar tapes with tensioning weights and attached scales.
- d) LVDT were used to measure out-of-plane displacements within the reactor sump. These were located in the center of each of the walls and the floor, and were supported on telescoping columns with pinned ends to eliminate the effects of their end motions.

No permanent instrumentation was installed to monitor behavior of the structures. The design, fabrication, inspection, and preoperational testing of the containment building ensures a structure capable of offering continued structural integrity over the life of the plant. It is therefore believed that there is no need for a special insurance surveillance program other than visual inspection of the exposed surfaces.

No post-operational structural surveillance program has been established.

The purpose of the structural proof test is to demonstrate the structural design and as-constructed adequacy of the containment. No mechanism has been identified which has not been considered in design that would lead to the degradation of this containment after initial proof testing and for this reason periodic structural proof testing is not required.

3.8.1.7.2 Tendon Surveillance

The applicant, its engineer-constructor, and its consultants believe that there is sufficient evidence in the history of the prestressed concrete industry to justify the specifying of an uniaxially prestressed concrete containment vessel such as the H.B. Robinson containment with full confidence that it will perform within the criteria set in its design. Conservative values have been used in estimating qualities of materials which affect the net prestressing force.

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As an example of the conservatism used, consider creep and shrinkage. Design values of 0.0003 in. shrinkage and 2.25 for coefficient of creep (creep stain/elastic strain) were specified. It is expected that values of 0.0001 in. shrinkage and 1.7 for coefficient of creep are realistic values based on preliminary estimates using as a guide (Reference 3.8.1-17).

Such a conservative design can only result in higher precompression stresses in the concrete and higher tensile stresses in the tendons. This is of little interest, since even with these higher tensile stresses, the tendons will never reach the tensile stress imposed upon them with the initial prestressing operation.

There is no practical method of surveying the tendon stress and corrosion, creep and shrinkage of the concrete for a grouted tendon. Known conservative analytical procedures, in addition to successful experience application for grouted tendons, do not warrant a surveillance program. However, two surveillance tendons similar to the service tendons and in a similar environment are provided. These may be uncovered at any time for surveillance of any corrosion.

The surveillance tendons consist of two short tendons similar to the service tendons. Each tendon consists of six - 1-3/8 in. \varnothing bars in 6 in. pipe sheath with anchor plates, prestressing hardware, and grout pipe identical except for length to the working tendons. They are embedded in a section of concrete approximating the same environment as that of the service tendons.

The program for inspection consists of removing one tendon after 5 years and the other after 25 years.

The removed tendons are sent to a commercial laboratory qualified to perform material tests and analysis. The tendon bars are removed from the sheath and the grout removed. The visual inspection is performed to detect and record evidence of corrosion. Tensile tests are then performed on selected bars to develop stress-strain diagrams and determine the bars' ultimate tensile strengths. The results of these tests are compared with the original properties to determine any significant changes. DEP retains a qualified engineering firm to assess the results of these tests and make recommendations.

The first containment surveillance tendon sample was removed in March, 1976. The reports detailing the results of examination are contained in References 3.8.1-31 through 3.8.1-33. The second containment surveillance tendon sample was removed in April, 1997. The report detailing the results of the examination is contained in Reference 3.8.1-34.

Based on the information presented in the reports, and on other available data on the Robinson containment system, the tendon surveillance program is judged to be satisfactory. The tests showed that surveillance tendon specimens tested exceeded the minimum breaking load of 238,000 pounds given in the FSAR. It can be reasonably concluded that similar results would be obtained if bars from the actual containment tendons were tested.

3.8.1.7.3 Initial and In-Service Leakage Rate Tests

Initial and in-service leakage rate tests are discussed in Section 6.2.6.

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TABLE 3.8.1-1

WEIGHTS OF MAJOR EQUIPMENT WITHIN CONTAINMENT STRUCTURE

<u>EQUIPMENT</u>	<u>NUMBER</u>	<u>TOTAL DRY WEIGHT, lb</u>
Reactor Vessel	1	790,639
Steam Generator	3	1,755,600
Pressurizer	1	175,400
Reactor Coolant Pump	3	576,300
Air Recirculation Unit	4	350,000
Accumulator Tank	3	225,000
Polar Crane	1	440,000*

* The total weight of the crane has increased 4400 lbs. or approximately 1%, due to modifications made in the fall of 1983 and in EC86800. This change is considered insignificant.

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TABLE 3.8.1-2

CONCRETE TEST CYLINDER DATA

	<u>MINIMUM NUMBER OF CYLINDERS</u>	<u>TEST BREAKS</u>			
		<u>3 day</u>	<u>7 day</u>	<u>28 day</u>	<u>Extra</u>
Until final determination of each design maximum	4	1	2	1	
For each Concrete Placement					
10 to 50 cu yd	3		1	1	1
50 to 100 cu yd	6		2	2	2
over 250 cu yd	9		3	3	3

The extra cylinders are only tested if it is necessary to substantiate 7- or 28-day test results.

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TABLE 3.8.1-3

REPEATED LOAD TEST OF STRESSTEEL BARS WITH VARIOUS FITTINGS

(All tests performed at Lehigh University)

<u>TEST OF</u>	<u>DIAMETER OF BAR</u>	<u>STRESS RANGE</u> ksi	<u>STRESS CHANGE</u> ksi	<u>CYCLES</u>	<u>REMARKS</u>
Howlett Grip Nut (11-28-66)	1-3/8	96 - 112	16	1,343,400	No indication of failure
COUPLER (11-27-57)	1-1/8	85 - 100	15	363,800	Failure
	1-1/8	85 - 95	10	2,120,000	Failure
	1-1/8	75 - 90	15	454,000	Failure
	1-1/8	85 - 95	10	1,659,000	Failure
	1-1/4	85 - 95	10	750,600	Failure
	1-1/4	85 - 95	10	715,800	Failure
	1-1/4	87.5 - 95	7.5	3,001,500	Specimen did not fail
					Bar tested statically
					Failed at 153 ksi
Wedge Anchor Assembly 12-12-56	1-1/8	85 - 100	15	3,005,900	Specimen did not fail
					Bar tested statically
					Failed at 155 ksi

Note: All tests run at 500 cycles/min.

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TABLE 3.8.1-4

HOWLETT GRIP NUT TEST REPORTS NO. 1 AND 2

MATERIAL: 1 Pc - 1-3/8 in. ϕ Stressteel Bar 1 ft - 6 in.

Special Grade - Threaded Two (2) Ends

1 Pc - 1-3/8 in. ϕ Stressteel Bar 2 ft - 0 in.

Special Grade - Threaded One (1) End

1 Pc - GN11 Grip Nut

1 Pc - TC-11 Threaded Coupler

1 Pc - TP-11 Threaded Plate (6x2x6)

Tapped Plate

Above material used to assemble one test specimen. The same specimen was used for both tests shown below.

FIRST TEST

Min. Load	Max. Load	Remarks
142.5 kip or 0.6 f's	156.9 kip or 0.66 f's	Testing speed = 500 cycles per min. Stopped test after 806,400 cycles with no indication of failure

SECOND TEST

Min. Load	Max. Load	Testing speed = 500 cycles per min.
142.5 or 0.60 f's	166.0 kip or 0.70 f's	Fracture of bar in threads of rod inside threaded plate in 407,700 cycles

Note: Bar sections used in test from Heat No. 528S0096

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TABLE 3.8.1-5

HOWLETT GRIP NUT TEST REPORT NO. 3

TEST NO. 3

Test Description

Fatigue test of GN11 grip nuts with front threads relieved.
Load to 143 kip minimum (.6 ultimate) to 167 kip maximum
(.7 ultimate) for 2 million cycles minimum.

2 - GN11 with inert threads relieved
2 - EP11 plates 7 x 2 x 7-1/2 in. ø hole
1 - Pc - 1-3/8 in. bar x 5 ft 392A - no threads

Test Results

Load Max. in Kip	Load Min. in Kip	Cycles (millions)	Remarks
167	143	2 (2,020,800) ^x	No failure occurred Testing was stopped according to the special orders in the letter of January 16, 1968 x machine stopped Spec. No. 392A No threads No splice

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TABLE 3.8.1-6

HOWLETT GRIP NUT TEST REPORT NO. 4

TEST NO. 4

Test Description

Fatigue test of CN11 grip nuts with front threads relieved.
Load to 143 kip minimum (.6 ultimate) to 178.5 kip maximum
(.75 ultimate) for 2 million cycles minimum.

2 - GN11 with insert threads relieved
2 - EP11 plates 7 x 2 x 7-1/2 - 1-1/2 in. hole
1 - Pc - 1-3/8 in. bar x 5 ft 392B - no threads

Test Results

Load Max. in Kip	Load Min. in Kip	Cycles (millions)	Remarks
178.5	143	2 (3,274,000) ^x	No failure occurred. Testing was stopped according to verbal instructions by Walter H. Larson April 12, 1968. Also see letter dated April 3, 1968 x machine stopped Spec. No. 392A No threads

3.8.2 STEEL CONTAINMENT

The H. B. Robinson containment is concrete with a steel liner as discussed in Section 3.8.1.

3.8.3 CONCRETE AND STRUCTURAL STEEL INTERNAL STRUCTURES OF THE CONCRETE CONTAINMENT

3.8.3.1 Description of the Internal Structures

3.8.3.1.1 Primary and Secondary Shield Walls

The Reactor Coolant System is surrounded by concrete shield walls. These walls provide shielding to permit access into the containment during full power operation for inspection and maintenance of miscellaneous equipment. These shielding walls also provide missile protection for the containment liner plate.

3.8.3.1.2 Steam Generators Support Structure

The steam generators are supported on a structural system consisting of four connected columns all welded together, fabricated of carbon steel members, with provisions for limited movement of the structure in a horizontal direction to accommodate piping expansion with a system of "Lubrite" plates, hydraulic snubbers, guides, and stops. The "Lubrite" plates, hydraulic snubbers, guides, and stops are designed as damped support to resist the action of seismic and pipe break loads. Sliding shoes at the top of the support structure permit radial thermal growth of the steam generators during heatup.

Steam generator lateral bracing is provided near the upper tube sheet elevation to resist lateral loads, including those resulting from seismic forces and pipe rupture forces. Additional bracing is provided at a lower elevation to resist pipe rupture loads.

3.8.3.1.3 Reactor Vessel Support Structure

The reactor vessel support structure consists of a circular box section ring girder, fabricated of carbon steel plates. The bottom flange of the girder is in continuous contact (except for openings for neutron detectors) with a non-yielding concrete foundation.

The reactor vessel has three supports located at alternate nozzles. 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. Each support shoe is designed to restrain vertical, lateral, and rotational movement of the reactor vessel, but allows for thermal growth by permitting radial sliding on bearing plates.

3.8.3.1.4 Reactor Coolant Pump Support Structure

Each reactor coolant pump is supported on a three-legged structural system consisting of three connected columns fabricated of carbon steel members, structural sections, and pipe. Provisions for limited movement of the structure in any horizontal direction to accommodate piping expansion is accomplished with a sliding "Lubrite" base plate arrangement and a system of tie rods and anchor bolts which restrain the structure from movement beyond the calculated limits. Sliding shoes at the top of the support structures permit radial thermal growth of the pumps during heatup.

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3.8.3.1.5 Pressurizer Support Structure

The pressurizer is supported on a heavy concrete slab spread between the concrete shield walls of its compartment. The pressurizer is a bottom skirt support vessel, resting on a type of ring girder.

3.8.3.2 Applicable Codes, Standards, and Specifications

See Section 3.8.1.2.

3.8.3.3 Design and Analysis Procedure

3.8.3.3.1 Concrete Internal Structures

3.8.3.3.1.1 Primary and Secondary Shield Walls

The primary shield has been designed for a uniform temperature of 50°F and for thermal gradients as shown on Figures 3.8.3-1 through 3.8.3-4.

The crane wall has been designed for uniform temperatures of 120°F and 50°F.

All thermal stress calculations assumed 70°F as a starting temperature.

The methods used for the design of the primary and secondary shields for both normal and accident conditions including critical stresses were as follows:

- a) Crane wall and Internals-Structure - This structure was treated as a space frame made up of linear elements, fixed at the base mat and the primary shield wall. The analysis was performed using the Service Bureau Corporation's Frame Analysis Program (FRAN) under the following loads:
 - 1) Dead load
 - 2) Jet forces at limiting points within each of the three compartments
 - 3) Internal pressures in each of the three compartments.

The critical stresses were:

1) Horizontal Elements

EI 247.50:

Concrete: - 1,520 psi (compression); 219 psi (shear)*

Reinforcement: +30,530 psi (tension)

EI 236.50:

Concrete: - 822 psi (compression); 129 psi (shear)*

Reinforcement: +30,890 psi (tension)

* Concrete reinforced for shear (stress is shear on gross section)

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Concrete: - 937 psi (compression)
Reinforcement: +20,280 psi (tension)

2) Vertical Elements

EI 236.5 concrete: 426 (psi) (shear)*
EI 227.0 (Bottom of Walls)

Concrete:	- 2,050;	- 2,170;	- 1,920	psi	(compression)
Reinf:	+28,640;	+29,070;	+25,370	psi	(tension)

*Concrete reinforced for shear (stress is shear on gross section)

The type of model used in this analysis is shown in Figure 3.8.3-5.

b) Primary Shield Wall - This structure was treated as a thick cylindrical shell cracked section with the following loads:

- 1) Dead load
- 2) Earthquake
- 3) Jet force
- 4) Compartment pressure
- 5) Internal pressure
- 6) Temperature

The following critical stresses were found:

- 1) Earthquake + Dead Load + Compartment Pressure + Jet Force

Vertical stresses:

Concrete = 240 psi (compression)

- 2) Dead Load + Thermal (Case I, Fig. 3.8.3-1)

Vertical stresses:

Concrete = - 780 psi (compression)

Steel = +3,600 psi (tension)

Hoop stresses:

Concrete - 127 psi (compression)

Steel 12,650 psi (tension)

- 3) Internal Pressure + Thermal + DL

Hoop stresses:

Inside Steel: +37,300 psi (tension)

Outside Steel: +35,750 psi (tension)

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The methods used for the computation of the jet forces on the primary and secondary shield walls due to a pipe break are as follows:

The jet force associated with a pipe break may be determined from the fluid momentum equation in the discharge pipe. For a free jet this axial force is conserved from plane to plane downstream of the discharge orifice. The force is given by:

$$F_j = P_1 A_{\text{pipe}} + W V_1 - \Delta P_{\text{frict}} A_{\text{pipe}} \quad (1)$$

where: F_j = the total jet force

P_1 = fluid pressure at a point 1 within the discharge pipe

V_1 = fluid velocity at a point 1 within the discharge pipe

W = the mass discharge rate

ΔP_{frict} = the friction pressure drop in the pipe between location 1 and the pipe exit. (ΔP_{frict} is negligible for short pipe).

A_{pipe} = the discharge area.

To eliminate complications associated with downstream fluid flashing and two phase flow, Equation (1) is evaluated at the vena contracta at which point single phase flow is assumed. The fluid velocity is given by a mechanical energy balance:

$$V_1 = \frac{2(P_r - P_1)}{\rho} \quad (2)$$

where: P_r = the pressure in the connected reservoir

ρ = fluid density

The mass flow rate is given by:

$$W = 0.6 \rho A_{\text{pipe}} \sqrt{\frac{2(P_r - P_1)}{\rho}} \quad \text{for a short pipe} \quad (3)$$

Substitution of (2) and (3) into (1) gives for a short pipe:

$$F_j = (1.2 P_r - 0.2 P_1) A_{\text{pipe}} \quad (4)$$

For cold fluid discharge, $P_1 \sim 0$.

For saturated fluid discharge, P_1 has been observed to increase from 0 to $0.55 P_r$ as the pipe length to diameter ratio increases from 0 to 7 because of downstream choking effects (Reference 3.8.3-1).

On the basis of the above, for hot subcooled fluid in the reservoir, P_1 is expected to vary from 0 to $0.55 P_{\text{sat}}$ as pipe L/D increases, where P_{sat} is saturation pressure. Therefore a jet force of $1.2 P_r A_{\text{pipe}}$ has been used in the design.

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Observations of visual jet expansion in discharge tests through short pipes indicate that for subcooled blowdown of hot fluid the jet expansion approximates a nearly hemispherical explosion (expansion half angle greater than 45°). During saturated blowdown the jet is more confined, having a half angle of about 15°.

The above estimates may be used to determine the area over which the jet force is spread. A structure intercepting the entire jet thrust would respond to force F_j .

3.8.3.3.1.2 Control Rod Drive Mechanism

The control rod drive mechanism is effectively shielded from missiles generated by a LOCA by the reactor cavity wall which is 6 ft thick reinforced concrete. The types of missiles considered were the same as for the crane wall. The 3 ft thick crane wall was checked and shown to preclude the possibility of missile penetration. Thus, the 6 ft thick reactor cavity wall will preclude missile penetration.

3.8.3.3.1.3 Reactor Coolant System Supports

The supports for the reactor coolant system have been designed to withstand the blowdown forces associated with the sudden severance of the reactor coolant piping so that the coincidental rupture of the steam system is not considered credible. Equipment supports design criteria are detailed in Section 3.8.1.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

In addition to the containment building, the following structures are designed to Seismic Category I requirements:

- a) Reactor Auxiliary Building
- b) Spent fuel pit
- c) Class I section of the Turbine Building
- d) Intake structure
- e) Concrete masonry walls in proximity with safety-related systems and equipment.

3.8.4.1 Description of the Structures

3.8.4.1.1 Reactor Auxiliary Building

The Reactor Auxiliary Building (RAB), including the control room and the diesel generator room, is a Class I structure and has been designed in accordance with the procedures described for the containment structure (see Section 3.8.1).

Since the original construction, a waste evaporator enclosure has been installed on the roof of the building (Figure 3.8.4-1). The design and analysis report for this Seismic Class I structure installation is contained in Reference 3.8.4-1.

The existing RAB, including the pile foundation, was analyzed to verify its structural adequacy to withstand all the original design loads plus the additional loads imposed by the waste evaporator enclosure and its associated equipment.

3.8.4.1.1.1 Method of Load Analysis

To determine the additional load imposed on the existing support walls, the design loads were considered to act in the following load combination:

$$D \pm 0.05D + 1.0 \text{ floor } L + 0.5 \text{ roof } L + L' + (W_T \text{ or } E')$$

To determine the additional load imposed on the existing pile foundation, the design loads were considered to act in the following load combination. The pile foundation was also checked for additional uplift and horizontal shear capacity.

$$D \pm 0.05D + 0.25 \text{ floor } L + 0.5 \text{ roof } L + L' + (W_T \text{ or } E')$$

The existing design roof live load, in the area of the enclosure, was deducted from the loading imposed on the existing structure.

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The analysis was based on the concrete design being performed according to the ACI 318-63 code using the ultimate strength method, as were the original calculations. The location of the center of gravity of the waste evaporator enclosure was determined in order to determine the additional loads on the existing Reactor Auxiliary Building pile group. The location of the center of gravity of the Reactor Auxiliary Building including the waste evaporator enclosure was also determined.

3.8.4.1.1.2 Result of Load Analysis

Using the design load combination specified, a new maximum pile compression load was determined. The new maximum shear force in the pile group was also determined and checked against the allowable shear capacity and was found to be satisfactory. The following safety factors were determined for the existing piles:

	<u>Safety Factor for Existing RAB</u>	<u>Safety Factor for Existing RAB and New Waste Evaporator Enclosure</u>	<u>Governing Load Condition</u>
Vertical Capacity	1.78	1.86*	Normal Operating Condition
Horizontal Shear Capacity	1.07	1.01	Hypothetical Earthquake Condition

* Location of the new waste evaporator enclosure reduced the eccentricity of the new configuration of the RAB.

No uplift in the pile group exists for any of the loading conditions. The safety factors determined for the pile group is based on an allowable pile capacity which uses a 25 percent reduction to the ultimate capacity of the piles, thus providing an additional safety factor for any locally weak piles.

The existing foundation mat of the RAB was investigated for the additional loads imposed by the waste evaporator enclosure. The existing mat was designed originally based on a very conservative vertical pile capacity of 185 kip/pile, considering all piles loaded uniformly. In accordance with the criteria, considering the reduced live load, the area lost due to walls and equipment and the reduced eccentricity, a new maximum vertical pile load of 182 kip/pile was determined. Since the new vertical pile load is less than the original design vertical pile load, the foundation mat is considered structurally adequate.

The existing interior and exterior walls of the RAB, which transmit the additional load from the waste evaporator enclosure to the foundation mat, were investigated. The original design of the most highly stressed existing exterior walls required bending tension reinforcement of 0.79 sq in./ft. With the additional compressive force imposed by the waste evaporator enclosure, the bending tension is reduced, resulting in a decrease in the required exterior wall reinforcement.

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The most highly stressed existing interior bearing walls, with the additional compressive force from the waste evaporator enclosure, is stressed to 850 psi. This is less than the allowable stress of 1150 psi permitted by the ACI 318-63 code. Therefore, the existing exterior and interior walls of the RAB are structurally adequate for the additional load imposed by the new waste evaporator enclosure.

3.8.4.1.2 Spent Fuel Pit

The spent fuel storage pit is designed for the underwater storage of spent fuel assemblies and control rods after their removal from the reactor. The spent fuel storage pit is constructed of reinforced concrete having 3 to 6 ft thick walls and is Class I seismic design. The entire interior basin face and transfer canal is lined with stainless steel plate.

3.8.4.1.2.1 Stress Analysis

The stress analysis for the high-density spent fuel racks was performed (as described in References 3.8.4-7 through 3.8.4-10) using the following load combinations specified in the "NRC Position for Review and Acceptance of Spent Fuel Storage and Handling Applications."

Elastic Analysis	Acceptance Limits
(1) $D + L$	Normal Limits of NF 3231.1a
(2) $D + L + E$	Normal Limits of NF 3231.1a
(3) $D + L + T_o$	Lesser of $2 S_y$ or S_u Stress Range
(4) $D + L + T_o + E$	Lesser of $2 S_y$ or S_u Stress Range
(5) $D + L + T_a + E$	Lesser of $2 S_y$ or S_u Stress Range
(6) $D + L + T_a + E'$	Faulted Condition Limits of NF 3231.1c

Definitions:

- D - Dead loads or their related internal moments and forces including any permanent equipment and hydrostatic loads.
- L - Live loads or their related internal moments and forces including any movable equipment loads.
- T_o - Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.
- T_a - Thermal effects and loads during highest temperature associated with the postulated abnormal design condition.
- E - Loads generated by the operating basis earthquake.
- E' - Loads generated by the safe shutdown earthquake.

The thermal loads due to rack expansion relative to the pool floor are negligible since the support pads are not structurally restrained in the lateral direction. The major seismic loads are produced by the operational basis earthquake (OBE) and safe shutdown earthquake (SSE) events.

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It was noted from the seismic analysis that the magnitude of stresses vary considerably from one geometrical location to the other in the model. Consequently, the maximum loaded cell assembly, grid assembly and the leveling pad assembly were analyzed. Such an analysis envelopes the other areas of the rack assembly.

The maximum seismic loads due to x direction and y direction shock were independently generated and combined by the square-root-sum-of-the-squares method to produce the resultant loads. The resultant loads were applied to the maximum loaded cell assembly, grid assembly and the leveling pad assembly to obtain the margins of safety.

The loads described in the seismic analysis (Section 3.7.3.5) were corrected by load correction factors derived from the nonlinear analysis. The computed stresses are below the allowable stresses as required by the ASME and B&PV Code, Section III, Subsection NF.

A fuel handling crane uplift analysis was performed which demonstrated that the rack can withstand the maximum uplift load of 3000 pounds of the fuel handling crane without violating the criticality acceptance criteria. Two accident loading conditions were postulated. The first condition assumed that the uplift load was applied to a fuel cell. The second condition assumed that the load was applied to the top grid.

A fuel assembly drop accident analysis was also performed to ensure that, in the unlikely event of dropping a fuel assembly, accidental deformation to the rack does not cause the criticality acceptance criteria to be violated, and the spent fuel pool liner will not be perforated.

In summary, the results of the seismic and structural analysis show that the H. B. Robinson spent fuel storage racks meet all the structural acceptance criteria adequately.

3.8.4.1.2.2 Fuel Bundle/Module Impact Evaluation

An analysis was performed to evaluate the effect of an impact load due to fuel assembly and fuel storage cell interaction during a seismic event. The fuel rack system consists of an array of cells which form the fuel rack structure and fuel assemblies. The fuel rack system is located in the spent fuel pool and is submerged in water.

Since the fuel assembly is stored within the cell, the gap between the fuel assembly grid and cell changes (i.e., opens and closes) during a seismic event. From the equation of motion for such a system, it is evident that the fuel rack system is nonlinear. This condition necessitates that a transient dynamic analysis can be performed.

The mathematical features of the nonlinear fuel rack model facilitate the determination of the fuel assembly/cell interaction and hydrodynamic mass (fluid mass) effects on the fuel rack response during seismic excitation. The effect of fuel assembly and fuel storage cell impact force on the rigid body displacements was obtained from the nonlinear analysis. The analysis was

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conducted with a minimum coefficient of friction of 0.2 and it was shown that the rigid body displacement was minimal (<0.3 inches). Thus, impact between adjacent rack modules or between a rack module and the pool wall is precluded.

The fuel assembly and fuel storage cell impact forces obtained from the nonlinear analysis were used to evaluate the effects on the fuel rack structure and fuel assembly structure. These loads are within the allowable limits of the fuel rack module materials and fuel assembly materials. Therefore, there is no damage to the fuel assembly or fuel rack module due to impact loads.

3.8.4.1.2.3 Effects of Increased Loads on the Fuel Pool Liner and Structures

The new spent fuel racks are free standing and are not connected to either the walls or floor of the pool as are the existing racks. Therefore, the effect of the new racks on the wall liner is less than that imposed by the existing racks. The sliding shear forces imparted to the floor liner under postulated earthquake conditions exceed those produced under the previous design; however, the sliding shear is well within the allowable working stresses of the liner material.

The final investigation showed that with the addition of one steel column under the fuel pool floor, as described in Section 9.1.2.1.5, the structure has adequate capacity to carry the increased loads imposed by the new high-density spent fuel storage racks.

The spent fuel pool structure has been evaluated for new loads based on the following criteria:

- a) Building Code Requirements for Reinforced Concrete. The ACI 318-63 Code.
- b) H. B. Robinson Unit No. 2 Final Safety Analysis Report.
- c) USNRC Operating Technical Position for Review and Acceptance of Spent Fuel Storage and Handling Applications.
- d) American Standards Association ASA A58.1-1955.
- e) American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Sixth Edition, 1963.
- f) Phillips Catalog F-1000, For Wedge Anchors.

Based on the above criteria the following is a listing of the primary loads considered in the structural evaluation.

- a) The dead weight of the structural elements including crane column dead loads and hydrostatic load from the pool water ($D_1 + D_2$).
- b) Live load including crane column live load (fuel cask) with impact and thrust (L_o).

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- c) Live load of existing fuel racks and fuel elements (L_1).
- d) Equipment load (Cask) (L_2).
- e) Equipment load (New Racks) (L_3).
- f) Wind loads.
- g) Cask drop equivalent static load on slab (FC).
- h) Seismic loads.
- i) A thermal loading (T_o) due to a pool water temperature of 150°F resulting in a $\Delta t = 80^\circ\text{F}$. The shrinkage of the concrete that has occurred since construction was calculated conservatively as an equivalent difference of 22°F thus reducing the effective $\Delta t = 80^\circ - 22^\circ = 58^\circ\text{F}$.

Load combinations are in accordance with ACI-318-63 Part IV B.

Numerous load combinations were used to evaluate the concrete structural elements.

For the final investigation, three dimensional finite element computer models were developed for the Spent Fuel Pool Structure. Plate elements were utilized to simulate the reinforced concrete wall and slab structure; beam elements were used to simulate the column. The computer model assumed the boundary to be fixed between fuel pool walls and supporting mat. The column was assumed fixed to the mat as well.

The analysis was performed using the MRI/STARDYNE Structural Analysis System (which is in the public domain). The static analysis package of MRI/STARDYNE - "STAR" was utilized to evaluate the internal forces and displacements induced in the spent fuel pool structure due to the primary loadings. Load combinations and force envelopes were developed using "STAR" output data exclusive of thermal loads. Internal forces due to thermally induced loads were added to maximum mechanical internal forces using elastic strain compatibility analysis methods. A thorough review of the envelope of maximum moments, axial forces, and shears in the critical structural elements indicated that the fuel pool structure is adequate to support the new rack loads under all possible loading combinations.

3.8.4.1.3 Class I Section of the Turbine Building

The Class I portion of the Turbine Building is north of the Class III portion and is a separate structure. All framework and supports for Class I equipment have been designed to Class I seismic design criteria. The sum of primary stresses resulting from operating conditions and the stresses resulting from the design earthquake was limited to 133 percent of allowable stresses, as permitted by the Uniform Building Code.

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All safeguards equipment in this Class I area is located on the ground floor (Elevation 226 ft). There is a Class I concrete ceiling over the top of this area to protect it from above. In addition, the Class I trench in this area is below grade with a checker plate which covers and protects the contents of the trench from falling debris in the event of an earthquake.

3.8.4.1.4 Intake Structure

The intake structure is designed as Seismic Class I, and is therefore not subject to collapse under earthquake loading.

The four service water pumps are located in three separate bays in the intake structure, the middle bay containing two pumps. The walls separating the bays and the deck above the piping are two and one half feet thick reinforced concrete.

3.8.4.1.5 Concrete Masonry Walls

In accordance with the requirements of Nuclear Regulatory Commission's IE Bulletin 80-11, the concrete masonry walls which are in the proximity of safety-related systems or equipment in the Reactor Building, Reactor Auxiliary Building, and the Fuel Handling Building have been analyzed and reinforced as necessary with structural steel supports to ensure that these walls will not collapse due to the hypothetical earthquake. Details are contained in References 3.8.4-2 through 3.8.4-6. The location of the concrete masonry walls is shown on Figure 3.8.4-2.

3.8.4.1.5.1 Design Criteria

The following criteria were used to analyze the concrete masonry walls and supports:

a) Material Specifications

- 1) Concrete block is Class B ($f'_c = 3,000$ psi)
- 2) Mortar is Class S, ($f'_c 1,800$ psi)
- 3) Structural steel and plate is ASTM A36
- 4) Concrete anchors are Phillips Wedge Anchors as manufactured by the Phillips Drilling Company, and
- 5) Welding electrodes conform to AWS A5.1 low hydrogen Class E70XX for Manual Shielded Metal-Arc Welding or AWS A5.17 F7X for Submerged Arc Welding.

b) Design Codes

- 1) American Concrete Institute (ACI) 67-23, Concrete Masonry Structures - Design and Construction

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- 2) ACI Standard, "Building Code Requirements for Concrete Masonry Structures", (ACI 531-79)
 - 3) American Institute of Steel Construction, (AISC) - Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings - Sixth Edition, Revised 1963
 - 4) American Welding Society (AWS) D1.1.76 - Structural Welding Code, and
 - 5) Phillips Catalog F-1000 dated May 1, 1973.
- c) Design Loads
- 1) Dead Load (D)
 - (a) Concrete Masonry = 143 pcf
 - (b) Structural Steel = 490 pcf
 - 2) Seismic Loads
 - (a) Hypothetical Earthquake (E) 0.2g base horizontal ground acceleration and 0.134g base vertical acceleration acting simultaneously
 - (b) Acceleration coefficients were obtained from the appropriate acceleration curves (Reference 3.8.4-3)

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- d) Load Combinations and Allowable Stresses
 - 1) For concrete masonry
(D) + (E) using allowable stresses as per ACI 67-23
 - 2) For structural steel
(D) + (E) using normal AISC working stresses

3.8.4.1.5.2 Analysis and Results

The block walls in question were first analyzed assuming normal ACI 531 allowable stresses in the mortar in horizontal joints as well as between wythes (collar joints) and between vertical edge of the block wall and the reinforced concrete wall or the floor to which it is joined. Based on these assumptions, walls were found to be satisfactory as they stood with no collar stress exceeding 2.5 psi as opposed to 8 psi generally allowable. Other mortar stresses were within the allowables of ACI 531.

In addition the walls were analyzed assuming only the allowable stress in the horizontal joints between blocks. The boundary conditions used in the analyses conservatively assumes that there is no bond or friction between wythes or between the top and side edges of a masonry wall panel and its adjoining reinforced concrete element. The masonry wall is vertically supported by a reinforced concrete floor. The floor connection is assumed to be a simple support with induced overturning load transmitted to the adjoining reinforced concrete elements by means of steel members and concrete anchors. Shear loads are transmitted to the floor by friction between the wall and the floor and/or by steel members attached with concrete anchors. Loads are transmitted across the vertical and overhead reinforced concrete element/block wall interface by means of steel members and concrete anchors. The steel members are either cantilevered off or framed between the adjacent reinforced concrete elements. Under these assumptions additional supports were provided to ensure the walls retained their function during and after the hypothetical earthquake. A review of the analyses, using reduced allowable stresses per ACI 531-79, Section 10.1.5, has been performed. For the purpose of the review, allowable masonry stresses, as shown in ACI 531-79, Table 10.1, were modified by the factors shown below:

Compression: $1 \times 2/3 \times 1.33 = 0.89$

Tension and Shear: $1 \times 1/2 \times 1.33 = 0.67$

All calculated masonry stresses were found to be less than or equal to these modified stresses after installation of the additional supports as shown in Reference 3.8.4-3.

No safety-related piping or equipment is known to be attached to the masonry walls included in these analyses. Therefore, no block pullout analysis was required. It should be noted, however, that wedge anchors were used to secure several reinforcing members to the block.

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Postulated missile impact loads are not applied to the Category I masonry walls considered, since all the block walls are considered protected from this type of loading.

Even though multi-wythe walls are provided, no additional strength due to that fact is considered. Consequently, all masonry wall analyses used for final design are based on the assumption that the walls are single wythe. (Note that the steel support framing is designed to resist loads induced by all wythes of multi-wythe walls.) However, calculations to account for the multiple wythes were performed.

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3.8.4.1.6 Concrete masonry in-fill penetrations. In accordance with the requirements of Nuclear Regulatory Commission's I. E. Bulletin 80-11, the concrete masonry in-fill penetrations that are in the proximity of safety-related systems or equipment in the Reactor auxiliary building, and the Fuel Handling building have been analyzed and repaired as necessary to ensure these walls will not collapse due to the hypothetical earthquake. Details are contained in Reference 3.8.4-11.

3.8.4.1.6.1 Design criteria. The following criteria were used to analyze the concrete masonry in-fill walls:

1. Material Specifications
 - a. Concrete brick grade is s-1 ($f_c' = 2000$ psi)
 - b. Mortar is type N, ($f_m' = 1275$ psi)
2. Design Codes
 - a. American Society of Testing Materials (ASTM) C-55, Concrete Building Brick
 - b. American Society of Testing Materials (ASTM) C-270, Mortar for Unit Masonry
 - c. ACI 530-88/ASCE 5-88, Building Code Requirements for Masonry Structures
 - d. ACI 530.1-88/ASCE 6-88, Specifications for Masonry Structures
3. Design Loads
 - a. Dead Load (D)
 - (1) Concrete Masonry
 - b. Seismic Loads
 - (1) Hypothetical Earthquake (E) 0.2g base horizontal ground acceleration and .134g base vertical acceleration acting simultaneously
 - (2) Acceleration coefficients were obtained from the appropriate acceleration curves (Reference 3.8.4-3)
4. Load Combinations and Allowable Stresses
 - a. For Concrete Masonry
 - (D) + (E) using allowable stresses per ACI 530-88/ASCE 5-88

3.8.4.1.6.2 Analysis and results.

The brick in-fill panels were analyzed assuming normal ACI 530-88/ASCE 5-88 allowable stresses in the mortar in horizontal joints and between the vertical edge of the brick in-fill panel and the concrete wall to which it is joined. Based on these assumptions, in-fill panels were acceptable with no stresses exceeding the allowables.

In most cases, the masonry in-fill panel is supported vertically by a concrete wall. These analysis conservatively assume no bond between the top block layer and the concrete wall and assumes simple supports on each side of the panel. Even though these panels are multi-wythe, no additional strength due to that fact is considered. Therefore, all in-fill panels were evaluated based on single wythe construction.

Several failure scenarios were evaluated for the in-fill panel which include:

1. Bond capacity between the brick and wall deterioration,
2. Bond failure between brick and mortar,
3. Pipe penetrations exerting loads on panels, and
4. Ductwork passing through penetrations.

It was determined that these cases were acceptable and in-fill panels were seismically stable.

3.8.5 FOUNDATIONS

3.8.5.1 Description of the Foundations

A field and laboratory investigation of subsurface conditions at the site was carried out by Dames and Moore. The borings completed at the site indicate that the soil conditions throughout the area are moderately consistent. Analysis based on this investigation indicated that a pile foundation would be the most satisfactory type for this site. A summary of the foundation investigation data is described in Section 2.5.4.

3.8.5.1.1 Containment Building

The containment is supported on pile foundations. The large depth of relatively low bearing strength soil occurring at the surface was a major factor in the selection of piles to support the containment. Piling safely carries the structural loads through the surface soils and transmits them to the dense soils underlying the area.

Load carrying capabilities of the piling beneath the containment was specified as follows:

- a) 225 ton downward capability
- b) 130 ton uplift capability, and
- c) 16 ton lateral capability.

These are ultimate capabilities. Factored capacities were reduced by appropriate capacity reduction factors as defined in Section 3.8.1.3.

Pile load tests were carried out to develop the type of pile used and a satisfactory method of installation.

These tests consisted of:

- a) The driving of 48 piles of various types and lengths
- b) Compression test loading 11 of these piles, and
- c) Lateral load testing a total of 16 piles with and without cap blocks, and
- d) Uplift load testing four piles.

As a result of these tests, the pile chosen to support the containment structure was a 12 in. diameter, 7/16 in. wall thickness steel pipe driven closed end to a penetration of 11 ft into the dense clay layer with a driving resistance of 7 blows per inch with a 48,750 ft-lb single acting steam hammer using a steel mandrel bearing on both the top and bottom of the pile. The pipe was filled with concrete after driving.

Pile load tests were performed by means of packing against a reaction beam held in place by means of reaction piles.

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Compression tests were performed on piles separated from the soil above the dense clay by an outer steel casing which was cleaned of all soil prior to testing the pile. This provided assurance that the pile embedment of 11 ft into the clay was sufficient to take all of the compressive load.

Uplift and lateral load tests were performed on piles in intimate contact with the upper soils.

The failure point for these load tests was defined as the point at which the load-deflection curve departed sensibly from a straight line. This was decided to allow an accurate determination of the effective modulus of subgrade reaction for the pile foundation (modulus equals the load required to produce a unit deflection).

The location of the area where these load tests were performed, and results of them, is shown in Figures 3.8.5-1 through 3.8.5-9. A summary of pile load data is given in Table 3.8.5-1 and 3.8.5-2.

During the installation of the piles for the containment, heave of previously driven piles caused by the driving of new piles was detected. A program was adopted to determine what the nature and cause of the pile heave was, and whether the load bearing capacity or modulus of subgrade reaction was affected by the heave in any way. The test program consisted of five compression tests on heaved piles, four uplift tests on heaved piles, one uplift test on a pile cased through the upper soils, redriving 10 percent of the heaved piles, and maintaining a close record in one area of the heave versus pile driving activity. As a result of the test program the following conclusions were drawn:

- a) Pile heave was the result of a general heave of the stiff clay layer into which the piles were driven.
- b) The piles were heaved along with the clay. No parting of the pile sides or tip from the clay occurred.
- c) The heaving caused no loss of load-bearing capability in the piles.

Results of the pile load tests obtained in this test program are shown in Figures 3.8.5-10 through 3.8.5-19.

Settlement of the pile foundation is expected to be 1/2 in. or less after completion of containment construction. Gross settlements are not expected to exceed 1 1/2 in. The containment has been designed to safely withstand at least 2 in. gross uniform or differential settlement. A 2 in. differential settlement across the entire building would result in an out-of-plumbness of about 5 min.

The piles were driven in a rectangular grid pattern beneath the containment at a spacing of 4 ft, resulting in a total of 923 piles. The plan layout and details of the piles are shown on Figure 3.8.1-3.

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The efficiency of a group of piles varies with the soil conditions, number of piles, spacing and dimensions of foundation, and is directly related to the skin friction and/or adhesion components considered in evaluating the bearing capacity of an individual pile along with the bearing capacity of the soil layer ultimately supporting the load. As these piles are driven into the dense soil strata underlying the area, which has more than sufficient bearing capacity, no reduction in pile load capacity due to group efficiency reduction was made.

The piles were designed to function by a combination of point bearing and friction in the hard silty-clay layer. No reliance is placed on the upper soils for vertical pile support, as these soils will tend to settle more than the deeper soils, under the long-term pile friction loading, thereby reducing the support from the upper soils. This is a conservative assumption for accident and earthquake conditions which are short-term conditions and would not result in settlement of the upper soils.

Several of the outer rows of piles around the containment will be subjected to uplift when the containment is pressurized for test or due to the design accident. In this case the piles will resist the uplift by skin friction in the upper soils. Pile load tests have confirmed the individual pile uplift capacity.

The total uplift does not exceed the weight of the soil engaged by the piles, thereby removing the possibility of an area failure. The upper soils can be used to resist uplift, as it is a short-term loading which will result in no plastic consolidation of the soil.

Negative skin friction at this site is not expected to occur. This is a phenomenon wherein the soil surrounding the piles receives a consolidating load, usually from a fill placed at the surface, or its own weight, and settles more than the piles compress or settle. The piles then receive load through friction from the adjacent soil as it settles. The soils beneath the site are slightly overconsolidated or normally consolidated, and no large falls are expected. As area consolidation or settlement of the site is not expected to take place, negative skin friction will not occur.

Negative skin friction due to pile heave will not materially add to the compressive load on the pile tip. Any negative skin friction which did occur will be relieved by slight consolidation in the restraining upper soils.

For seismic loadings the piles were considered as moving with the surrounding earth; hence, the full base shear on the base slab was transmitted to the piles.

This is somewhat conservative as there will be some transmittal of shear load from the building to the surrounding earth.

The relative horizontal movement assumed to occur between the clay strata and the containment slab during the maximum hypothetical earthquake is 4 in. The basis for this assumed movement is the anticipated maximum ground motion which is expected to occur during this earthquake. Wave amplitudes are theoretically doubled as they reflect from a free surface such as the ground surface at the Robinson plant. On this basis, the wave amplitudes at some distance below the surface would be one-half those on the surface. The ground movement expected at

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the Robinson site during the maximum hypothetical earthquake is estimated to be four inches. It was further assumed that the movement at the ground surface and at depth might be lagging due to the difference in the characteristics of surface materials and the hard clay layer and dense sands underlying the area. These assumptions then resulted in a total design movement for the piles of four inches. This movement was taken to occur between the hard clay layer approximately fifty feet below grade and the top of the piles approximately ten feet below grade.

The use of steel pipe piles ensure ductility of the pile system. In addition the piles are designed for a 4 in. differential deflection between the base slab and the pile tip due to differential (assumed) lateral displacements within the soil mass in which the pile is driven. As such, all piles are exposed to this displacement whether or not they support a structure.

It is not expected that the soil system is altered enough by the driving of piles into it to affect the relative stiffness between adjacent foundation systems. All foundations were assumed to be moving with the surrounding earth. Allowance has been made in the design for a differential settlement of 2 in. between the containment and other structures. All adjacent Class I structures are separated by a 2 in. isolation joint.

3.8.5.2 Loads and Loading Combinations

Referring to Section 3.8.1.3, the combined factored load equations, in which all the load components are axisymmetric, cause only axial load on the piles. Lateral load and bending moment is introduced by the factored load combinations in Section 3.8.1.3.2 b), c), and d), containing tornado, wind, or seismic loads among which the seismic loads are found to be the critical.

The pile loads corresponding to the load combinations containing seismic loads are assumed for the plant to include:

- a) The loads transferred to the piles as a result of the dynamic analysis of the containment structures
- b) The loads imposed on the piles as a result of a 4 in. differential deflection between the base slab and the pile tip, and
- c) Additional pile loads introduced when the axial load is superimposed on the deflected pile shape, assuming a) and b) are concurrent.

The pile loads in case a. consist of an axial load, lateral load, and fixed-end moment at the top of the pile. The lateral load is obtained by dividing the total seismic shear at the base of the containment by the total number of the piles provided. The lateral load at the top of piles gives rise to a distribution of bending moment which dies out rapidly from the top of the pile by virtue of the pile being in an elastic foundation. The fixed-end moment at the top is the maximum and equal to $P/2\lambda$.

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The analysis of the pile corresponding to case b) considers a pile being fixed at the top and hinged at the tip, and given a differential deflection of 4 in. between the supports as a result of the seismic ground motion. No distributed lateral load is assumed to act along the pile. Furthermore, the mass and stiffness of the pile are considered negligible in comparison with those of the surrounding soil so that the effects of the soil-pile interaction is believed to be insignificant. The lateral force and the fixed-end moment at the pile top in this case are given respectively by:

$$P = \frac{3EI\Delta}{\text{liter}^3}$$

$$M = \frac{3EI\Delta}{\text{liter}^2}$$

in which $\Delta = 4$ in. and $\ell = 50$ ft.

The effect of an axial load on bent piles has been shown to be predictable by the method of beams on an elastic foundation provided that the radii of the bent pile and the distribution of the axial force along the entire pile length are known (Reference 3.8.5-1). The intensity of the distributed lateral load is determined first by:

$$q(y) = \frac{N(y)}{R(y)}$$

where: $N(y)$ = the net axial force on a pile section at a distance y from the top of the pile

$R(y)$ = the radius of curvature of the pile at the point considered. To be consistent with the deflected shape assumed in case b) the variation of the radius of curvature is obtained from:

$$\frac{1}{R(y)} = \frac{M(y)}{EI} = \frac{3\Delta}{\text{liter}^2} \left(1 - \frac{y}{\text{liter}}\right)$$

The axial force is conservatively assumed constant, neglecting the skin friction between the pile and the soil entirely. This then reads:

$$q(y) = \frac{3N\Delta}{\text{liter}^2} \left(1 - \frac{y}{\text{liter}}\right)$$

It can be seen from this equation that the distribution assumes a triangular pattern which varies from a maximum of $3N\Delta/\square$ at the fixed top to zero at the pile tip (Reference 3.8.5-1).

With the distribution of the lateral load, q , thus defined, the shear and moment in the pile can be analyzed by the technique of beams on an elastic foundation. The expressions for the shear and moment of a beam on an elastic foundation, fixed at one end and the other hinged, and subjected to a triangular load, are readily available (Reference 3.8.5-2). In particular, the maximum shear and moment at the fixed support are given respectively by: (Reference 3.8.1-1)

$$P = \frac{3N\Delta}{2\lambda^2 \text{liter}^3} (2\lambda \text{liter} - 1)$$

$$M = \frac{3N\Delta}{2\lambda^3 \text{liter}^3} (\lambda \text{liter} - 1)$$

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The strength of the pile subject to the factored load combinations including seismic loads can now be investigated. The lateral load and moment to be used in the plotting are the algebraic sum of those presented in the above three cases.

The investigation indicates that the containment pile supports function satisfactorily under all the factored load combinations considered. The plots of the actual interacting pile loads are well confined within the interaction diagrams as shown in Figure 3.8.5-20 and Figure 3.8.5-21.

3.8.5.3 Design and Analysis Procedures

In the absence of combined vertical and lateral pile load tests, the pile capacities governed by the supporting strength of subsoil under combined loads are determined analytically. An interaction diagram, vertical versus lateral, is constructed to serve as a limiting boundary for the design of piles.

The load carrying capacity of piles is analyzed by considering the pile as a semi-infinite beam on an elastic foundation. The pile top is assumed to be prevented from rotation since it is embedded into the concrete mat. The coefficient of horizontal subsoil reaction, assumed constant, is derived from the results of test piles discussed in Section 3.8.5.1. The equation to determine the coefficient of subsoil reaction from Reference 3.8.5-1 is:

$$Y_o = \frac{2P\lambda^2}{K} - \frac{2M_o\lambda^2}{K}$$
$$Y_o = \frac{P\lambda}{K} \quad (1)$$

where: Y_o = lateral deflection at top of pile = 1/2 in.

$$\lambda = 4\sqrt{\frac{K}{4EI}}$$

K = coefficient of horizontal subsoil reaction

E = modulus of elasticity of pile group

I = moment of inertia of pile cross section

P = lateral load at top of pile - 40 kip

$$M_o = \frac{P}{2\lambda} = \text{fixed top moment}$$

The K -value can be determined by Equation (1) since Y_o , P , and EI are known, and is found to be equal to 140 kip/ft³.

The piles are designed to withstand various factored load combinations. All external loads could be reduced to an axial and a lateral load acting simultaneously at the fixed top of piles.

It is necessary, therefore, to examine the effect of interaction of axial and lateral loads on the capacity of the piles.

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The interaction diagram governed by the supporting strength of subsoil is obtained by considering a semi-infinite beam on elastic foundation subjected to an axial load and a lateral load at its fixed-top (Reference 3.8.5-1).

For axial load in compression,

$$Y_o = \frac{4P\beta\lambda^2}{K(3\beta^2 - \alpha^2)} - \frac{M_o}{EI(3\beta^2 - \alpha^2)}$$

$$Y_o = \frac{P}{3\beta^2 - \alpha^2} \left(\frac{4\beta\lambda^2}{K} - \frac{I}{2\beta EI} \right)$$

where:

$$\alpha = \lambda^2 + \frac{N}{4EI}$$

$$\beta = \lambda^2 + \frac{N}{4EI}$$

$$M_o = \frac{P}{2\beta} = \text{fixed top moment}$$

$$N = \text{axial load in compression}$$

For an axial load in tension, a similar expression can be written by interchanging α and β . It is to be noted that, when the axial load is absent, this is identical to Equation (1), above.

The relationship between axial and lateral load is calculated by limiting the lateral deflection at the top of piles to a specified amount (1/2 in.) established by tests discussed in Section 3.8.5.1.

It was found that the increase in axial compression is accompanied by only a slight decrease in lateral load, and the increase in axial tension resulted in a slight increase in lateral load, as expected. Thus, the N-P interaction diagram governed by the soil strength can be approximated closely by three constant boundaries representing compression, uplift, and lateral load capacities established by the pile tests with the appropriate reduction factor $\phi = 0.75$, namely, compression = 337.5^k, uplift = 195^k, and lateral load = 30^k, as shown in Figure 3.8.5-20.

Flexibility of the pile foundation in the vertical direction has been accounted for in the design by addition of a torsional spring in the model used for the dynamic analysis. The torsion spring simulates the rocking motion to account for shortening and elongation at the piles.

In the horizontal direction the piles and base slab were assumed as moving with the earth.

As there is no change in the spectra, no computations are necessary.

The dynamic effect of earthquake loadings on pile supports is evaluated in terms of equivalent static loads obtained from the containment dynamic earthquake analysis described in Section 3.7.

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The analysis of piles with respect to the lateral earthquake load considers piles as being fixed at the top and supported on an elastic foundation. As a result, the maximum values of lateral soil pressure, pile bending moments, and shearing forces occur at the pile top and damp out rapidly towards the pile tip. The lower two-thirds of the pile is therefore practically unstressed under a lateral load.

The required strength of the pipe is provided by the combined action of steel pipe and concrete fill. The contribution of the concrete fill to the compressive strength of the pile section is calculated on the usual assumption that a plane section remains plane after deformation and no slip occurs between steel-concrete interface. It is to be noted, however, that the validity of the assumptions does not necessarily depend upon the integrity of bond between steel pipe and concrete in the vicinity of the pile top. In case of a local bond failure near the top, the physical confinements (namely, the rigid mat above and the effective bond area over a considerable length of the pile below) should enable the pile sections to deform as assumed.

The interaction diagram, axial forces vs. bending moments, governed by the structural strength of the pipe (Figure 3.8.5-21) is based on the derivation contained in the ACI Publication SP-7, Ultimate Strength Design of Reinforced Concrete Columns. In the derivation, the circularly arranged reinforcement is approximated by a thin circular ring of an equivalent area. The application of the derived equations to the pipe under consideration is therefore exact.

The derivation is made in two parts by considering the steel ring and the circular concrete section separately. In each case, the expressions for the net force and moment are derived on satisfaction of the conditions of equilibrium and compatibility of strains in compliance with the requirements stipulated in Part IV-B-Ultimate Strength Design, ACI 318-63. The interaction diagram is constructed by summing up the force and moment of the two parts corresponding to a position of the neutral axis assumed.

For the stepwise derivation and the definition of terms of the equations, reference is made to the ACI Publication SP-7. Only the final expressions for the force and moment are given herein.

$$F_s + F_c = \frac{\phi f_y \rho_t t^2}{4} \left\{ \theta_3 + \theta_4 - \pi + \frac{g}{2 K_v \psi} [\sin \theta_4 - \sin \theta_3 - (\theta_4 - \theta_3) \cos \theta_1] \right\} \\ + 0.2125 \phi f_c' t^2 [\theta_2 - \sin \theta_2 \cos \theta_2]$$

$$M_s + M_c = \frac{\phi f_y \rho_t g t^3}{8} \left\{ \sin \theta_3 + \sin \theta_4 + \frac{g}{2 K_v \psi} [1/2(\sin \theta_4 \cos \theta_4 - \sin \theta_3 \cos \theta_3) \right. \\ \left. + 1/2(\theta_4 - \theta_3) - \cos \theta_1 (\sin \theta_4 - \sin \theta_3)] \right\} + 0.070833 \phi f_c' t^3 \sin^3 \theta_2$$

It is noted that, unlike the conventional reinforced concrete columns, the concrete is completely encased inside the steel pipe; therefore, the g-value should be equal to unity. Other numerical constants used in the calculations are:

$$f_y = 35,000 \text{ psi}$$

$$f_c' = 3,000 \text{ psi}$$

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$$T = 12.4375 \text{ in.}$$

$$\psi = \frac{\epsilon_y}{\epsilon_u} = 0.4023$$

$$\rho_t = 0.1408$$

$$\phi = 0.75$$

3.8.5.3.1 Lateral Load Tests

The pile load tests were run to determine the relative deflection-load characteristics between the top of the pile/foundation slab and the top of the soil. The design value for lateral deflection was selected as the point at which the load deflection curve departed sensibly from a straight line (Section 3.8.5.1.1); in this case, about 0.35 in. In no factored load case will the pile top deflect more than this value (relative to the top of the soil) and the piles have been considered as acting linearly under all factored load conditions. The restraint at the top of the piles for lateral load tests was 12 in. embedment into a 3 ft thick x 8 ft x 4 ft concrete pile cap in which two piles were embedded such as to give identical restraint as for the service piles used for construction (Figure 3.8.5-8). Therefore, no correction for imperfect restraint was necessary.

The piles were assumed as moving with the surrounding earth, therefore no account was made of the mass of earth moving with the piles other than the relative 4 in. deflection from tip of pile to top of pile due to different movements of this mass of earth.

For laterally loaded piles the displacement, which is greater at the top of the pile, decreases in a continuous manner with depth so that the greatest strain in the soil in the vicinity of the pile is at the ground surface. The problem of evaluating the soil reaction in a region near the surface, as well as the entire length, is a highly complex one; therefore, the soil pile interaction is analyzed employing the horizontal subgrade reaction or beam on elastic foundation concept (Reference 3.8.1-1).

The elastic analysis utilized to establish the pile-soil reaction has indicated that the reaction is concentrated in the upper 30 percent of the pile. These findings have been verified by many investigators in field tests and, in fact, were instrumental in establishing necessity of the elastic approach to develop the realistic soil pressure.

Since the load is realistically considered and concentrated near the top of the pile, the cyclic effects of the earthquake-induced loading should not alter the soil pressure distribution; in fact, the soil-pile system may not be able to react to a rapidly cycling load and may, therefore, not displace as much as the tested static case.

Laboratory tests on the soils have shown that there is a minor increase in strain in the soil under rapid cyclic loading, inferring that no increase in horizontal displacements due to soil failure should be anticipated.

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The lateral pile load tests (Figure 3.8.5-9) which were cycled show little hysteresis at the design lateral load for slowly cycled loads. This coupled with the rapid cyclic tests in the laboratory provide assurance that a gradual increase in horizontal displacements will not occur.

3.8.5.3.2 Evaluation of Earthquake Effects

The dynamic stresses and strains generated in the foundation soil mass during the maximum hypothetical earthquake will not modify the cohesion or angle of internal friction. The uplift and compressive capability of the piles, which are dependent basically on the dense clay as shown by pile load tests, is therefore not reduced by the earthquake loadings. Uplift capability of the piles supported only by the dense clay was shown in test pile TP 2 uplift test (Table 3.8.5-2).

The lateral stability of the piles relies upon the upper sands and silts. The possibility of pore water pressure build up was considered in this strata during the motion associated with the maximum hypothetical earthquake. Considering published laboratory data on soils having similar grain size characteristics and density, and subjected to cyclic loadings commensurate with the earthquake, a pore pressure increase of 30 percent could occur. If such an increase (30 percent) were to occur, a decrease in effective soil strength of 30 percent would result, and an increase in lateral pile deflection of 50 percent would be required to develop the required loadings.

This additional deflection would lower the effective lateral "k" value, however, the pile structural calculations reflect this "k" value as a fourth root, thereby increasing structural loads a small amount however they remain within the defined limits of the interaction diagram shown in Reference 3.8.1-1.

3.8.5.3.3 Critical Stresses in Anchor Straps and Bearing Surfaces

Two types of piles, Type 5 and Type 6, are used under Class I structures. Type 5 piles are compression piles for which no anchor straps are required. Type 6 piles are those subject to tension with anchor details previously presented. The pile anchorages are designed to withstand the factored load combinations specified in the Section 3.8.1.3. For the evaluation of the axial loads, lateral loads, and fixed top moments of piles, see Figure 3.8.5-21. The critical stresses in the anchor straps and the concrete-piles bearing interface are analyzed in the following manner (see Figure 3.8.5-22):

- a) Given 6 - 1 1/4 in. x 1 1/4 in. plain bar per pile
- b) Total cross sectional area = 9.56 sq in.
- c) Minimum yield stress = 40,000 psi (ASTM A306 Gr80);
- d) Total embedment surface area (neglecting the bends) = 2,010 sq in.
- e) Ultimate bond stress = 20 psi (ACI 318-63);
- f) Maximum tensile stress = $\frac{140,000}{9.56} = 14,700$ psi;

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g) Maximum bond stress = $\frac{140,000}{2,010} = 70$ psi

The following assumptions are made in the analysis:

- a) The embedded portion of the piles is rigid.
- b) Plane sections remain plane.
- c) The deviation of the neutral axis from the mid-point of the embedment is only slight; the ultimate bearing stress distribution is approximated by two rectangular stress blocks with parameters identical to those established for concrete compression at ultimate strength.

The bearing stress intensity f_b and the location of the neutral axis can be determined from the following equilibrium conditions:

$$\Sigma F = 0, \quad C_1 - C_2 - P = 0 \quad (1)$$

$$\Sigma M = 0, \quad C_1 (0.425 \cdot 1_1) - C_2 [1 - 0.425 (1 - 1_1)] + M = 0 \quad (2)$$

where: $C_1 = 0.85 f_b 1_1 d$

$$C_2 = 0.85 f_b (1 - 1_1) d$$

Equations (1) and (2) are solved simultaneously to yield $1_1 = 6.3$ in. and $f_b = 3,360$ psi which is only slightly in excess of $\phi (0.85 f'_c) = 0.9 (0.85) 4,000 = 3,060$ psi. The slight overstress is considered insignificant since the ultimate bearing stress of concrete with lateral confinement is considerably higher than the ultimate compressive stress of unconfined concrete cylinders.

3.8.5.3.4 Selected Equivalent Foundation Modulus

The results from the test piles presented in the Table 3.8.5-1 were used to select the equivalent foundation modulus of 580 lb/in^3 (1000 kip/ft^3). The piles of interest were TP 2, TP 4, TP 5, and TP 6. TP 2 and TP 4 were driven into the dense clay with a steel casing preventing any load transmittal to the upper soils (Section 3.8.5.1.1). TP 5 and TP 6 were in intimate contact with the upper soils.

As would be expected TP 2 and TP 4 responded as a softer spring than TP 5 and TP 6 reflecting the long length of column with no soil removing load. They had a "k" value of about 12000 kip/ft . TP 5 and TP 6 had a "k" value of 16000 kip/ft . Since the service piles were not to be cased, the "k" value of 16000 kip/ft was chosen as the design value. For piles spaced on a $4 \text{ ft} \times 4 \text{ ft}$ grid this is equivalent to a uniform subgrade modulus of 1000 kip/ft^3 .

The pile supports are treated as a series of elastic springs in the base slab (mat) analysis. The pile axial loads are obtained by multiplying the spring constant (1600 kip/ft) to the mat deflections.

The results of the tests performed to check the effect of pile heaves are presented in Table 3.8.5-2.

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Disregarding TP 2, which was a cased pile, these results verified quite well the selected value for "k". The values ranged from 13,500 kip/ft to 28,500 kip/ft with an average value of 18,122 kip/ft (14,956 kip/ft if the highest single value, 28,500 is eliminated as erroneous).

3.8.5.4 Materials, Quality Control, and Corrosion Protection

3.8.5.4.1 Materials

Pipe piles were specified to be new pipe meeting the requirements of ASTM A252 Grade 2 "Standard Specifications of Welded and Seamless Steel Pipe Piles". Bottom closure plates were welded on before driving the piles.

3.8.5.4.2 Quality Control

See Section 3.8.1.6.2.

3.8.5.4.3 Pile Corrosion Protection

Any steel structure in soil (even without the protection afforded by concrete) is progressively less susceptible to corrosion as the electrical resistivity of the soil increases. Soil resistivity measurements taken in August, 1958, prior to construction of the since retired fossil plant and as reconfirmed by measurements taken at the construction site in December, 1966, have established that the soil resistivity is so high that the possibility of active corrosion is minimal.

The loss of steel from piles underneath the containment structure is expected to be very small in the absence of cathodic protection because of the high-resistivity soil environment.

The flow of direct current set up by galvanic corrosion potentials is the critical element in corrosion conditions since the amount of metal removed is in proportion to the amount of current discharged from a structure. Anything that will increase the circuit resistance in a corrosion cell will reduce the current and reduce corrosion. High soil resistivity accomplishes this.

In general, soils having resistivities above 10,000 ohm-cm are considered noncorrosive. As the resistivities increase above this value, the risk of corrosion becomes progressively less.

Prior to construction of the since retired fossil plant at the H.B. Robinson plant site, a corrosion survey in August, 1958, revealed that the average soil resistivities to 25 ft depth were in the order of 600,000 ohm-cm. A new survey was made in December, 1966, to verify that the soil resistivities in the area are high. Comparative results are as follows:

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Average Depth in Ft	Soil Resistivities in Ohm-Cm Measured in 1958 -- 33 Locations			Soil Resistivities in Ohm-Cm Measured in 1966 -- 27 Locations		
	Average	Maximum	Minimum	Average	Maximum	Minimum
2.5	648,000	2,500,000	23,000	414,000	2,390,000	35,000
5.0	630,000	2,300,000	35,000	450,000	2,199,000	59,000
7.5	658,000	1,650,000	51,000	409,000	1,721,000	83,000
10.0	690,000	1,800,000	64,000	377,000	2,103,000	61,000
15.0	652,000	1,650,000	75,000	347,000	1,577,000	26,000
25.0	682,000	1,880,000	95,000	277,000	1,864,000	8,000

It will be noted that the 1966 average values are less than those measured in 1958. This is attributed to a higher ground water table in 1966 than in 1958 caused by the incursion of water from the cooling water lake which had not been built at the time the 1958 measurements were taken, plus the fact that during the 1966 tests the ground was saturated by test well pumping operations at some of the test locations. The lake and ground water (both in excess of 20,000 ohm-cm resistivity), in combination with the high resistivity predominantly sand environment, operate to reduce average resistivity values.

Even under the wet conditions obtained at the site, average values of resistivity are high and the lowest made at the many test locations (26,000 ohm-cm) down to 15 ft depth (covering soils in which structures subject to corrosion would be buried) is well above the 10,000 ohm-cm value below which cathodic protection is considered. The one lower reading of 8,000 ohm-cm at the 25 ft depth was taken at a very wet location approximately 700 ft from the containment structure and would have no effect on it.

Corrosion of steel pipe piles below the containment structure is expected to be minimal in view of the high soil resistivity. The only point where any additional protection will be required is at the junction of bare pile steel and the concrete foundation. At this point, two coats of cold-applied Koppers' Bitumastic 50 or equal has been applied so that the coating extends into the reinforced concrete and to a depth of at least 12 in. below the concrete.

Average soil resistivities at the construction site taken to a depth of 65 ft (which embraces the depth to which piles will be driven) were found to be as follows during the December, 1966, soil survey:

143,500 ohm-cm average, 525,800 maximum, 44,200 minimum

On this basis, conservative computations indicate that the 40-year corrosion loss on piles beneath Unit 2 will be approximately 0.0022 percent or a total weight loss of approximately 103 pounds based on 1500 12 in. diameter steel pipe piles. This considered negligible corrosion.

In addition, studies on pile corrosion in soil made by the U.S. Bureau of Standards have established that the corrosion of piles that had been driven into undisturbed soil is negligible even after many years of service including locations where the soil is far more aggressive than at the H.B. Robinson plant site. Piles were pulled having service periods up to 40 years.

Ebasco corrosion engineers inspected some of these piles at the Bureau of Standards and verified the absence of corrosion. The results of the pile test work are published in Reference 3.8.5-3.

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TABLE 3.8.5-1

SUMMARY OF PILE LOAD TEST DATA

<u>AREA NO.</u>	<u>PILE NO.</u>	<u>PILE TYPE</u>	<u>DEPTH OF PENETRA- TION</u>	<u>TYPE OF LOAD TEST</u>	<u>MAXIMUM LOAD APPLIED</u>	<u>MAXIMUM PILE HEAD MOVEMENT</u>	<u>ULTIMATE PILE CAPACITY</u>	<u>PILE HEAD MOVEMENT @ U.P.C.</u>	<u>MODULUS OF PILE RE- ACTION IN KIP/FT</u>	<u>REMARKS</u>
I	PP 1	12"φ Pipe	61.7'	Lateral	24 ^T	2.04"	-	-	N/A	Used as a Guide
	PP 2	14 BP 102	70.0'	Lateral	24 ^T	2.17"	-	-	N/A	Used as a Guide
	RS 1	Raystep	68.0'	Lateral	26 ^T	1.72"	10 ^T	0.43"	N/A	Jetted 50'
	RS 3	Raystep	68.0'	Lateral	26 ^T	2.01"	10 ^T	0.57"	N/A	Jetted 58'
	TP 13	12"φ Pipe	63.6'	Compression	250 ^T	1.31"	130 ^T	0.20"	15 600	
	TP 14	14 BP 102	69.0'	Compression	370 ^T	2.00"	250 ^T	0.53"	11 300	
	TP 15	14 BP 102	64.0'	Compression	225 ^T	0.62"	190 ^T	0.38"	12 000	
	RS 2	Raystep	69.0'	Compression	400 ^T	1.46"	300 ^T	0.94"	7 670	Jetted 54'
	TP 1	Composite	69.0'	Compression	350 ^T	0.68"	225 ^T	0.38"	14 200	
	TP 2	12"φ Pipe	69.0'	Compression	300 ^T	1.28"	225 ^T	0.47"	11 500	
II	TP 3	Composite	77.3'	Compression	405 ^T	0.89"	300 ^T	0.59"	12 200	
	TP 4	12"φ Pipe	68.5'	Compression	300 ^T	0.88"	225 ^T	0.42"	12 900	
	TP 5	12"φ Pipe	69.0'	Uplift	120 ^T	0.18"	120 ^T	0.18"	16 000	
	TP 6	12"φ Pipe	69.0'	Uplift	150 ^T	0.33"	130 ^T	0.26"	16 000**Correction for Test 6 Days Later	

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TABLE 3.8.5-1 (Cont'd)

SUMMARY OF PILE LOAD TEST DATA										
AREA NO.	PILE NO.	PILE TYPE	DEPTH OF PENETRATION	TYPE OF LOAD TEST	MAXIMUM LOAD APPLIED	MAXIMUM PILE HEAD MOVEMENT	ULTIMATE PILE CAPACITY	PILE HEAD MOVEMENT @ U.P.C.	MODULUS OF	REMARKS
									PILE RE-ACTION IN KIP/FT	
TP 7		Composite	67.0'	Uplift	120 ^T	0.24"	120 ^T	0.24"	12 000	
TP 8		Composite	67.0'	Uplift	190 ^T	0.44"	175 ^T	0.39"	16 000*	*Correction for Test 8 Days Later
TP 5&6		12"φ Pipes	69.0'	Lateral	60 ^T	0.91"	16 ^T /Pile	0.39"	N/A	Fixed End
TP 5&6		12"φ Pipes	69.0'	Lateral	60 ^T	1.13"	16 ^T /Pile	0.46"	N/A	Fixed End
TP 7&8		Composites	67.0'	Lateral	60 ^T	0.88"	16 ^T /Pile	0.34"	N/A	Fixed End
AP 1		12"φ Pipe	69.0'	Lateral	38 ^T	2.02"	16 ^T	0.54"	N/A	Free End
AP 2		12"φ Pipe	69.0'	Lateral	38 ^T	1.90"	16 ^T	0.58"	N/A	Free End
AP 3&4		12"φ Pipes	60.0'	Lateral	60 ^T	0.80"	16 ^T /Pile	0.31"	N/A	Fixed End
IIITP 11A		'0' Step Taper	58.0'	Compression	200 ^T	1.16"	125 ^T	0.34"	8 840	
TP 12		'0' Step Taper	63.0'	Compression	225 ^T	0.57"	175 ^T	0.36"	11 700	
TP 16		'000' Step Taper	59.0'	Compression	185 ^T	2.00"	75 ^T	0.15"	12 000	
TP 11A		'0' Step Taper	58.0'	Lateral	5 ^T	2.00"	3 ^T	0.60"	N/A	Free End Jetted 25'

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TABLE 3.8.5-1 (Cont'd)

SUMMARY OF PILE LOAD TEST DATA										
AREA NO.	PILE NO.	PILE TYPE	DEPTH OF PENETRATION	TYPE OF LOAD TEST	MAXIMUM LOAD APPLIED	MAXIMUM PILE HEAD MOVEMENT	ULTIMATE PILE CAPACITY	PILE HEAD MOVEMENT @ U.P.C.	MODULUS OF PILE RE-ACTION IN KIP/FT	REMARKS
		'0'								
TP 12		Step Taper	63.0'	Lateral	5 ^T	1.00"	3 ^T	0.38"	N/A	Free End
		'000'								
AP 10A		Step Taper	64.0'	Lateral	16 ^T	2.00"	7 ^T	0.52"	N/A	Free End
		'000'								
AP 12		Step Taper	67.0'	Lateral	16 ^T	1.36"	7 ^T	0.44"	N/A	Free End

NOTES: Depth of Penetration measured from Grade - El. 226

N/A: Not Applicable

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TABLE 3.8.5-2

SUMMARY OF PILE LOAD TEST DATA

<u>CONTAINMENT QUADRANT</u>	<u>PILE NO.</u>	<u>HEAVE</u>	<u>DEPTH OF PENETRATION</u>	<u>PENETRATION INTO CLAY</u>	<u>MAXIMUM LOAD</u>	<u>GROSS UPLIFT OR SETTLEMENT AFTER UNLOADING</u>		<u>"K" (Kip/Ft)</u>
						<u>OR</u>	<u>NET UPLIFT OR SETTLEMENT</u>	
SW	TP 2	-	69'	13'	142 ^T U	.338"	.073"	10,400
SW	592	.29"	58'	11'	240 ^T C	.358"	.116"	16,900
SW	816	.27"	58'	11'	240 ^T U	.352"	.087"	20,800
SW	637	.36"	57'	11'	330 ^T C	.662"	.340"	15,400
NW	93	.21"	56'	11'	130 ^T U	.163"	.054"	20,800
NW	271	.32"	59'	11-1/2'	230 ^T C	.415"	.201"	13,500
NE	365	.13"	61'	15'	132 ^T U	.206"	.065"	15,400
NE	167	.13"	69'	18'	225 ^T C	.402"	.148"	13,500
SE	685	.23"	56'	11'	150 ^T U	.130"	.019"	28,500
SE	850	.37"	57'	11-1/2'	228 ^T C	.295"	.118"	18,300

Note: K computed at required
pile capacity
225 tons compression
130 tons uplift

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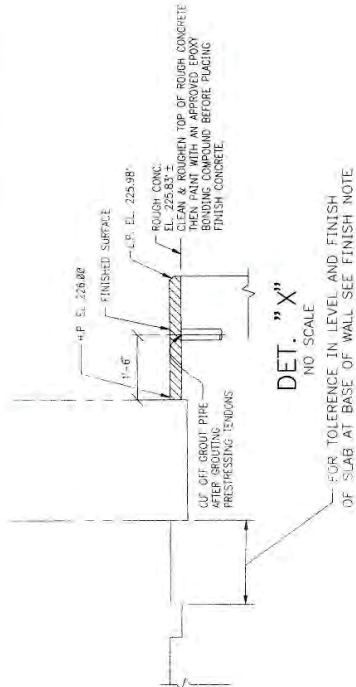
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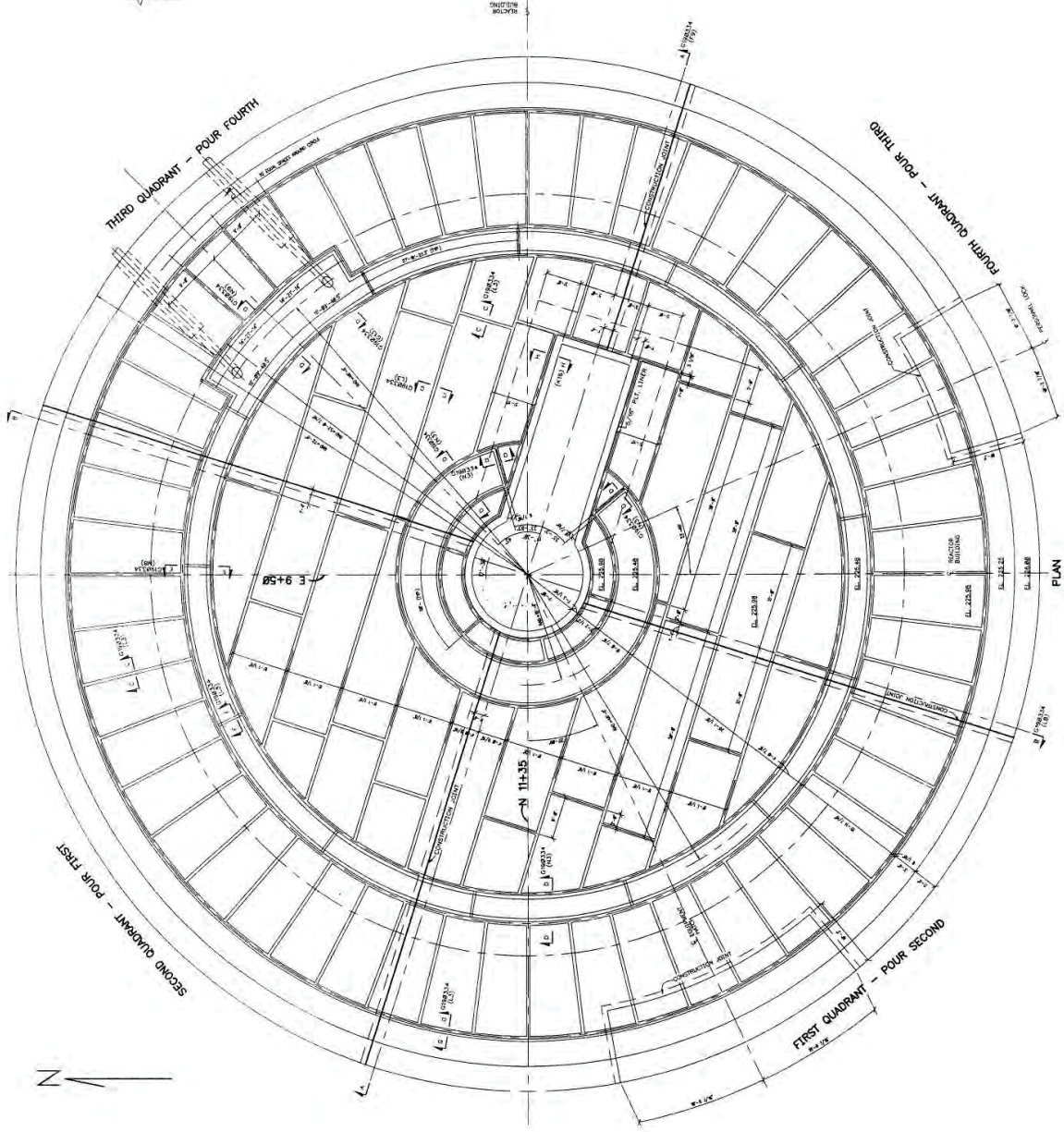


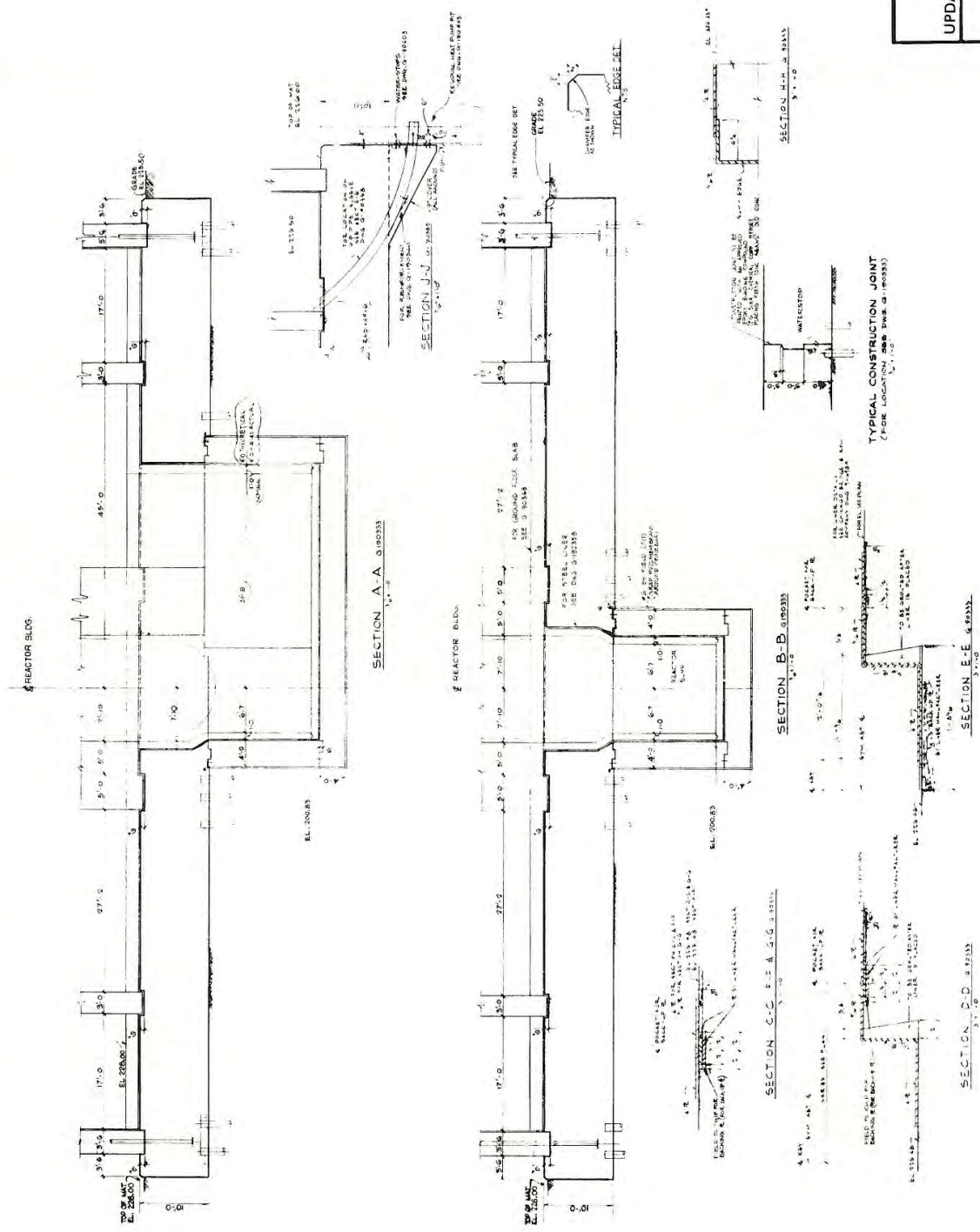
QUANTITIES

5.638 CU. YDS.
CONCRETE
BASE SLAB

NOTES

CURRENT A.C.I. STANDARDS SHALL GOVERN FOR ALL DESIGN AND CONSTRUCTION UNLESS OTHERWISE NOTED.
CONCRETE SHALL BE CLASS AA (4800 PSI)
SEE SPECIFICATION EBSACO 14-85-CONCRETE-WASCONIT.
ALL ANCHOR BOLTS, DRAIN PIPES, PIPE SLEEVES, ETC. SHALL BE IN POSITION BEFORE CONCRETE IS PLACED.
CONCRETE TEMPERATURE AT TIME OF PLACING NOT TO EXCEED 75°F IF DRY BULB TEMPERATURE IS 85°F OR HIGHER.
EXCEPT IN AREA AT BASE OF CONTAINMENT WALL, FINISH SHALL BE 1/2" INCH IN 18" FT. NO FINISH SHALL BE EXCEED 1/8" INCH IN 18" FT. NO TO EXIST DUE TO AGGREGATE, FLOAT MARKS, ETC.
AREA AT BASE OF WALL BENEATH MANHOLE OF REACTOR SHALL BE RUBBER FLOAT OR STEEL TROWEL FINISH. LEVEL SHALL BE 1/8" INCH IN 36" FT. 1/4" INCH IN ENTIRE WALL PERIMETER.



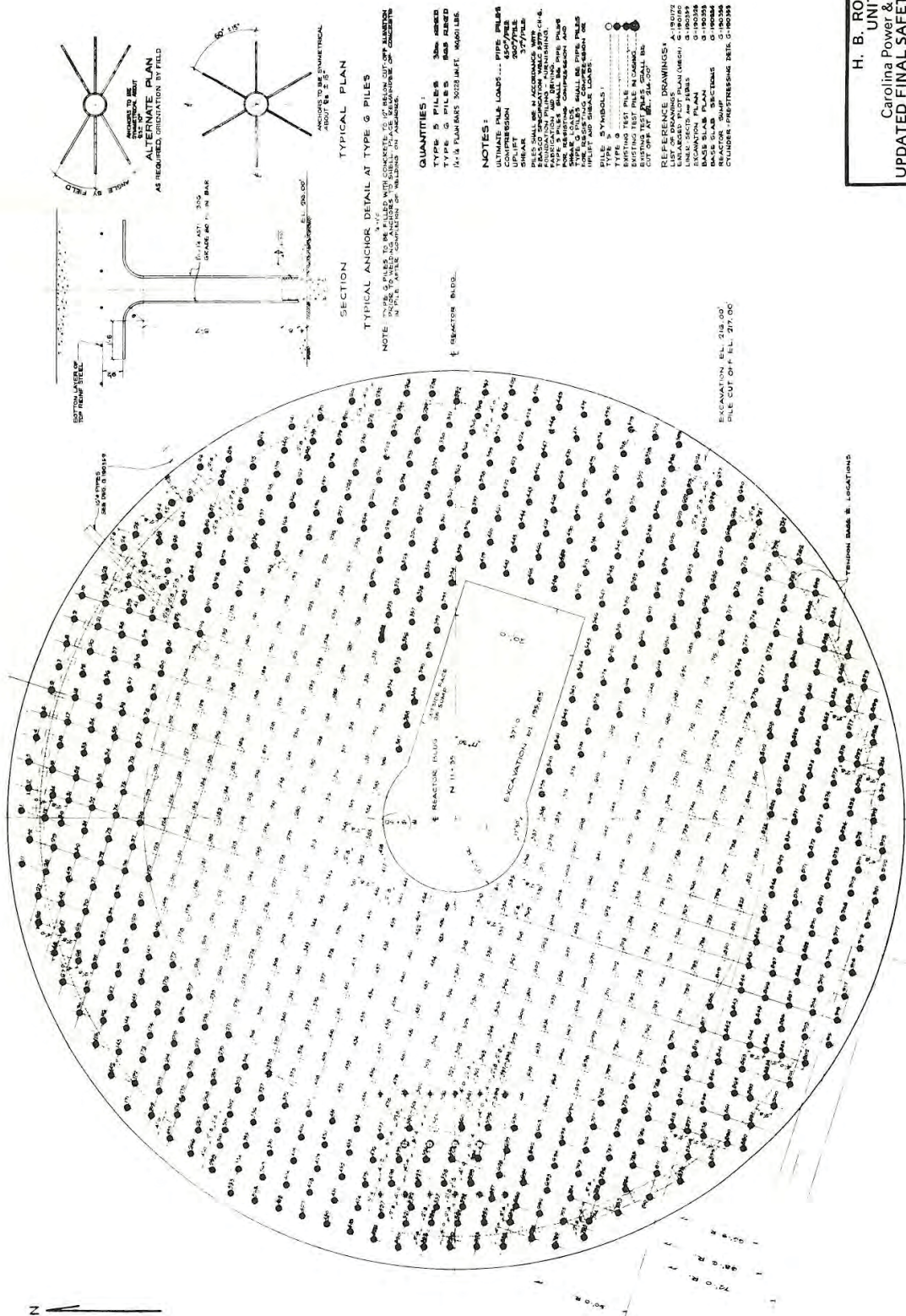


QUANTITIES:
FOR QUANTITIES SEE DWG. G-100333

NOTES:
1. SEE DWG. G-100333 FOR ADDITIONAL NOTES.
2. SEE DWG. G-100333 FOR ADDITIONAL NOTES.

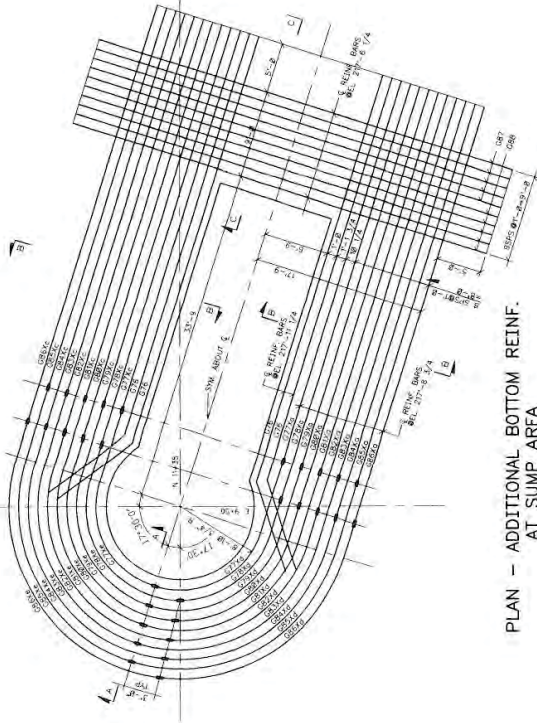
REFERENCE DRAWINGS:
1. SEE DWG. G-100333 FOR ADDITIONAL NOTES.
2. SEE DWG. G-100333 FOR ADDITIONAL NOTES.

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UPDATED FINAL SAFETY ANALYSIS REPORT
REACTOR BUILDING BASE SLAB
SECTIONS - MAS.
FIGURE 3.8.1 - 2



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REACTOR BUILDING PILING PLAN



QUANTITIES:

REINF. BASE-SUMP REINFORCING SCHEDULE
(ADWELDS (#145))

NOTES:

CURRENT AIA STANDARDS SHALL GOVERN FOR ALL DESIGN AND CONSTRUCTION UNLESS OTHERWISE NOTED.
REINFORCING PLANS AND SPECIFICATIONS SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF THE ACI 308R-90 CONCRETE REINFORCING PLANS AND SPECIFICATIONS. EDITION 308R-90 SHALL BE USED FOR ALL REINFORCING PLANS AND SPECIFICATIONS.
PLACING DIMENSIONS ARE GIVEN TO CENTER OF BARS UNLESS NOTED.
ALL BARS SHALL HAVE 2" MINIMUM CONCRETE COVER UNLESS OTHERWISE NOTED.
SHIFT OR BEND BARS TO CLEAR ANCHOR BOLTS, DRAINS, ETC. AS NECESSARY. MINIMUM CLEARANCE SHALL BE 1" (MIN ORID) ± 3 INCHES.

REFERENCE DRAWINGS:

LIST OF DRAWINGS
PLOT PLAN
REACTOR BUILDING REINFORCING SCHEDULE (SH #1)
REACTOR BUILDING REINFORCING SCHEDULE (SH #2)
REACTOR BUILDING REINFORCING SCHEDULE (SH #3)
REACTOR BUILDING REINFORCING SCHEDULE (SH #4)
REACTOR BUILDING REINFORCING SCHEDULE (SH #5)
REACTOR BUILDING REINFORCING SCHEDULE (SH #6)
REACTOR BUILDING REINFORCING SCHEDULE (SH #7)
REACTOR BUILDING REINFORCING SCHEDULE (SH #8)
REACTOR BUILDING REINFORCING SCHEDULE (SH #9)
REACTOR BUILDING REINFORCING SCHEDULE (SH #10)

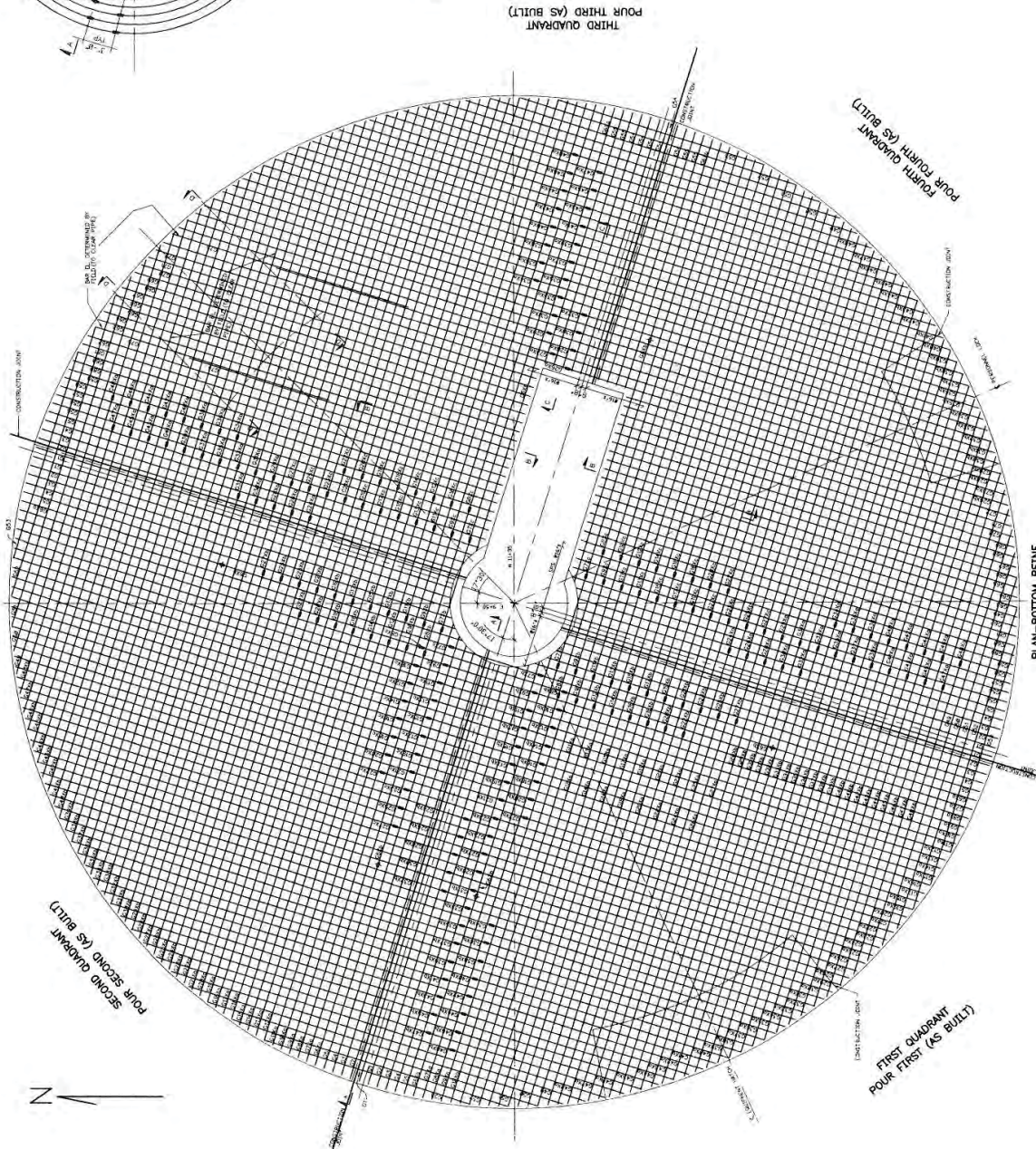
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UNIT 2

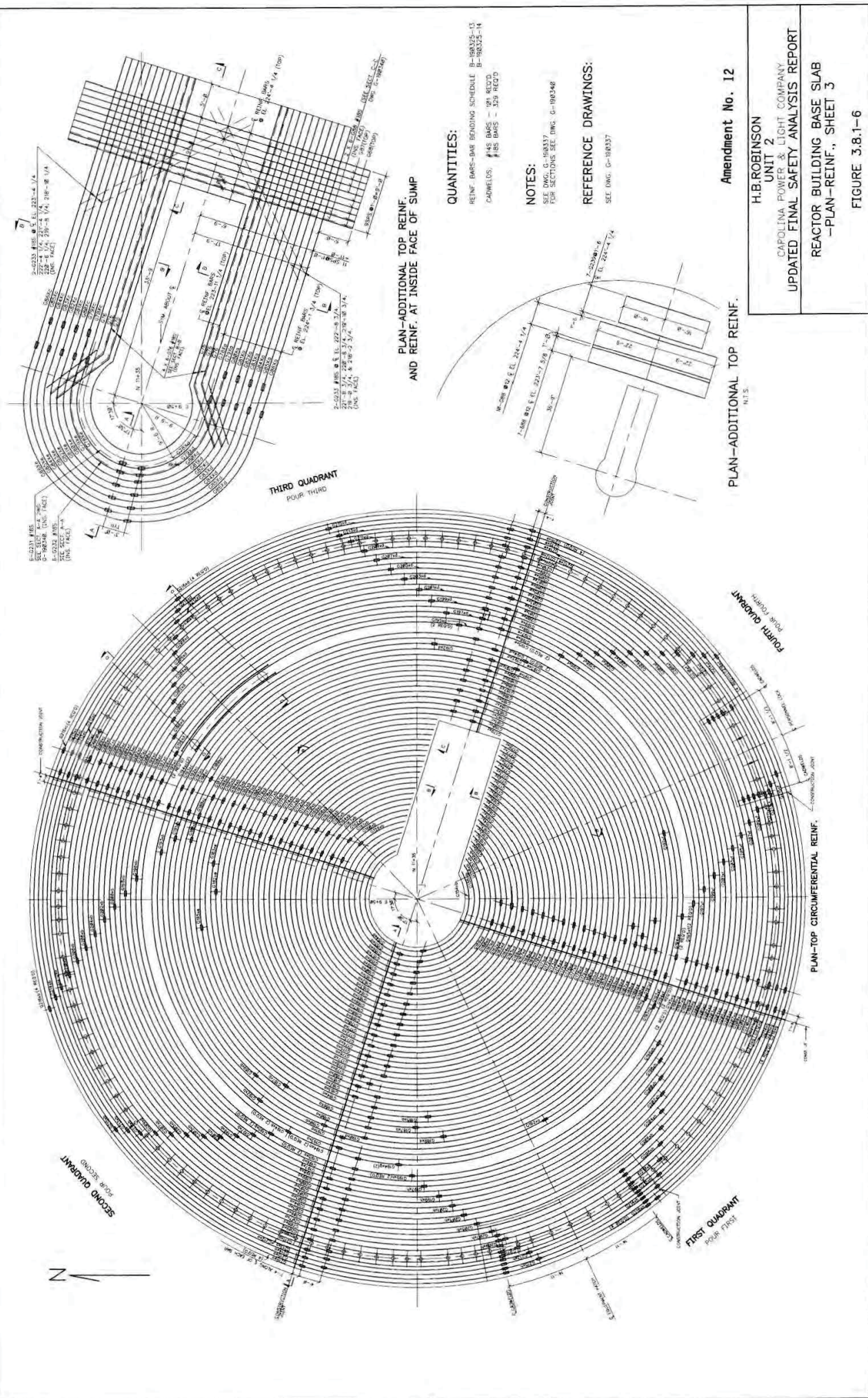
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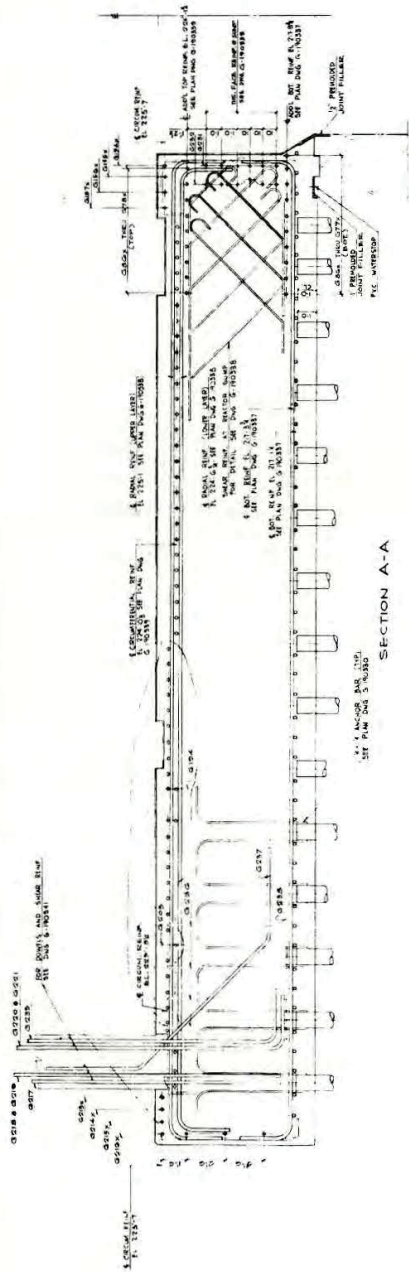
REACTOR BUILDING BASE SLAB
-PLAN-REINF., SHEET 1

Amendment No. 12

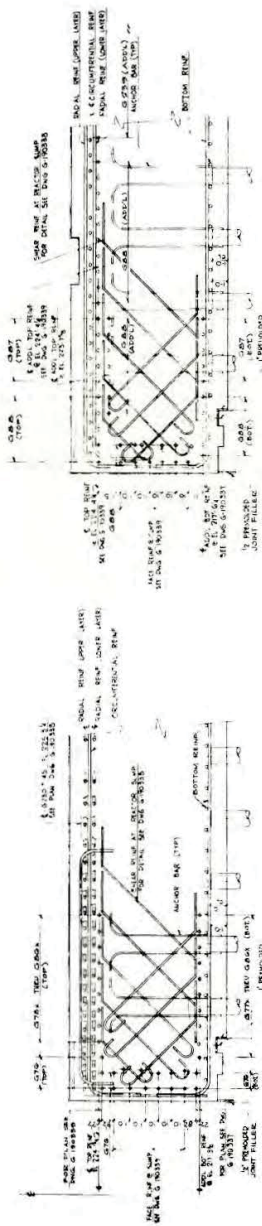
FIGURE 3.8.1-4





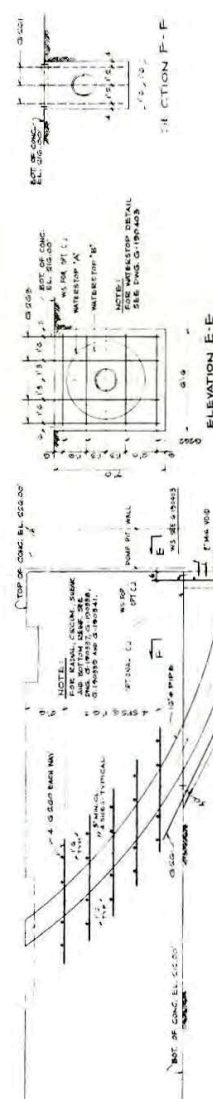


SECTION A-A



SECTION B-B

SECTION C-C



SECTION D-D

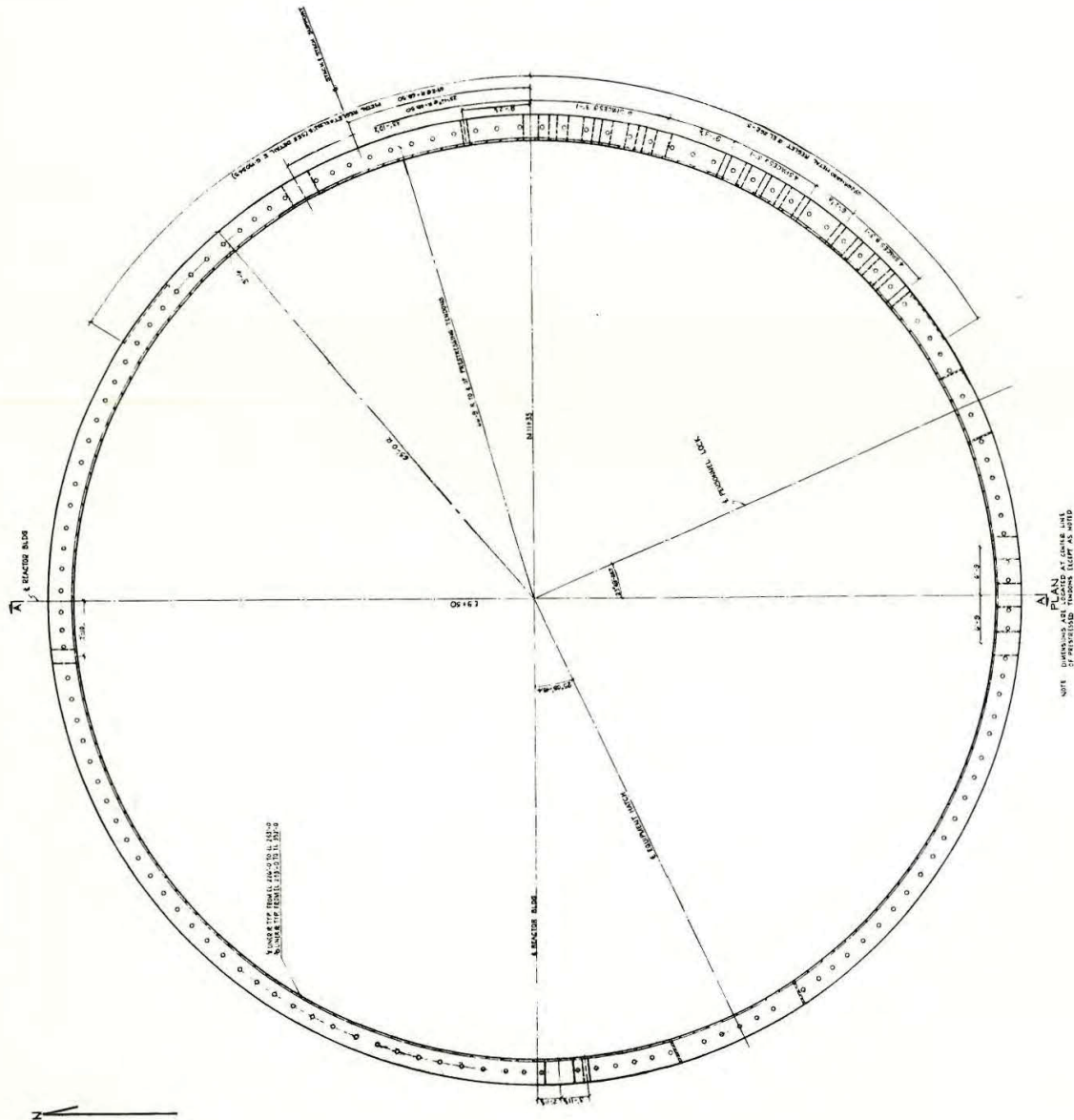
ELEVATION E-E

SECTION F-F

QUANTITIES:
CONC. BASE SLAB 8-10225.0
REIN. 8-10225.0
PTE. WATERSTOP 8-10225.0

NOTES:
1. SEE DRAWING 8-10331
2. SEE DRAWING 8-10332

REFERENCE: DRAWINGS:
1. SEE DRAWING 8-10331
2. SEE DRAWING 8-10332



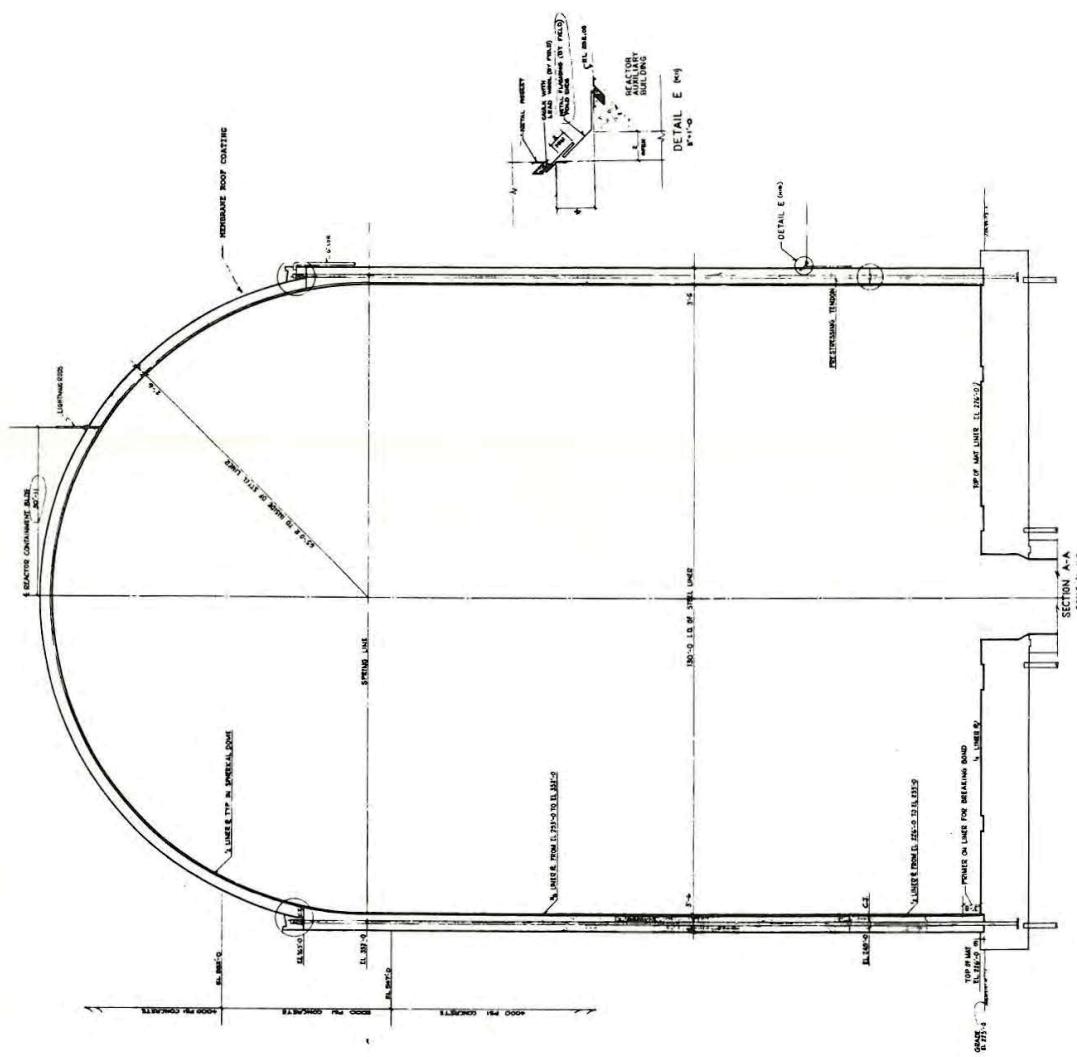
QUANTITIES
METAL REINLET 130'-4"
CONCRETE 4000 PSI CLASS AA 7620 CU. YDS.
CONCRETE 5000 PSI CLASS AA 1710 CU. YDS.

NOTES:
CURRENT A.C. STANDARDS SHALL APPLY FOR ALL DESIGN AND CONSTRUCTION UNLESS OTHERWISE NOTED.
CONCRETE SHALL BE CLASS AA 4000 PSI & 5000 PSI.
SEE SPECIFICATION EMBACO 14-45 FOR CONCRETE.
ALL ANCHOR BOLTS, DRAIN PIPES, PIPE SLEEVES, ELECTRICAL CONDUITS AND EMBEDDED PARTS SHALL BE IN POSITION BEFORE CONCRETE IS PLACED.

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CONTAINMENT VESSEL PLAN - MAS.

FIGURE 3.8.1 - 8

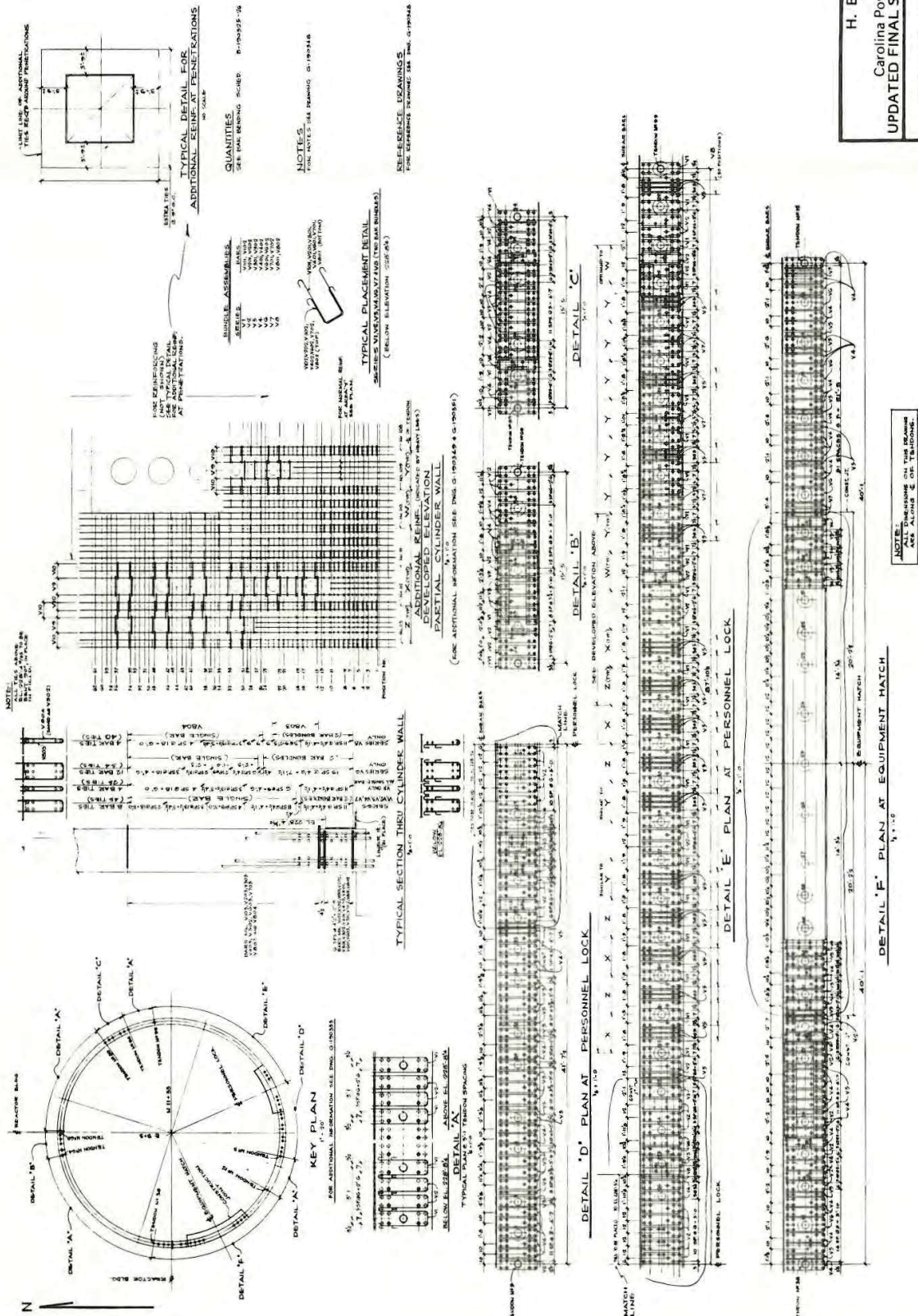


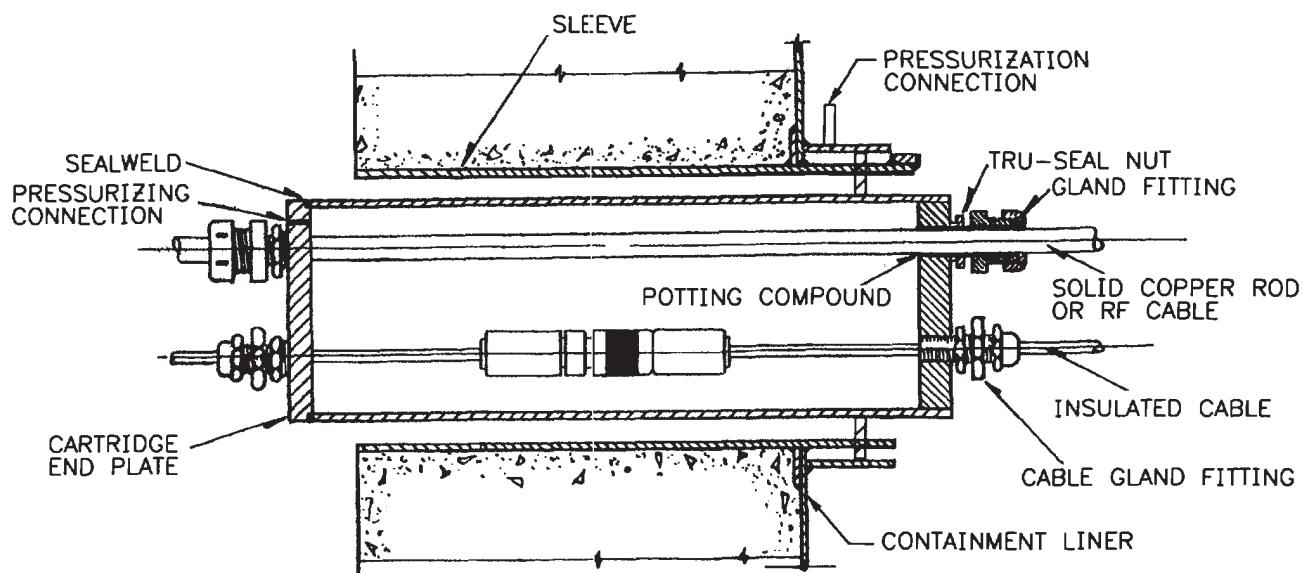
AMENDMENT NO. 6

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 CONTAINMENT VESSEL
 SECTIONS AND DETAILS - DETAILS
 FIGURE 3.8.1 - 9

G 190 343

LOCATION SCHEDULE - HORIZONTAL REINFORCING									
FROM POSITION	TO POSITION	NUMBER	IS	MAINTENANCE	NO.	FROM POSITION	TO POSITION	NUMBER	IS
110	111	178	178	178	178	111	112	179	179
112	113	179	179	179	179	113	114	180	180
114	115	180	180	180	180	115	116	181	181
116	117	181	181	181	181	117	118	182	182
118	119	182	182	182	182	119	120	183	183
120	121	183	183	183	183	121	122	184	184
122	123	184	184	184	184	123	124	185	185
124	125	185	185	185	185	125	126	186	186
126	127	186	186	186	186	127	128	187	187
128	129	187	187	187	187	129	130	188	188
130	131	188	188	188	188	131	132	189	189
132	133	189	189	189	189	133	134	190	190
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136	137	191	191	191	191	137	138	192	192
138	139	192	192	192	192	139	140	193	193
140	141	193	193	193	193	141	142	194	194
142	143	194	194	194	194	143	144	195	195
144	145	195	195	195	195	145	146	196	196
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148	149	197	197	197	197	149	150	198	198
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164	165	205	205	205	205	165	166	206	206
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168	169	207	207	207	207	169	170	208	208
170	171	208	208	208	208	171	172	209	209
172	173	209	209	209	209	173	174	210	210
174	175	210	210	210	210	175	176	211	211
176	177	211	211	211	211	177	178	212	212
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392	393	319	319	319	319				





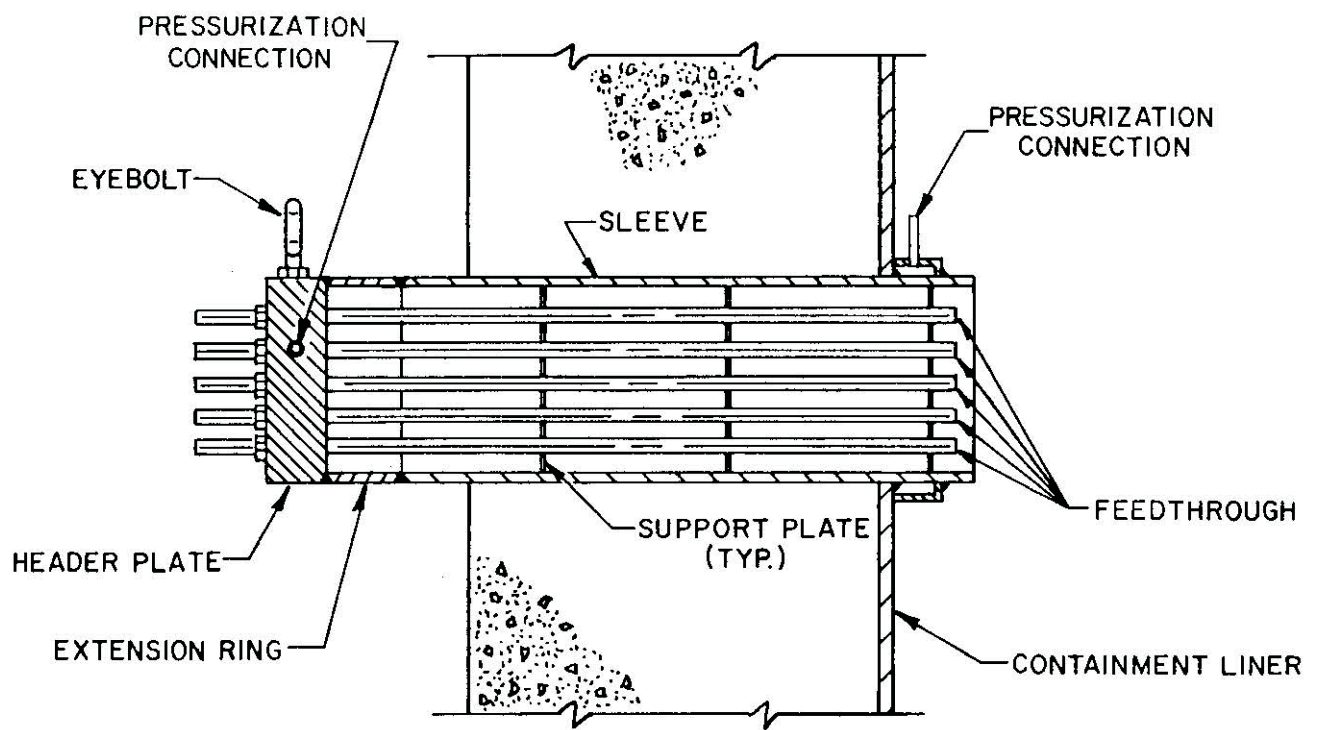
ELECTRICAL PENETRATION
CARTRIDGE DESIGN

REF. DWG. 5379-1867 REV. 3
REVISION 16

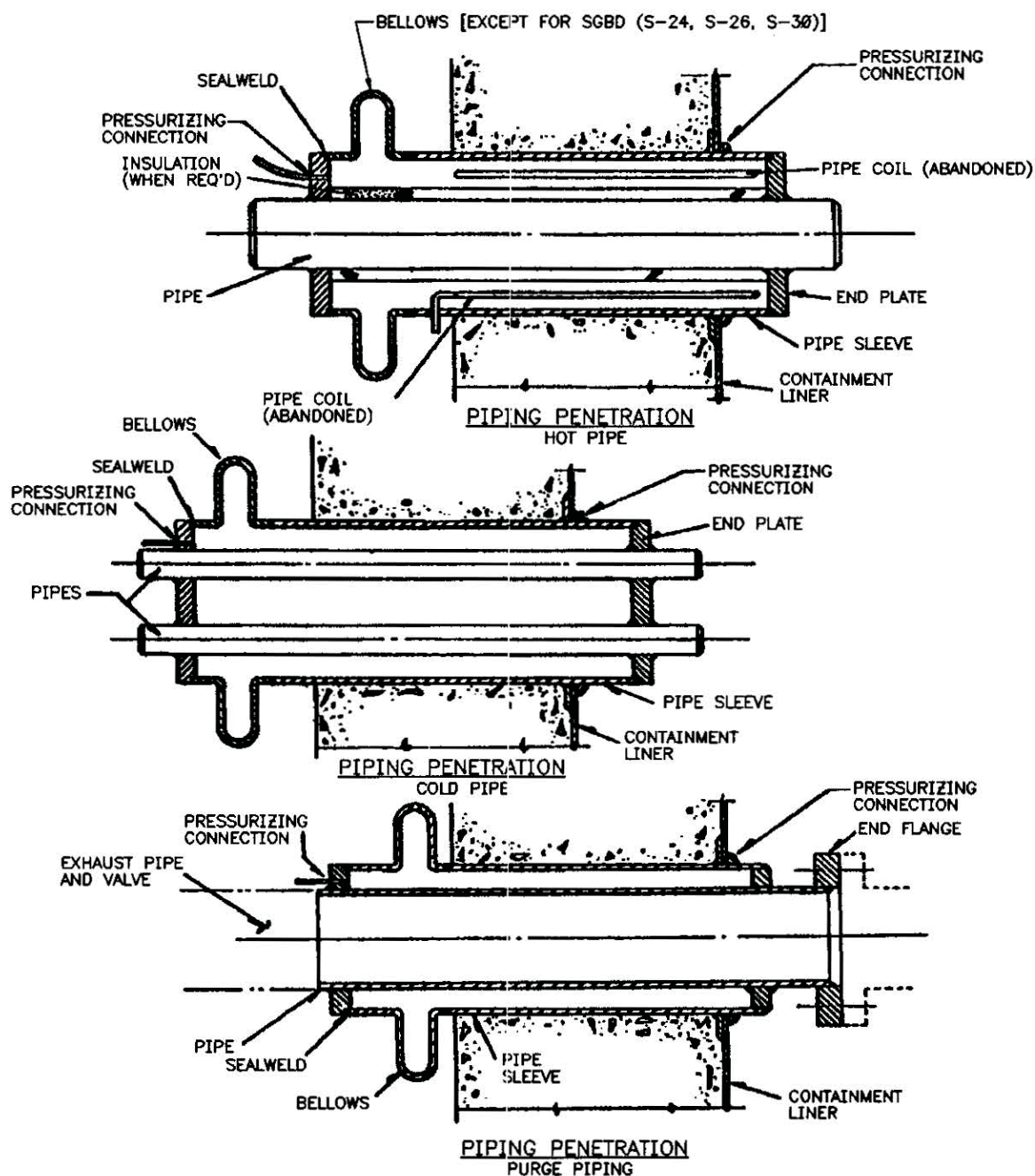
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SAFETY ANALYSIS REPORT

ELECTRICAL PENETRATIONS

FIGURE
3.8.1-14



AMENDMENT 3

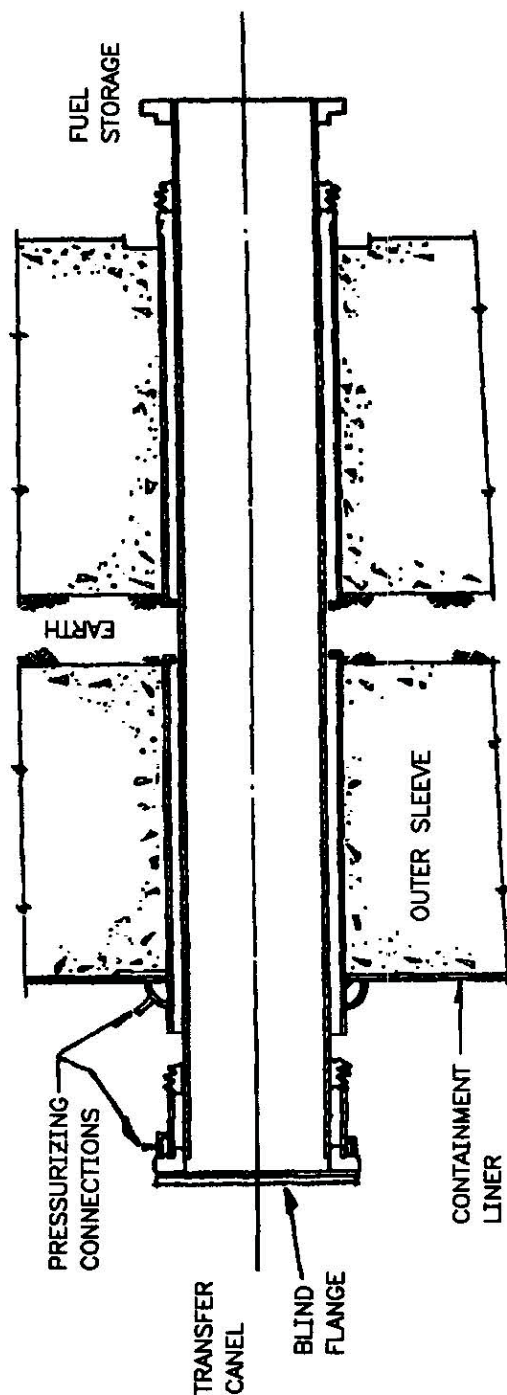


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TYPICAL PIPING PENETRATIONS

FIGURE
3.8.1-15

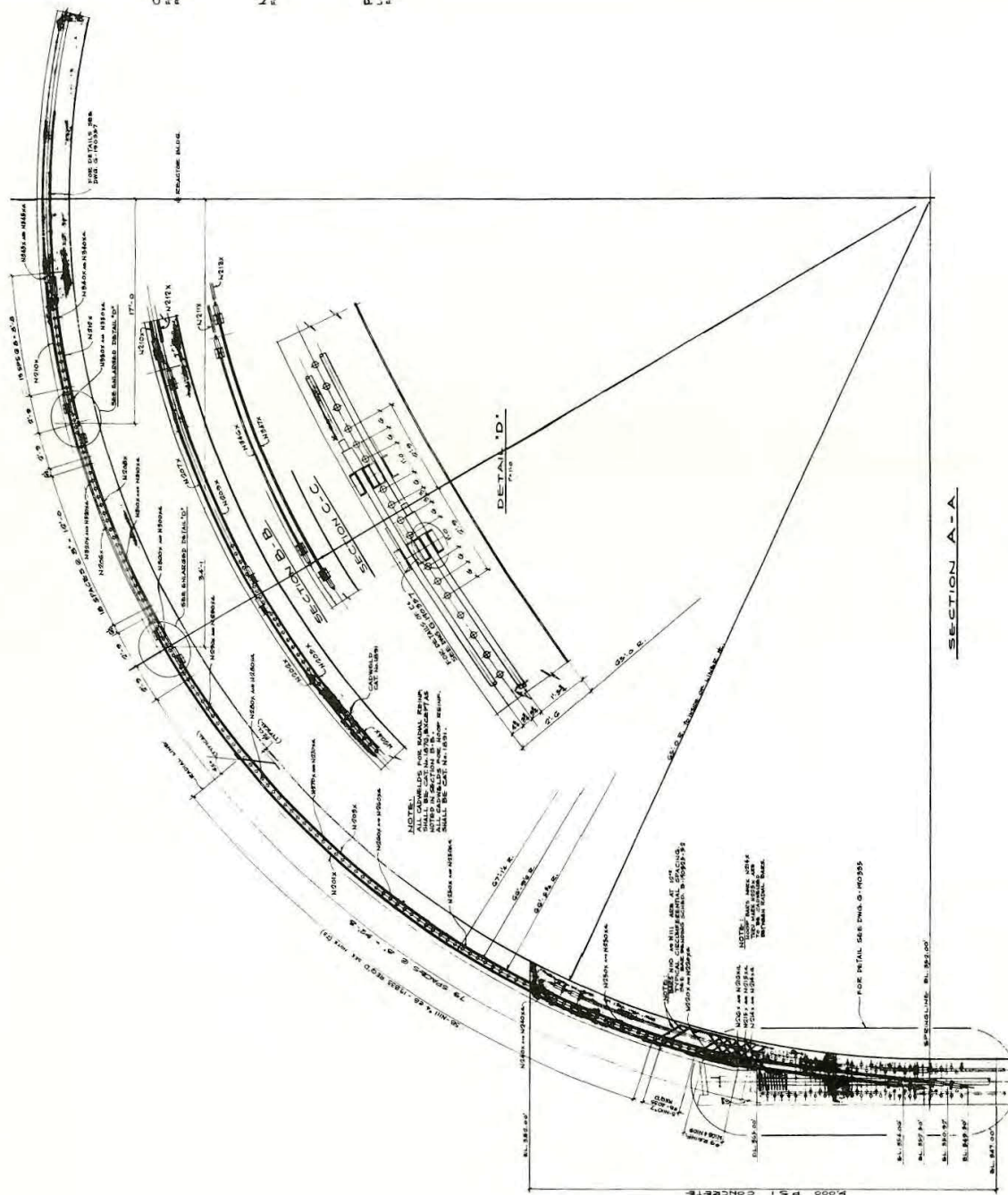


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SAFETY ANALYSIS REPORT

FUEL TRANSFER PENETRATION

FIGURE
3.8.1-16



QUANTITIES:
FOR ORDER, SEE BAR BENDING SCHEDULE B-190323-52753
FOR CADWELDS SEE G-190355

NOTES:
FOR NOTES SEE DWG. Q-190365

REFERENCE DRAWINGS:
 LIST OF DRAWINGS - A-190172
 FOR ADOL. REF. DWGS. SEE C-190355

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REACTOR BUILDING DOME
- DETAILS - REINF
FIGURE 3.8.1 - 18



QUANTITIES
AS PER DETAIL.

NOTES
ALL STEEL SHALL BE ASTM A 36 UNLESS OTHERWISE
NOTED.

REFERENCE DRAWINGS

LIST OF DRAWINGS	
REACTOR BLOCK	DOSE, PLAN, REINF
MIXER BLOCK	DOSE, DETS., REINF

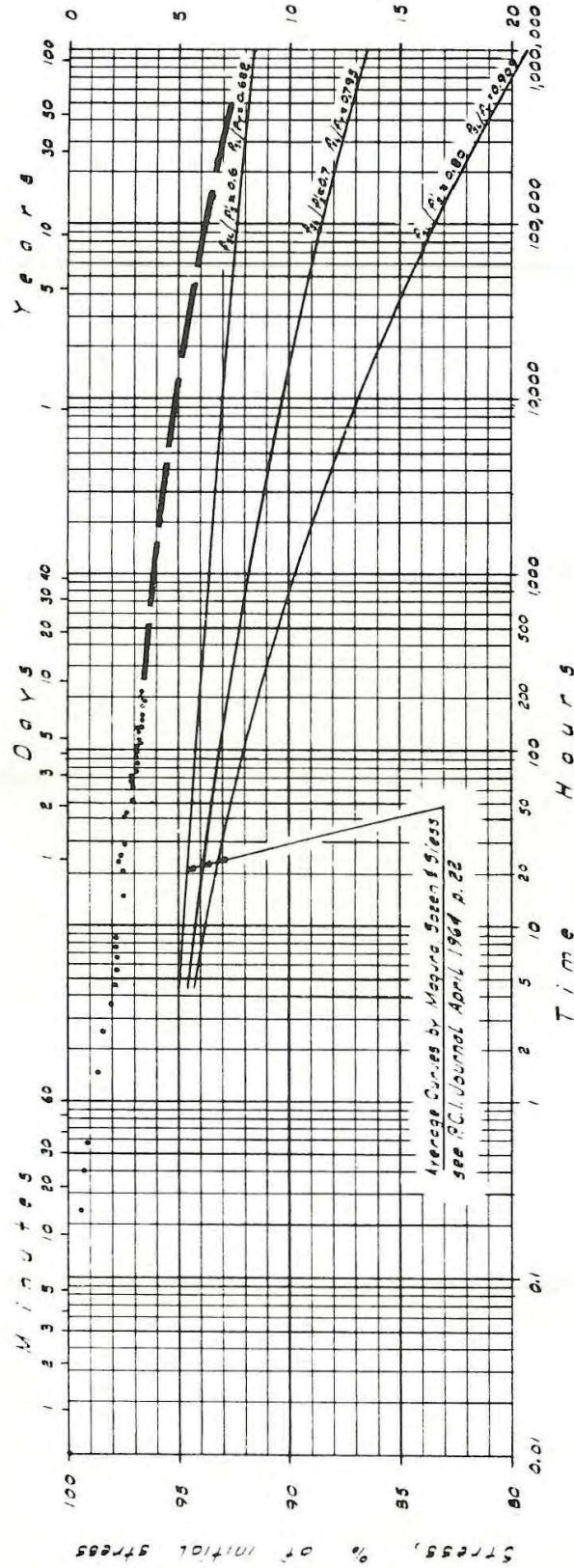
A-150792
G-150305
G-150386

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REACTOR BUILDING DOME
STEEL PLAN, SECTIONS AND DETAILS

FIGURE 3.8.1 - 19

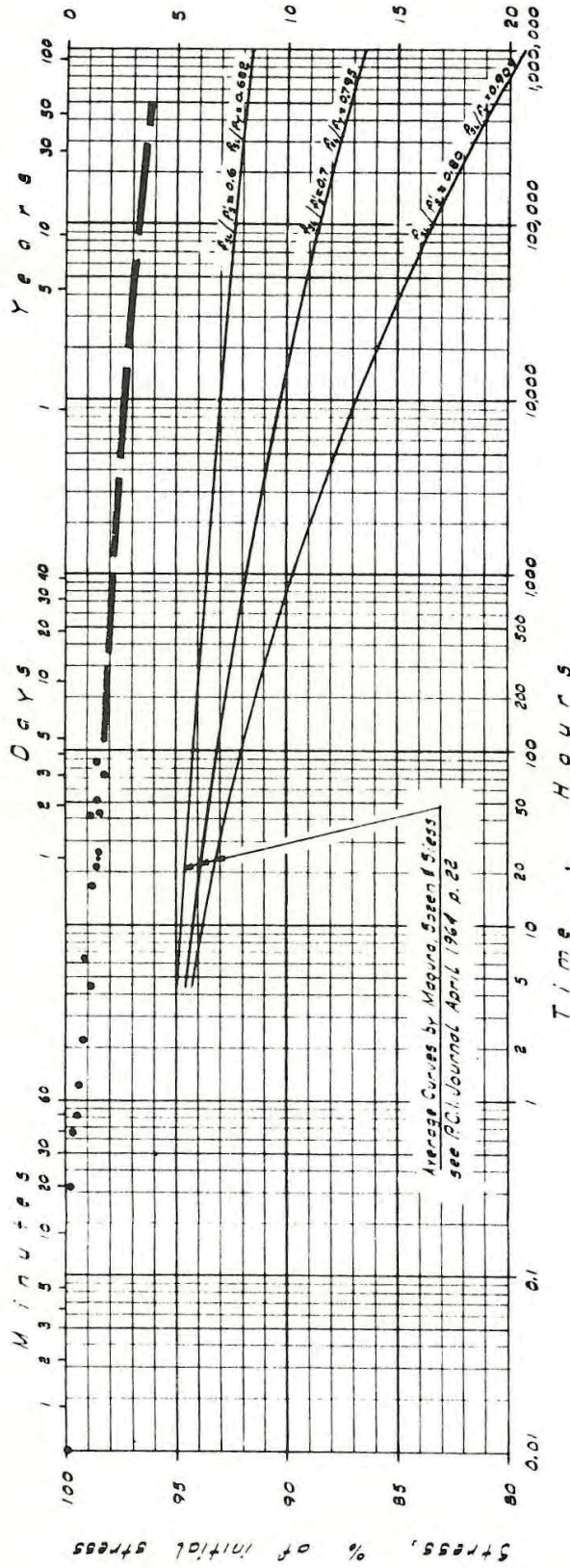
RELAXATION TEST



Tendon: $\frac{1}{8}$ " Stress steel bar Initial Stress = $f_i = 128.0$ ksi $f_s = 160.1$ ksi
 Tested by: Fritz Engineering $f_i/f_s = 800\%$ Temp = $90^\circ F$
 Laboratory - Lehigh Univ. $f_r = 0.1\%$ offset stress = 147 ksi $f_i/f_r = 87\%$
 Date of test: January 24, 1967

Graph No. 1

RELAXATION TEST



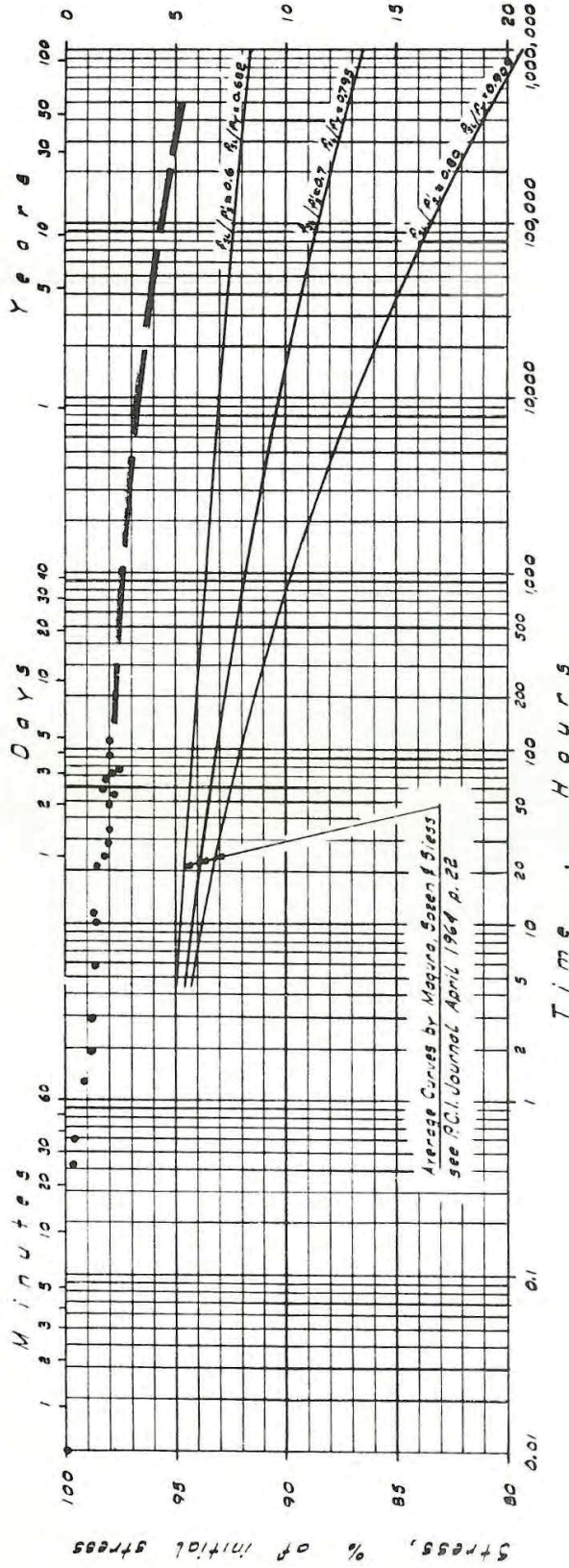
Tendon: 1 1/8" Stress steel Bar Initial Stress = $f_i = 110.7$ ksi $f'_s = 158.6$ ksi

Tested by: Fritz Engineering Laboratory - Lehigh Univ. $f_{si}/f'_s = 70\%$ Temp = 82 °F or

Date of test: July 14, 1957 $f_r = 0.1\%$ offset stress = ____ ksi $f_{si}/f_r = \%$

Graph No. 2

RELAXATION TEST



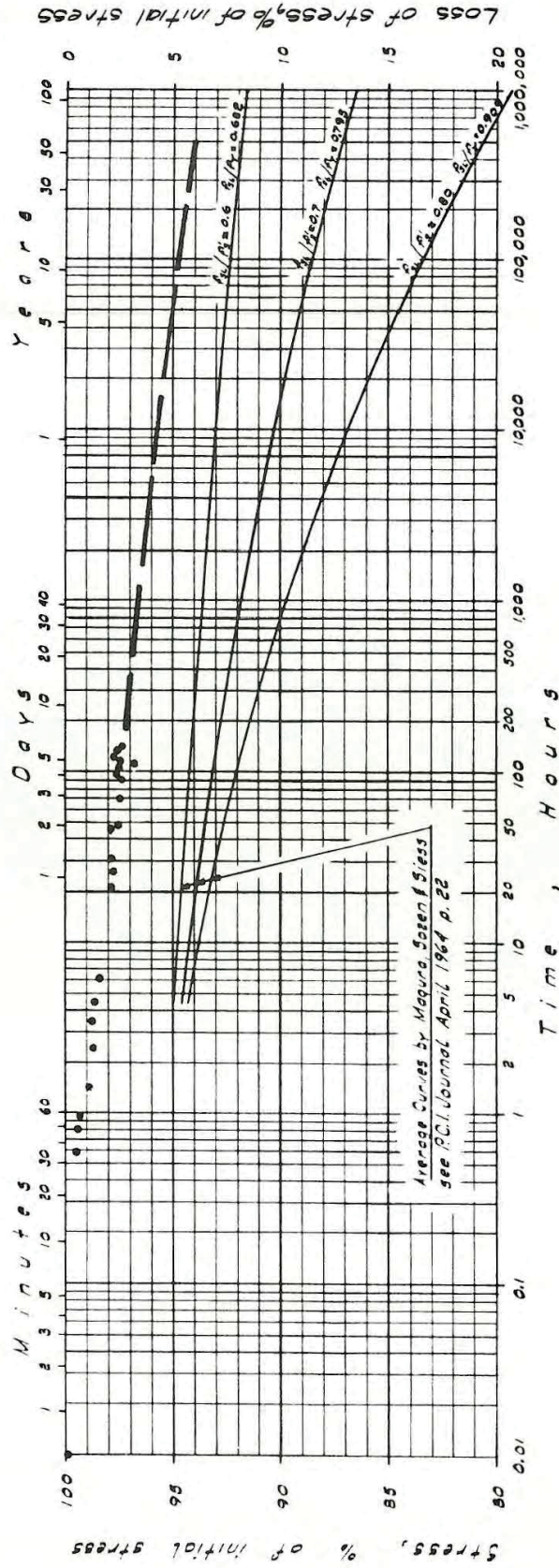
Tendon: 1-1/8" Stresssteel Bar Initial Stress = $f_i = 120.7$ ksi $f_s = 157.4$ ksi $f_s/f_i = 1.303$

Tested by: Fritz Engineering Laboratory - Lehigh Univ. $f_s/f_i = 77\%$ Temp = 81 °F

Date of test: July 17, 1957 $f_r = 0.1\%$ offset stress = ksi $f_s/f_r = 1574\%$

Graph No. 3

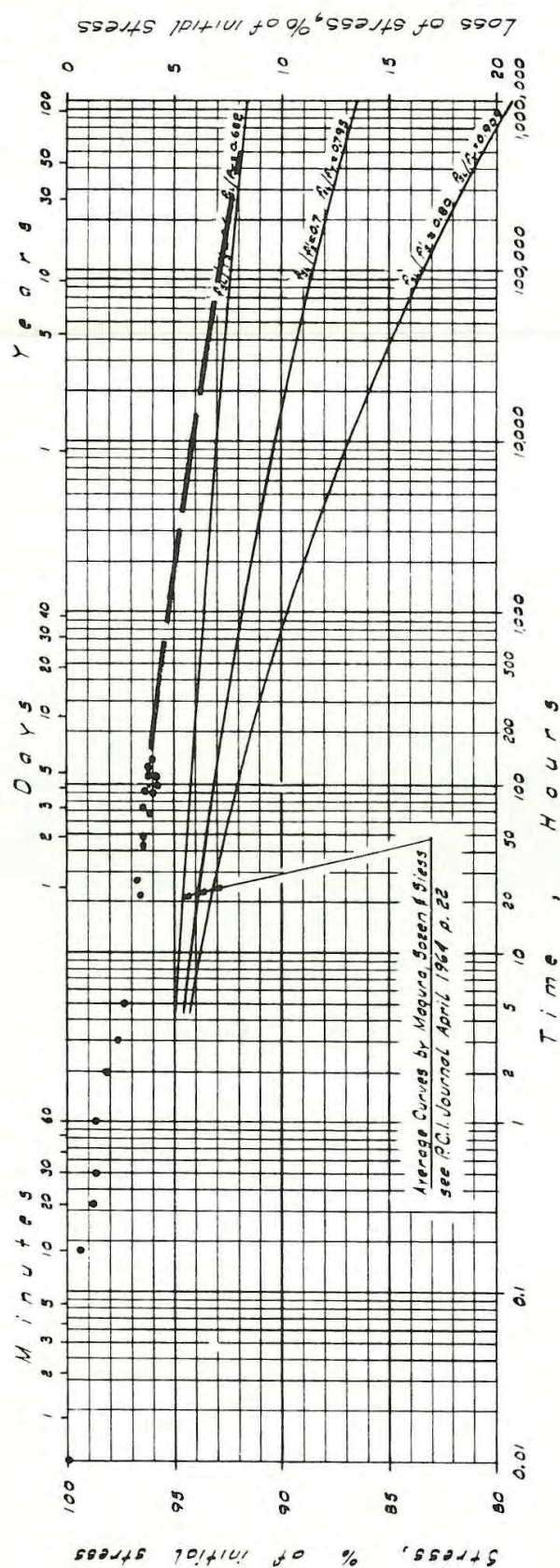
RELAXATION TEST



Tendon: 1/8" ϕ Stresssteel Bar Initial Stress = $f_0 = 110.6$ ksi $f'_0 = 160.2$ ksi
 Tested by: Fritz Engineering Laboratory - Lehigh Univ $f_{si}/f'_0 = 69\%$ Temp = 76.0 °F
 Date of test: October 9, 1953 $f_y = 0.1\%$ offset stress = ksi $f_{si}/f_y = \underline{\hspace{1cm}}$ %

Graph No. 4

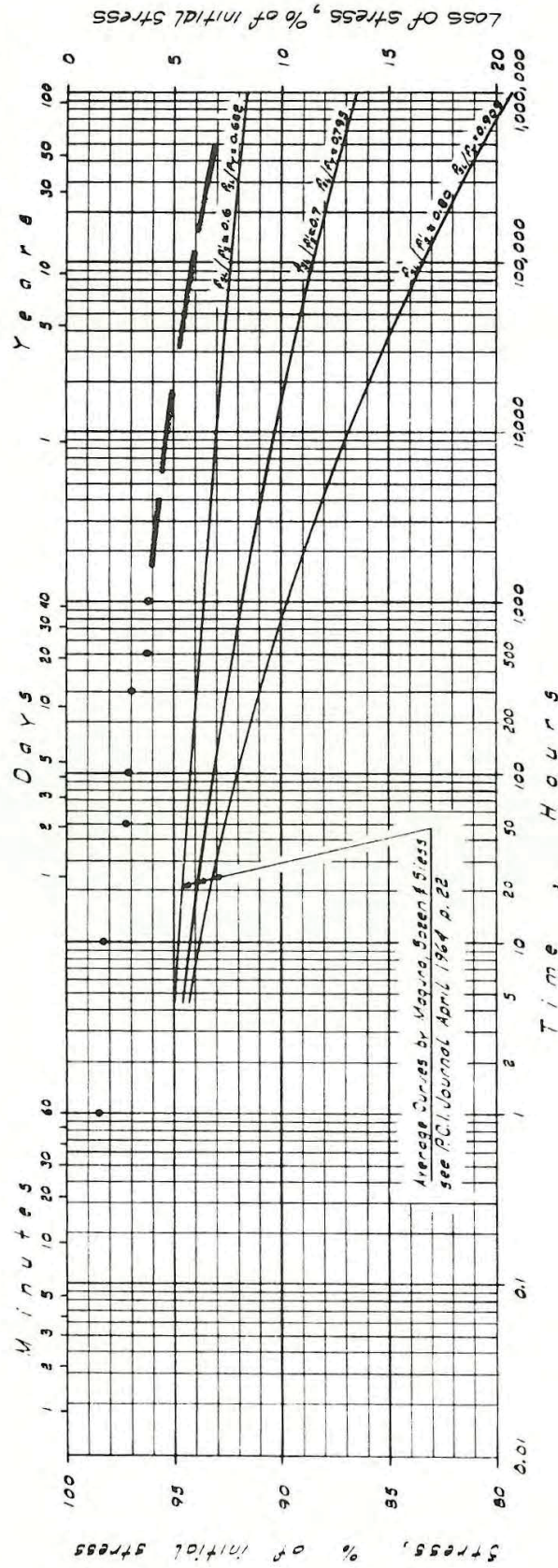
RELAXATION TEST



Tendon: 1/8" ϕ Stresssteel Bar Initial Stress = $f_{si} = 120.7$ ksi $f'_s = 159.0$ ksi
 $f_{si}/f'_s = 76.0\%$ Temp = 76.0 °F
 $f_y = 0.1\%$ offset stress = ksi $f_{si}/f_y = \underline{\hspace{1cm}}\%$

Graph No 5

RELAXATION TEST



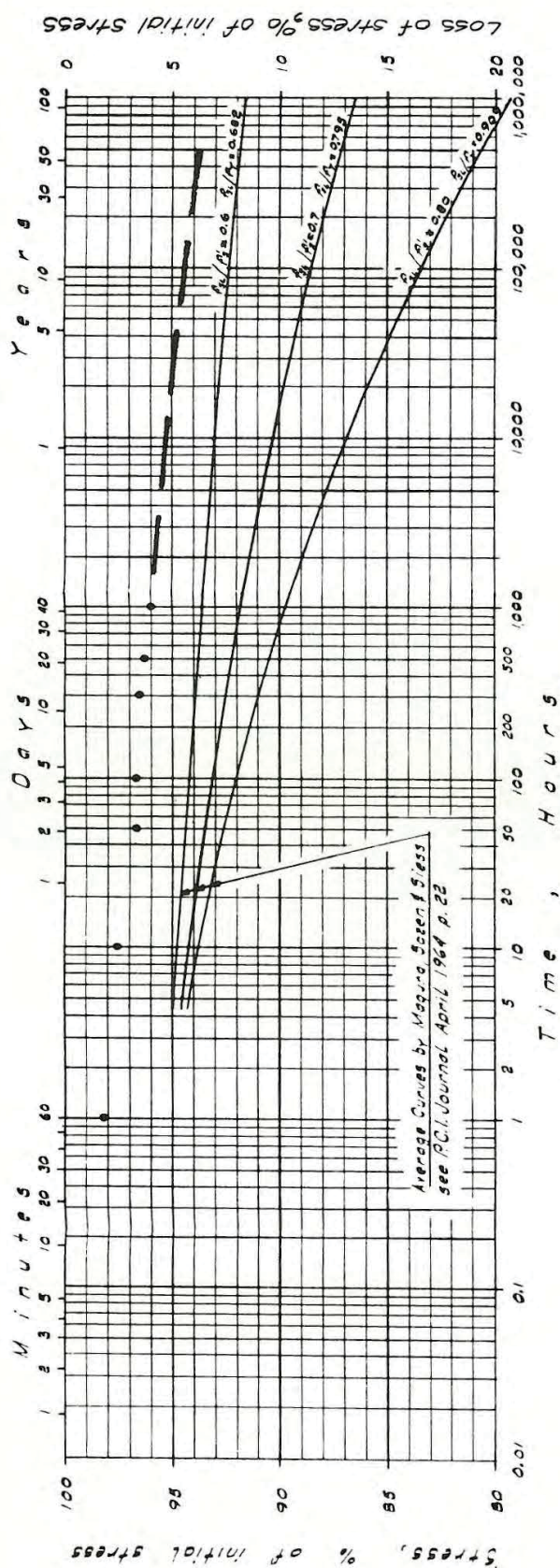
Tendon: 1" rolled bar Barrow Steel Initial Stress = $f_i = 94$ ksi $f_s = 150$ ksi
 Works Ltd. - Cast No. 8/62/112
 Tested by: The United Steel Companies Ltd. $f_s / f_i = 66.5$ %
 Research & Development Dept.
 Date of test: July 3 1958 $f_r = 0.1$ % offset stress = ____ ksi $f_s / f_r =$ %
 Temp = room

Graph No 6

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1" DIA. ROLLED BAR
 RELAXATION TEST
 FIGURE 3.8.1 - 25

RELAXATION TEST



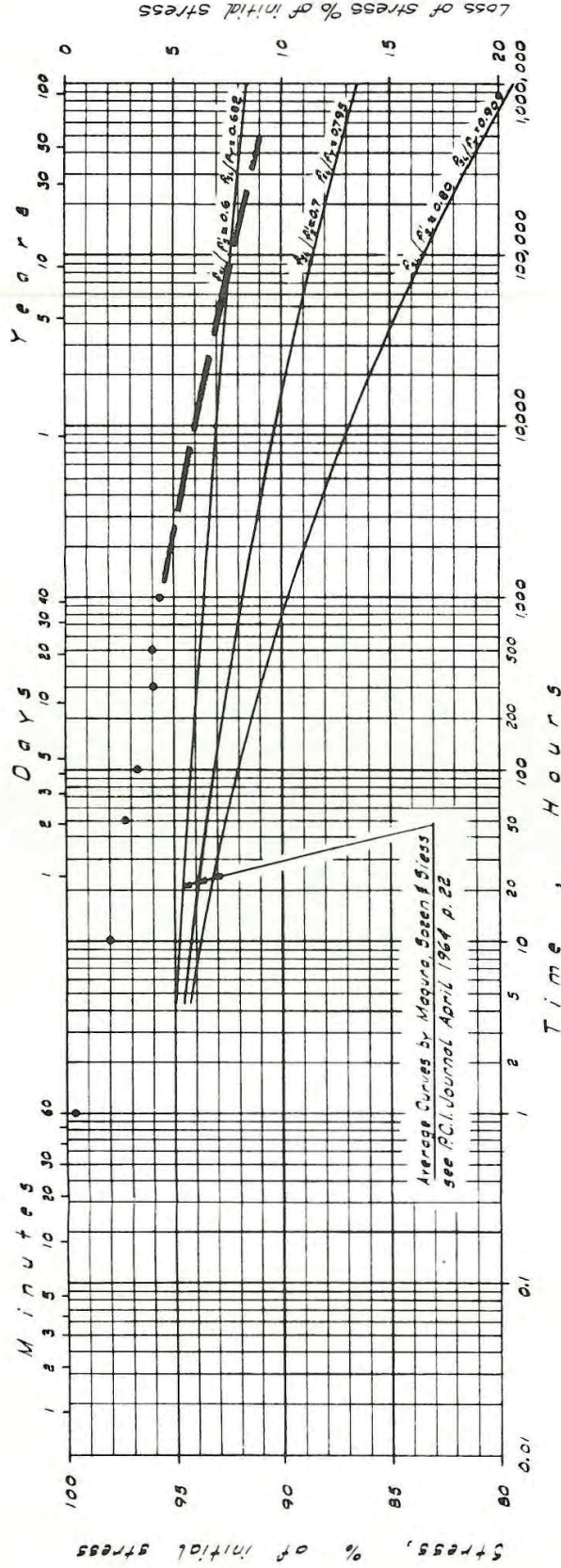
Tendon: 1" rolled bar, Barrow Steelworks Ltd. (cast #B/842/112) Initial Stress = $f_i = 101$ ksi $f'_i = 150$ ksi
 Tested by: The United Steel Companies Ltd. $f_{s1}/f'_i = 67.3$ % Temp = $\frac{room}{temp}$ °F
 Date of test: July 3, 1958 $f_y = 0.1$ % offset stress = ____ ksi $f_{s1}/f_y =$ %

Graph No. 7

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1" DIA. ROLLED BAR
 RELAXATION TEST
 FIGURE 3.8.1-26

RELAXATION TEST



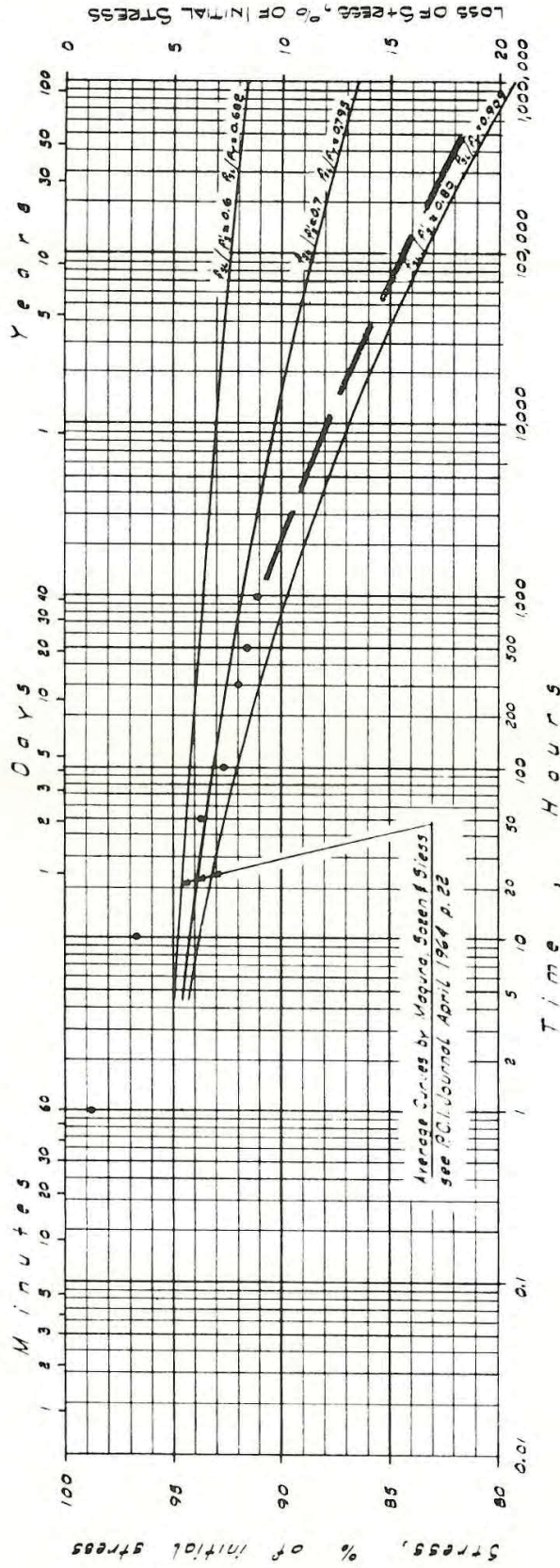
Tendon: 1" ϕ rolled bar, Barrow Steelworks Ltd (cast # B/862/112) Initial Stress = $f_{si} = 78.4$ ksi $f'_s = 150$ ksi

Tested by: The United Steel Companies (Ed.) $f'_s / f_s = 52.2$ % Temp = 212 °F

Date of test: June 30, 1957 $f_r = 0.1$ % of set stress = ____ ksi $f_{si} / f_r =$ %

Graph No. 8

RELAXATION TEST



Tendon: 1" dia rolled bar - Barrow
 Steel Works Ltd. - Cast # B/862/112
 Tested by: The United Steel Companies Ltd. f₁: f₂: f₃ = 52.3 %
 Research & Development Dept.
 Date of test: June 30, 1957
 f_r = 0.1 % offset stress = _____ ksi
 f₁: f₂: f₃ = _____ %
 Initial Stress = f₁ = 78.4 ksi f₂ = 150 ksi
 Temp = 392 °F

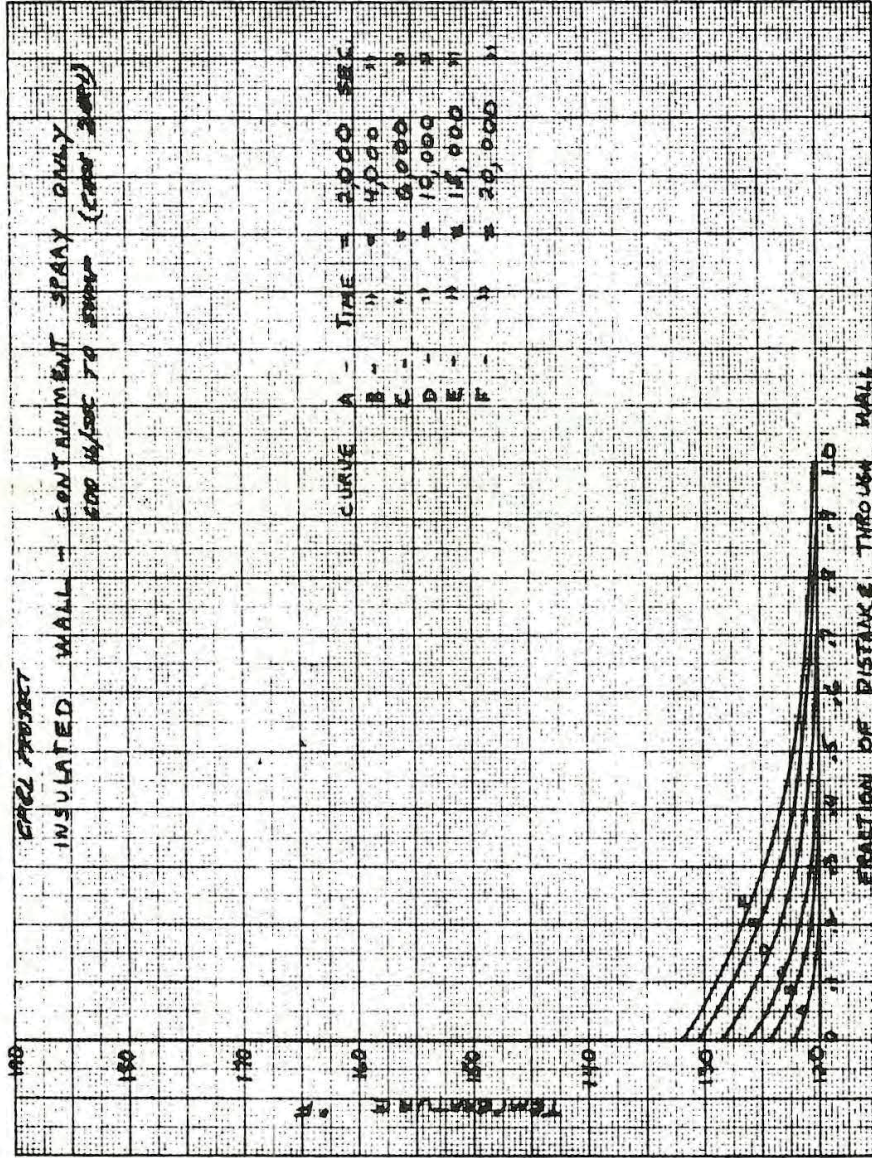
Graph No 9

BL. 365'	BL. 342'	BL. 320'	MOMENT (100')	SHEAR (K/1)	INTERNAL MOMENT (100')	TANGENTIAL FORCE (K/1)
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40	40	40	40	40	40	40
20	20	20	20	20	20	20
0	0	0	0	0	0	0
100	100	100	100	100	100	100
80	80	80	80	80	80	80
60	60	60	60	60	60	60
40						

NOTE: MOMENTS SHOWN ON TENSION SIDE.
(+) INDICATES TENSION, (-) INDICATES COMPRESSION.

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ACCIDENT TEMPERATURE - LINER LOADS



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UNIT 2

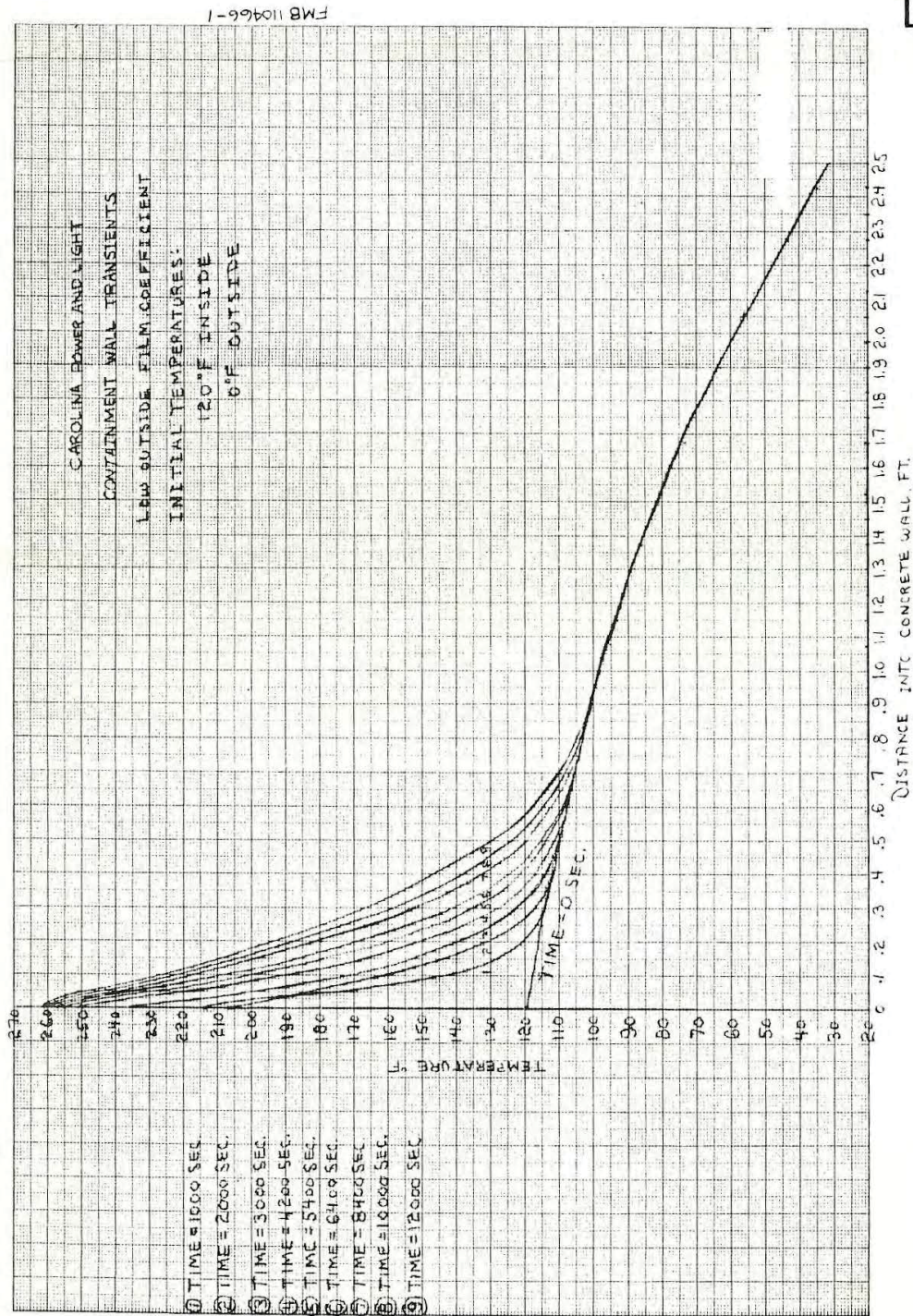
Carolina Power & Light Company

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TEMPERATURE PROFILES INSULATED WALL

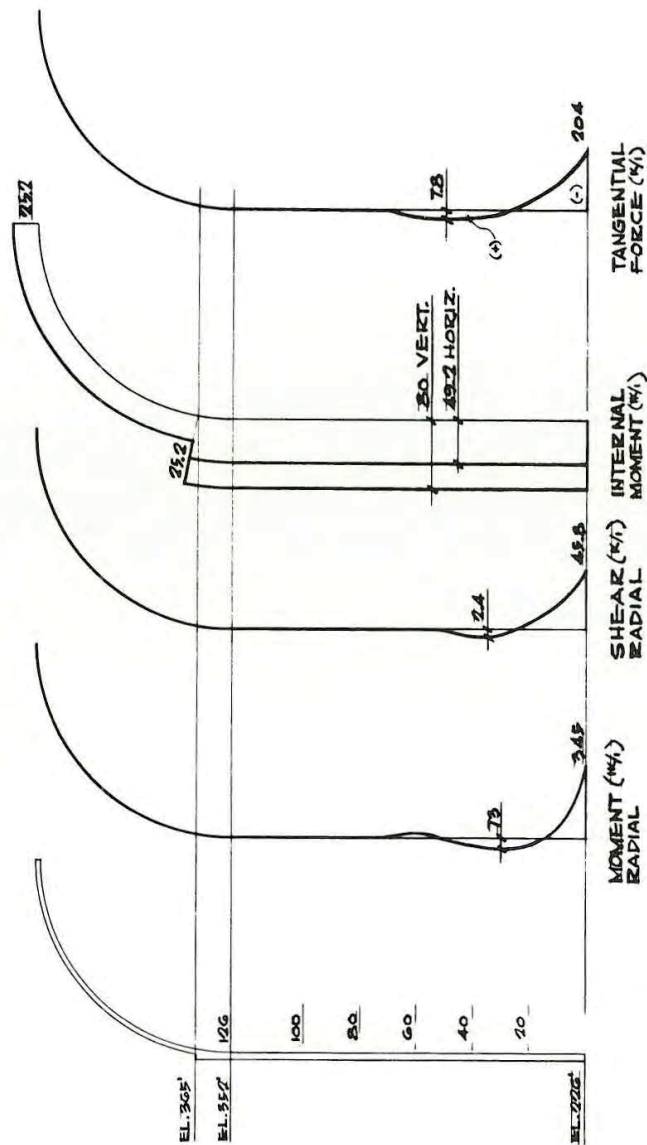
CONTAINMENT SPRAY ONLY

FIGURE 3.8.1 - 30



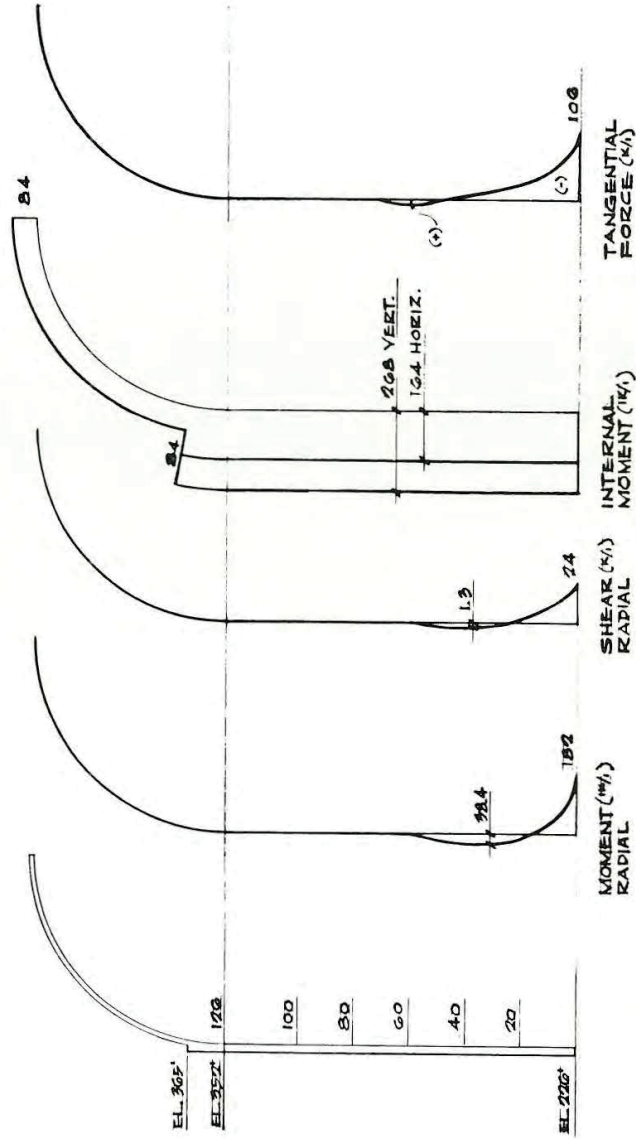
FMB 110466-1

SUMMER OPERATION
(CALCULATIONS ASSUME USE OF LINER #.)



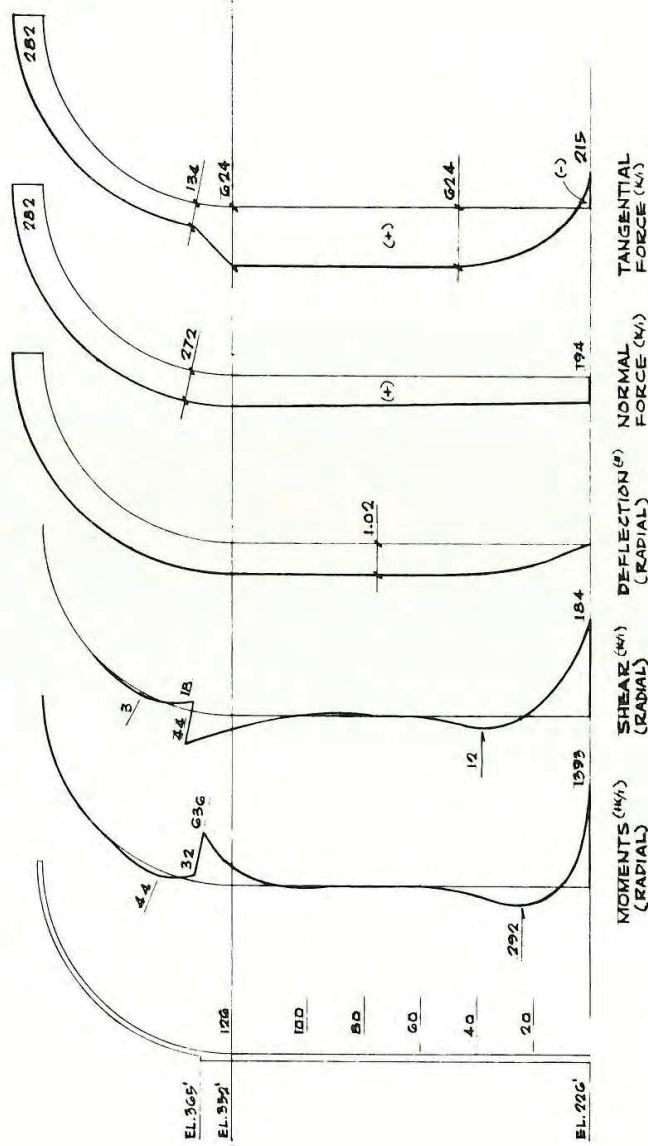
NOTE: MOMENTS SHOWN ON TENSION SIDE.
(+) INDICATES TENSION (-) INDICATES COMPRESSION.

WINTER OPERATION
(CALCULATIONS ASSUME USE OF LINER #2)



NOTE: MOMENTS SHOWN ON TENSION SIDE.
(+) INDICATES TENSION (-) INDICATES COMPRESSION.

$$1.0D \pm 0.05D + 1.5P + 1.0(T+TL)$$



NOTE: VALUES SHOWN ARE MAX. AT EACH POINT. THEY MAY NOT OCCUR AT THE SAME TIME.
 MOMENTS SHOWN ON TENSION SIDE. (+) INDICATES TENSION. (-) INDICATES COMPRESSION.
 OPERATING TEMPERATURE CONSIDERED ONLY AT THE BOTTOM OF THE CYLINDER.

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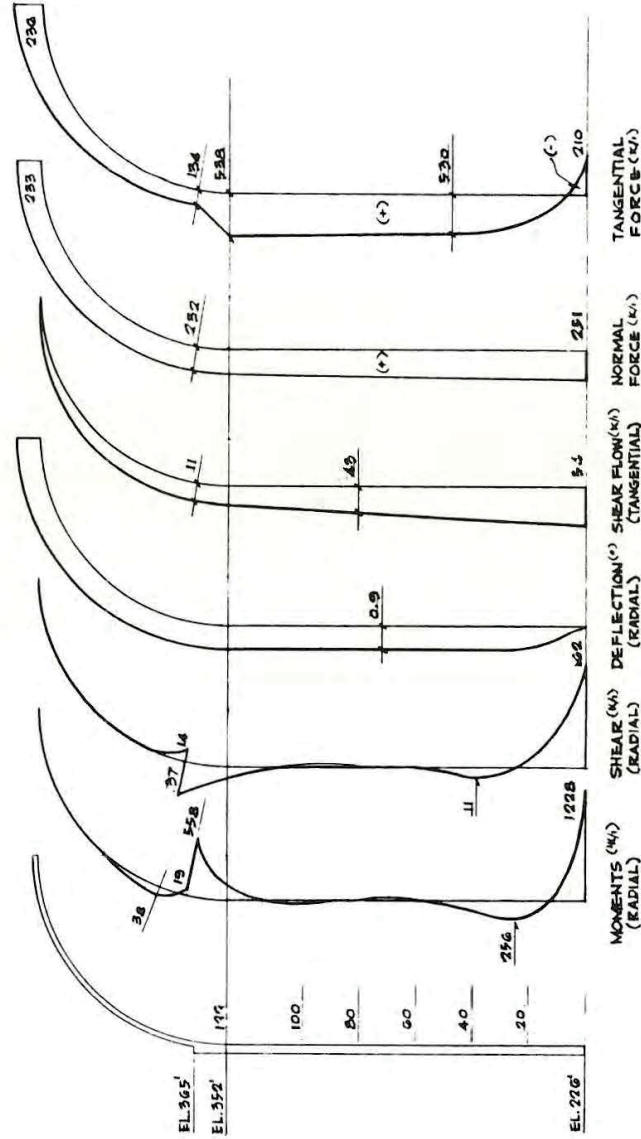
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MOMENT STRESS
 AND DEFLECTION DIAGRAM

$$C = 1.00 \pm 0.050 + 1.5P + 1.0(T+TL)$$

FIGURE 3.8.1 - 34

$$1.0D \pm 0.05D + 1.25P + 1.0(T' + TL') + 1.25E$$



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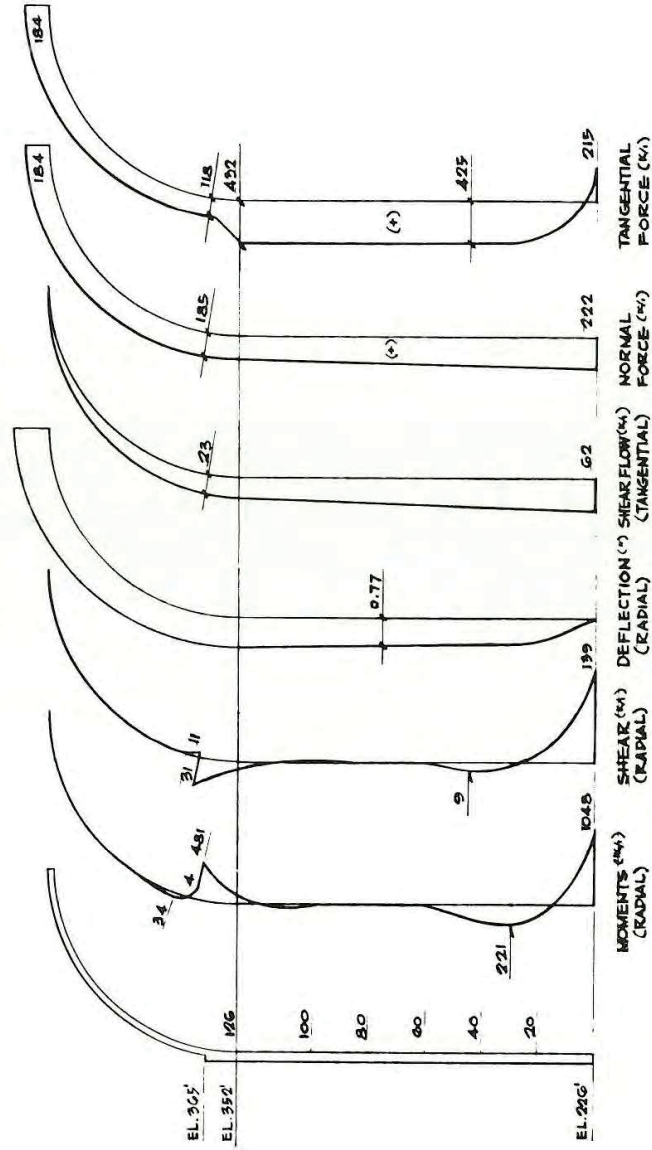
UPDATED FINAL SAFETY ANALYSIS REPORT

MOMENT, STRESS, DEFLECTION
AND FLOW DIAGRAM

$C = 1.0D \pm 0.05D + 1.25P + 1.0(T' + TL') + 1.25E$

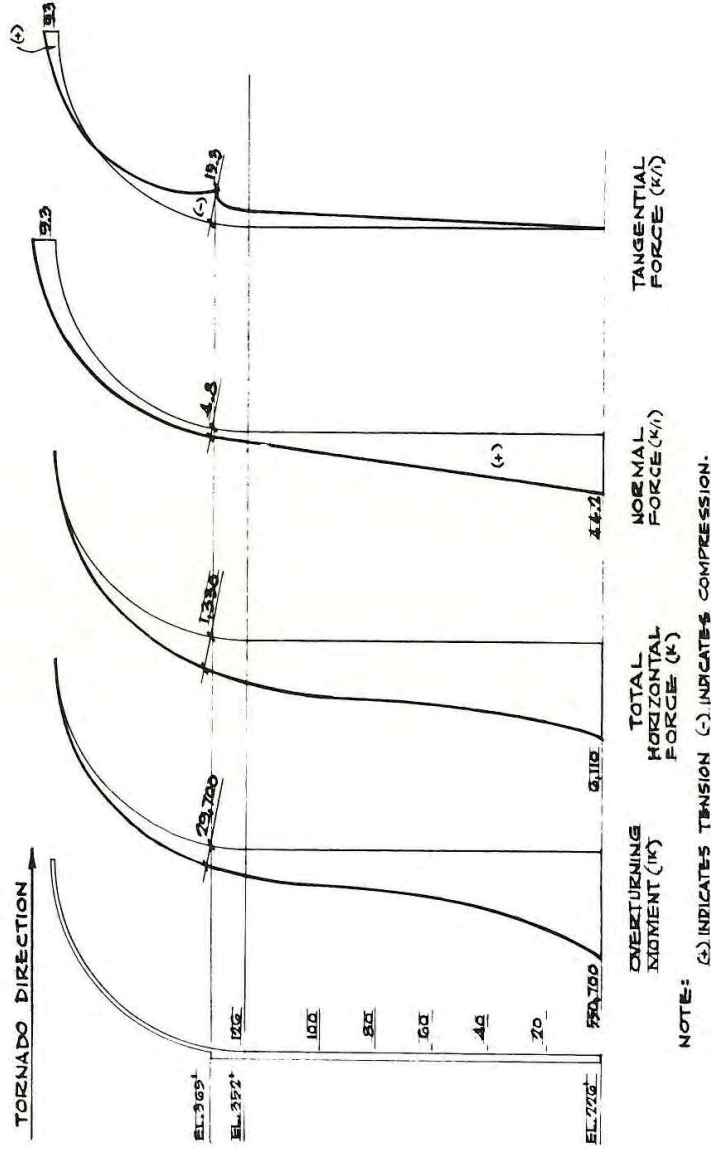
FIGURE 3.8.1 - 35

$$1.0D \pm 0.05D + 1.00P + 1.0(T' + TL') + 1.00E'$$

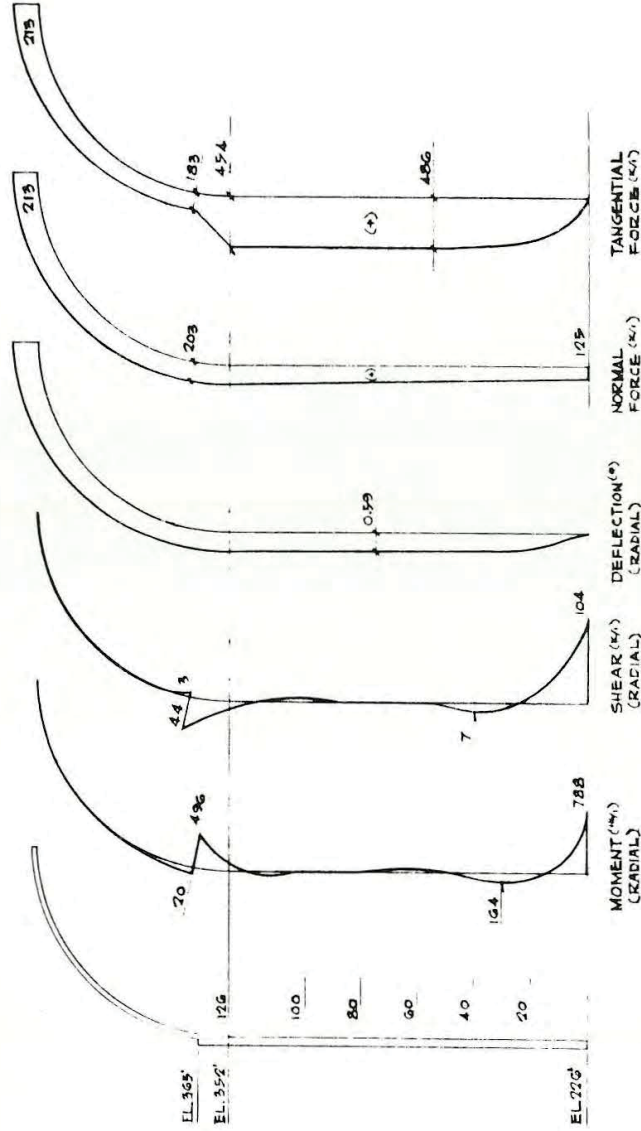


NOTE: VALUES SHOWN ARE MAX. AT EACH POINT. THEY MAY NOT OCCUR AT THE SAME TIME. MOMENTS SHOWN ON TENSION SIDE. (+) INDICATES TENSION (-) INDICATES COMPRESSION. OPERATING TEMPERATURES CONSIDERED ONLY AT THE BOTTOM OF THE CYLINDER

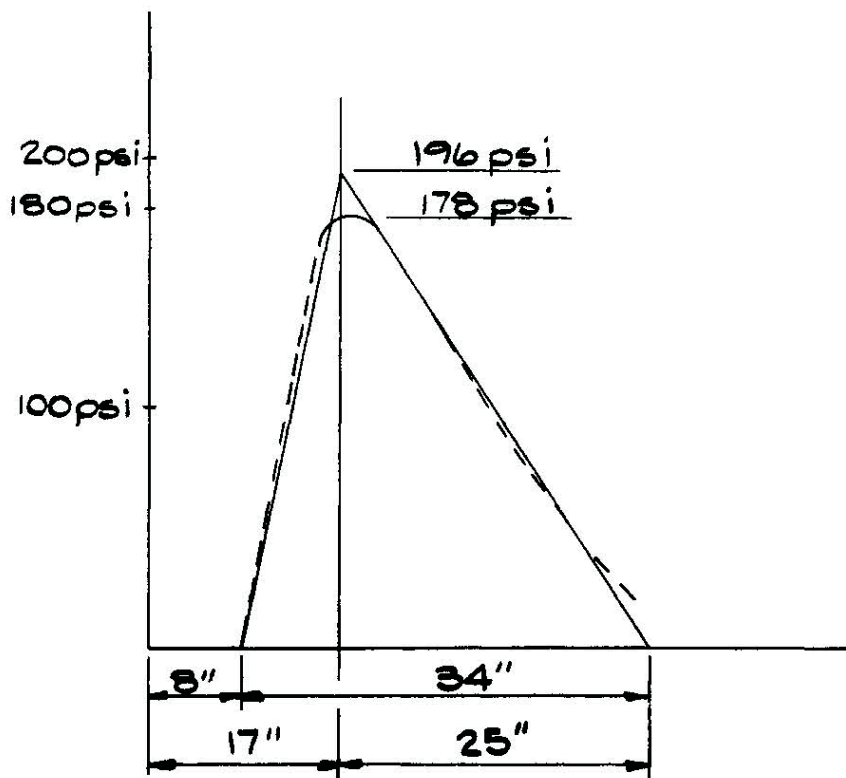
TORNADO LOADING



1.0D ± 0.05D + 1.15P



NOTE: VALUES SHOWN ARE MAX. AT EACH POINT. THEY MAY NOT OCCUR AT THE SAME TIME. MOMENTS SHOWN ON TENSION SIDE. (+) INDICATES TENSION, (-) INDICATES COMPRESSION. OPERATING TEMPERATURES CONSIDERED ONLY AT THE BOTTOM OF THE CYLINDER.



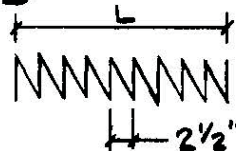
THE TENSILE FORCE PER UNIT WIDTH = $\frac{1}{2} \times 34 \times 196 = 3330^{\#}$
 AND FOR 37" WIDTH THE TOTAL FORCE = $3330 \times 37 = 123,000^{\#}$

$f_s = 40,000 \text{ psi}$ $\phi = 0.9$

USE #4 2-#4 TAKING $2 \times 0.2 \times 40,000 \times 0.9 = 14,400^{\#}$

IF #4 SPIRAL IS USED

PITCH = $2\frac{1}{2}"$ AND



$$L = \frac{123000 \times 2.5}{2 \times 14,400} \approx 11" \text{ REQ'D}$$

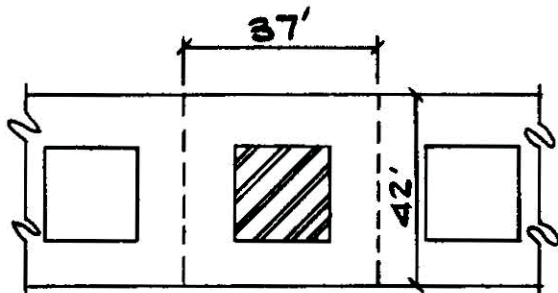
TO BE CONSERVATIVE PROVIDE $L = 20"$ MIN.

FOR SPALLING FORCE

THE TOTAL SPALLING FORCE = $9500^{\#}$

TRY: 2-#4 BARS $F = 0.9 \times 40,000 \times 0.4 = 14,400^{\#} (> 9,500^{\#})$

END BLOCK DESIGN FOR BURSTING FORCE



$$f_c = 5000 \text{ psi}$$

$$2a' = 18.5''$$

$$\frac{a}{d} = 0.44$$

$$2a = 42''$$



THE AVERAGE COMPRESSION

$$p = \frac{1150000}{37 \times 42} = 740 \text{ #/sq"} \quad \text{#/sq"} = \frac{\text{lb}}{\text{sq in}}$$

$$\text{FOR } \frac{a}{d} = 0.44$$

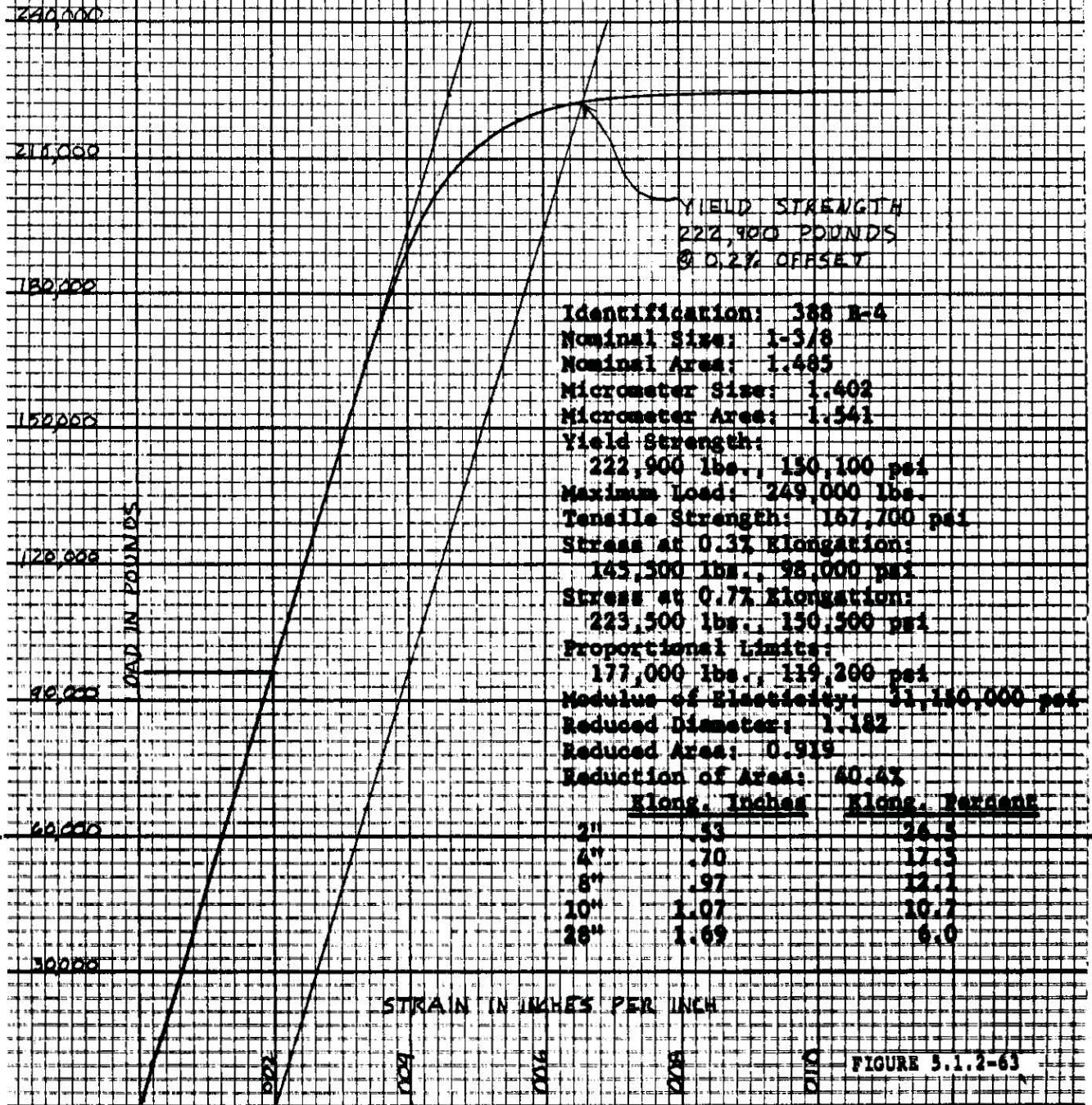
$$\text{MAX TENSION \& PRESS} = 0.24 - p = 178 \text{ #/sq"} \quad \text{#/sq"} = \frac{\text{lb}}{\text{sq in}}$$

THE MAXIMUM VALUE OCCURS AT $0.8a = 0.8 \times 21 = 17''$ FROM THE BEARING SURFACE.

THE ZERO STRESS POINT OCCURS AT $0.87a = 0.87 \times 21 \approx 18''$ FROM THE BEARING SURFACE

388 B-4
330,000 FULL
LOW MAG

PITTSBURGH TESTING LABORATORY
PG-20104, Lab. No. 673574-A
Stress Strain Curves made for
Ebasco Services, Inc.



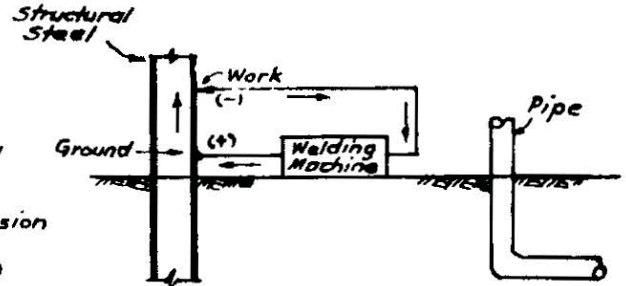
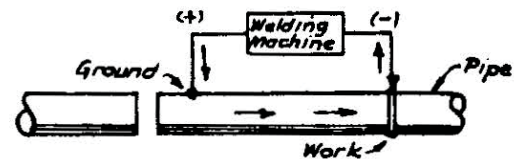
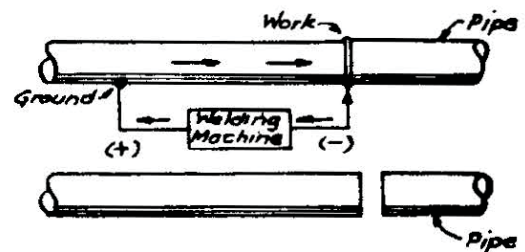
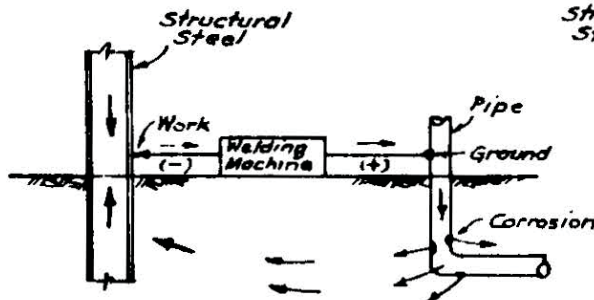
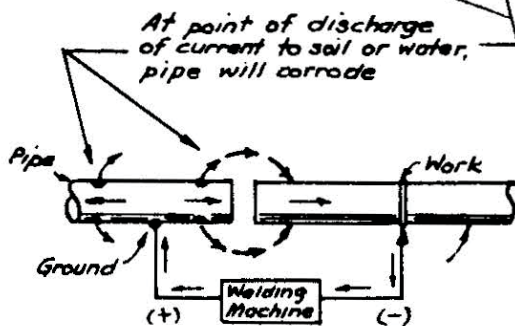
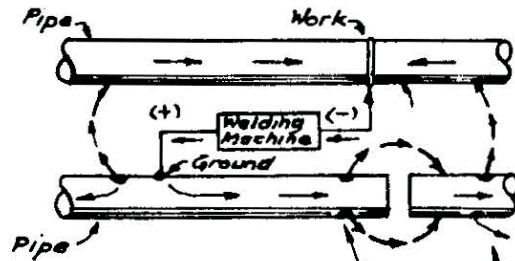
INCORRECT METHODS Of Grounding Welding Machines

Where welding-machine ground is connected to a structure that is not metallically continuous with work, corrosion will occur. If the ground is positive as shown, the grounded structure will corrode; with reverse polarity the structure being welded will corrode.

CORRECT METHODS Of Grounding Welding Machines to avoid corrosion

Where welding-machine ground is connected to a structure that is metallically continuous with work, welding current follows a continuous metallic path and corrosion is avoided.

STRAIGHT POLARITY WELDING



NOTE:

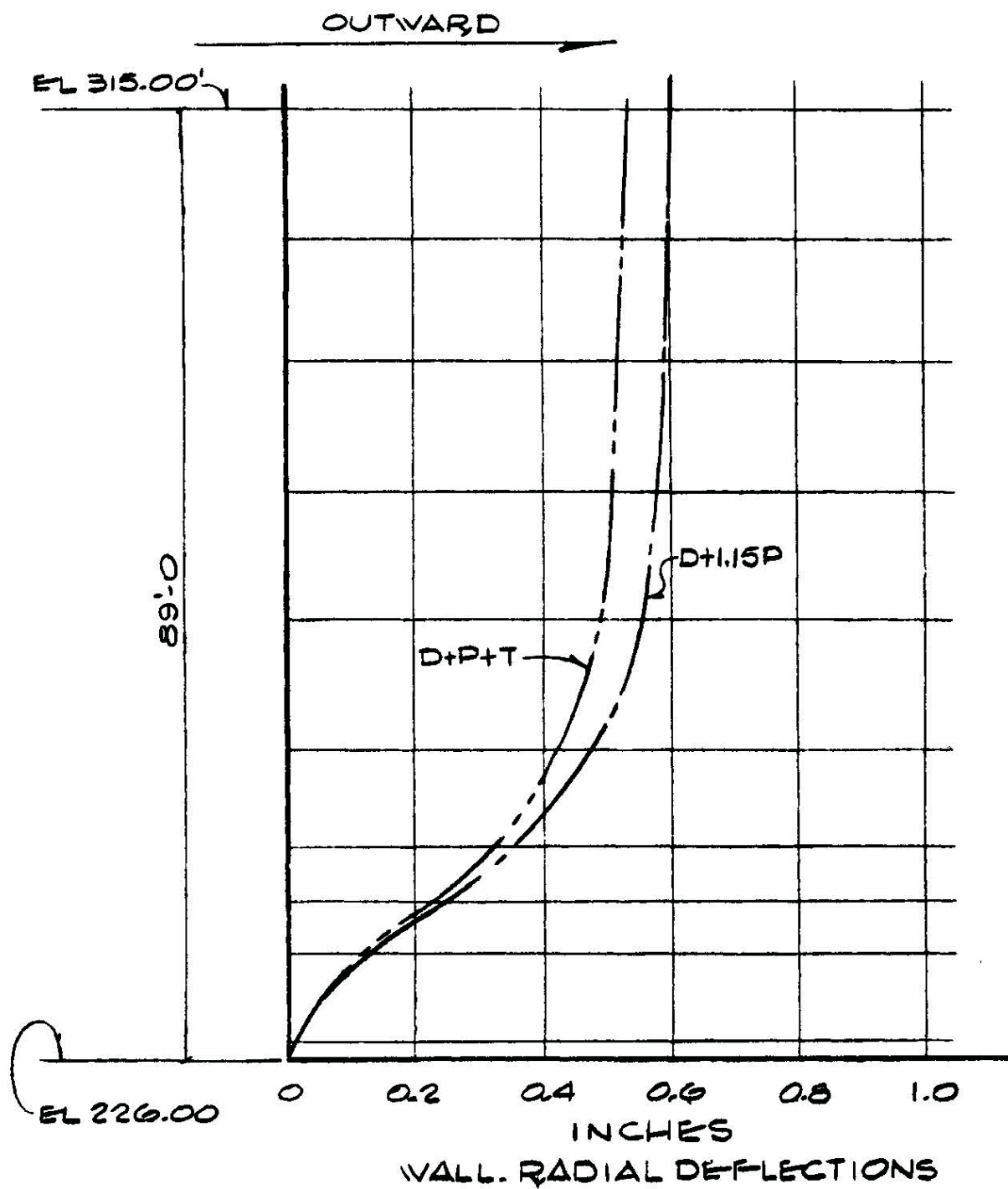
This guide applies when pipes are in contact with earth or water, or are connected to structures in contact with earth or water, and welding is performed with d-c electric machines.

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UNIT 2

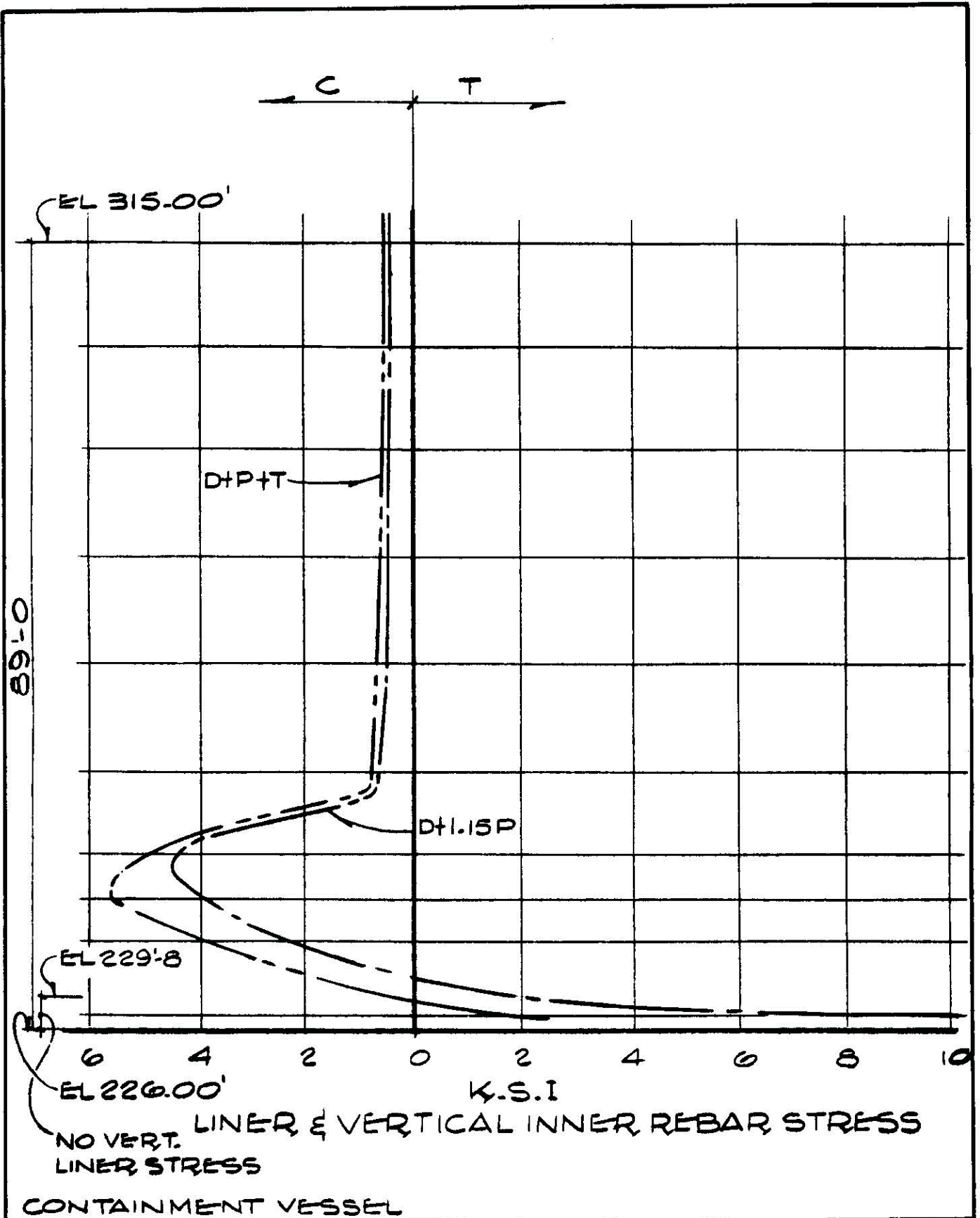
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ELECTRIC WELDING
GROUNDING OF DC WELDING MACHINES
TO PREVENT CORROSION

FIGURE
3.8.1 - 43



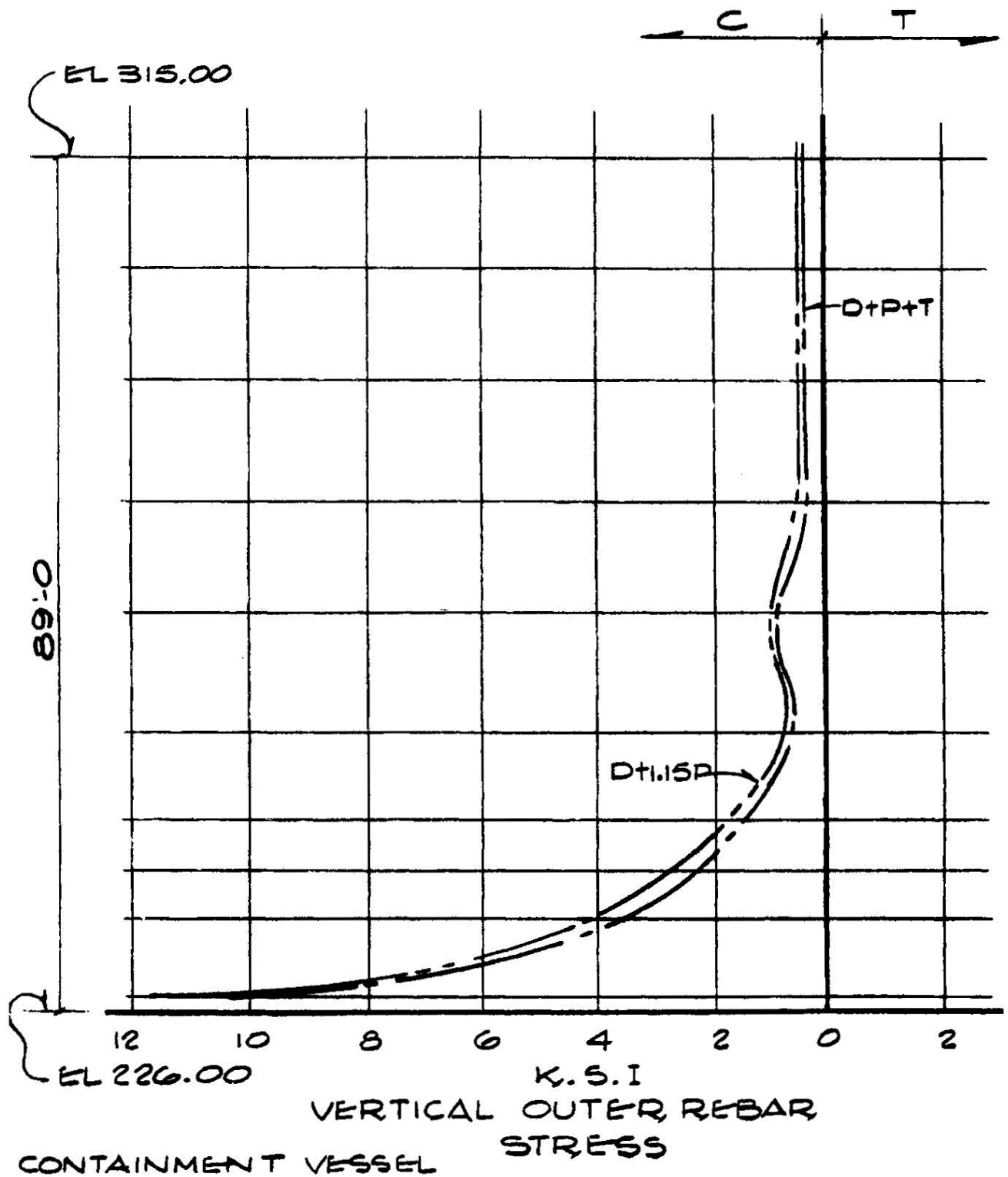
CONTAINMENT VESSEL



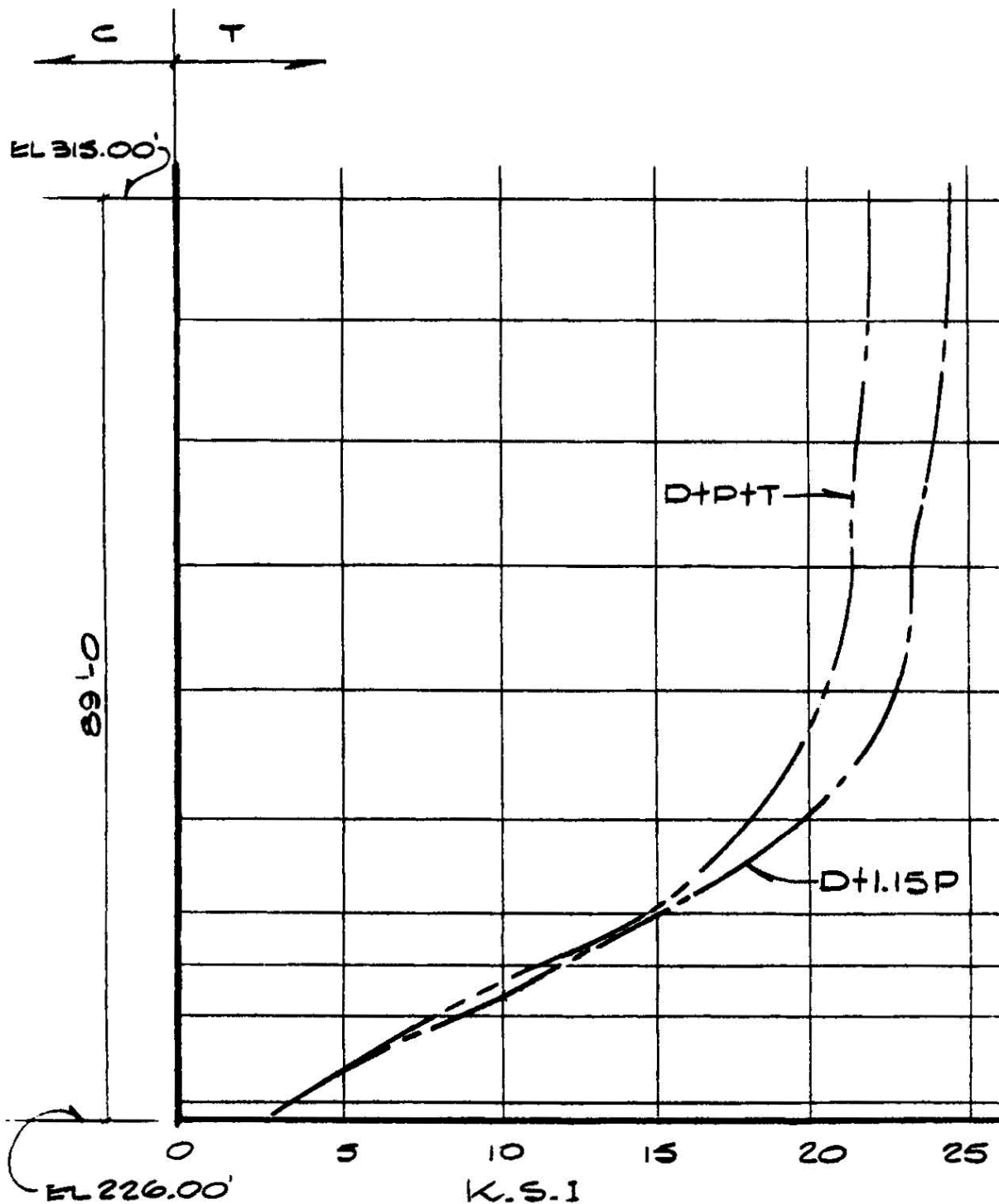
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CONTAINMENT VESSEL LINER AND
VERTICAL INNER REBAR STRESS

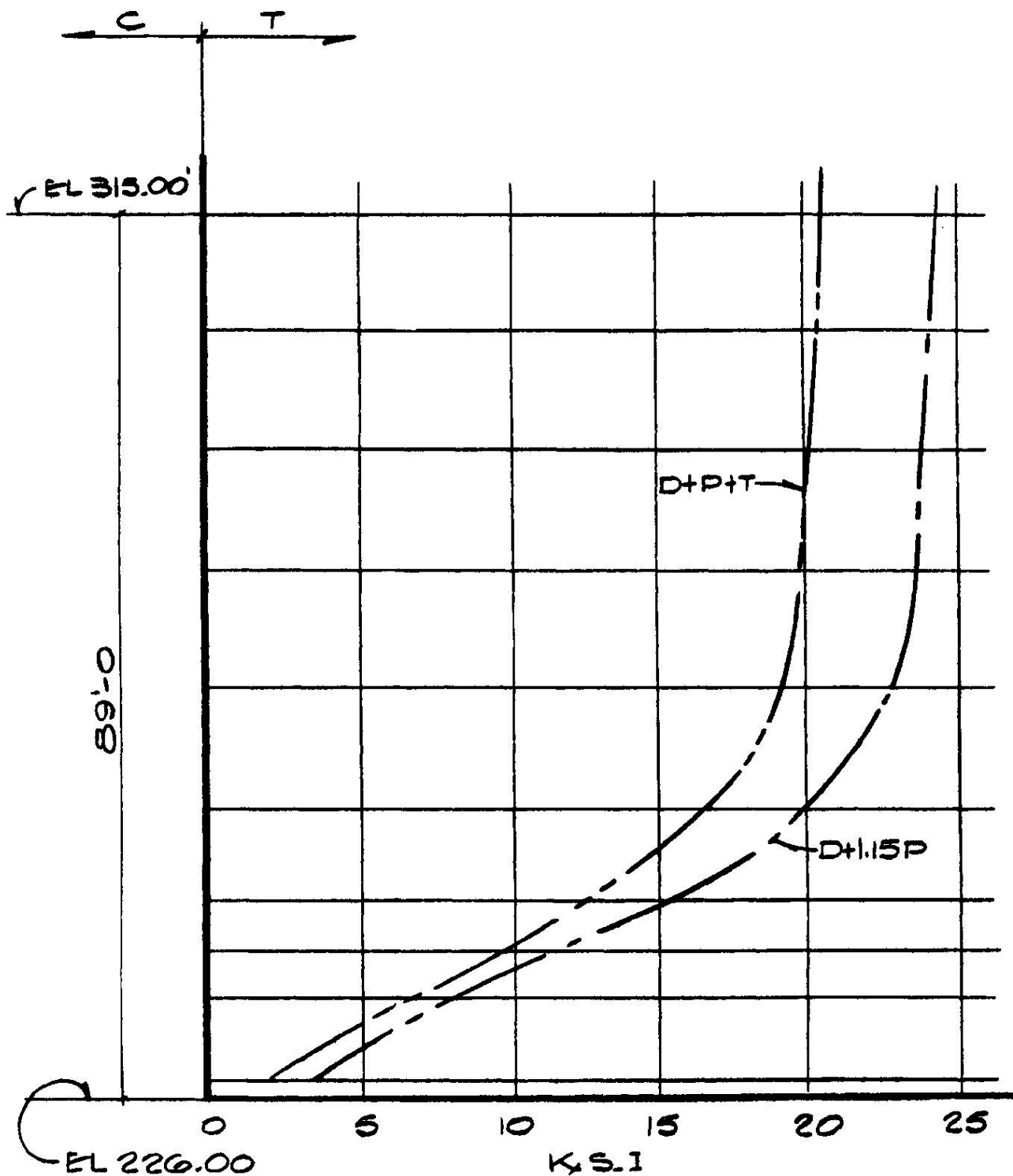
FIGURE
3.8.1 - 45



<p>H. B. ROBINSON UNIT 2</p> <p>Carolina Power & Light Company</p> <p>UPDATED FINAL SAFETY ANALYSIS REPORT</p>	<p>CONTAINMENT VESSEL VERTICAL OUTER REBAR STRESS</p>	<p>FIGURE</p> <p>3.8.1 - 46</p>
----------------------------------------------------------------------------------------------------------------------------	-----------------------------------------------------------	---------------------------------



OUTER HOOP REBAR STRESS
CONTAINMENT VESSEL



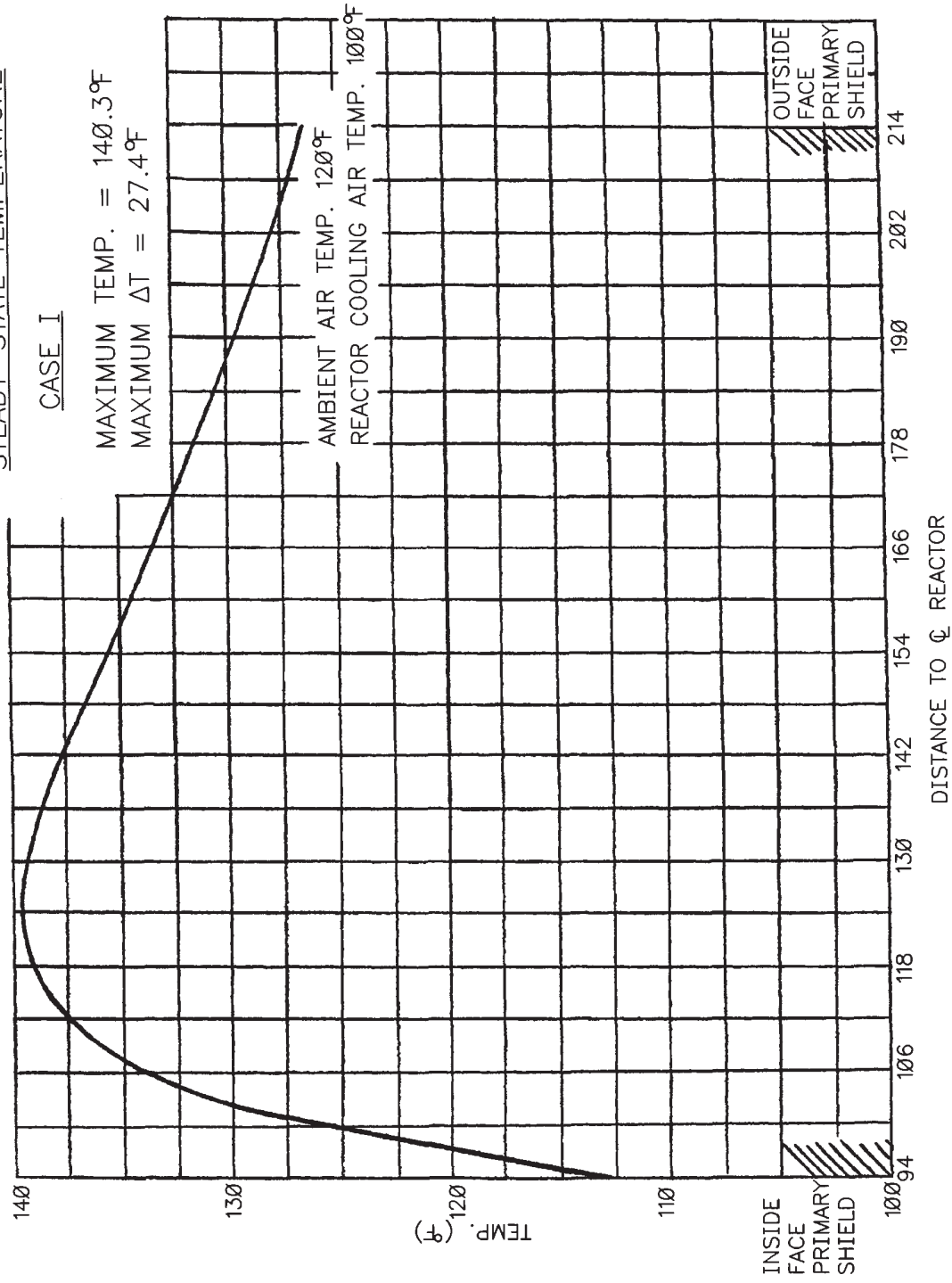
LINER & INNER HOOP REBAR STRESS
CONTAINMENT VESSEL

STEADY-STATE TEMPERATURE

CASE I

MAXIMUM TEMP. = 140.3°F

MAXIMUM ΔT = 27.4°F



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PRIMARY SHIELD THERMAL GRADIENT

STEADY STATE CASE I

REVISION 19

FIGURE 3.8.3-1

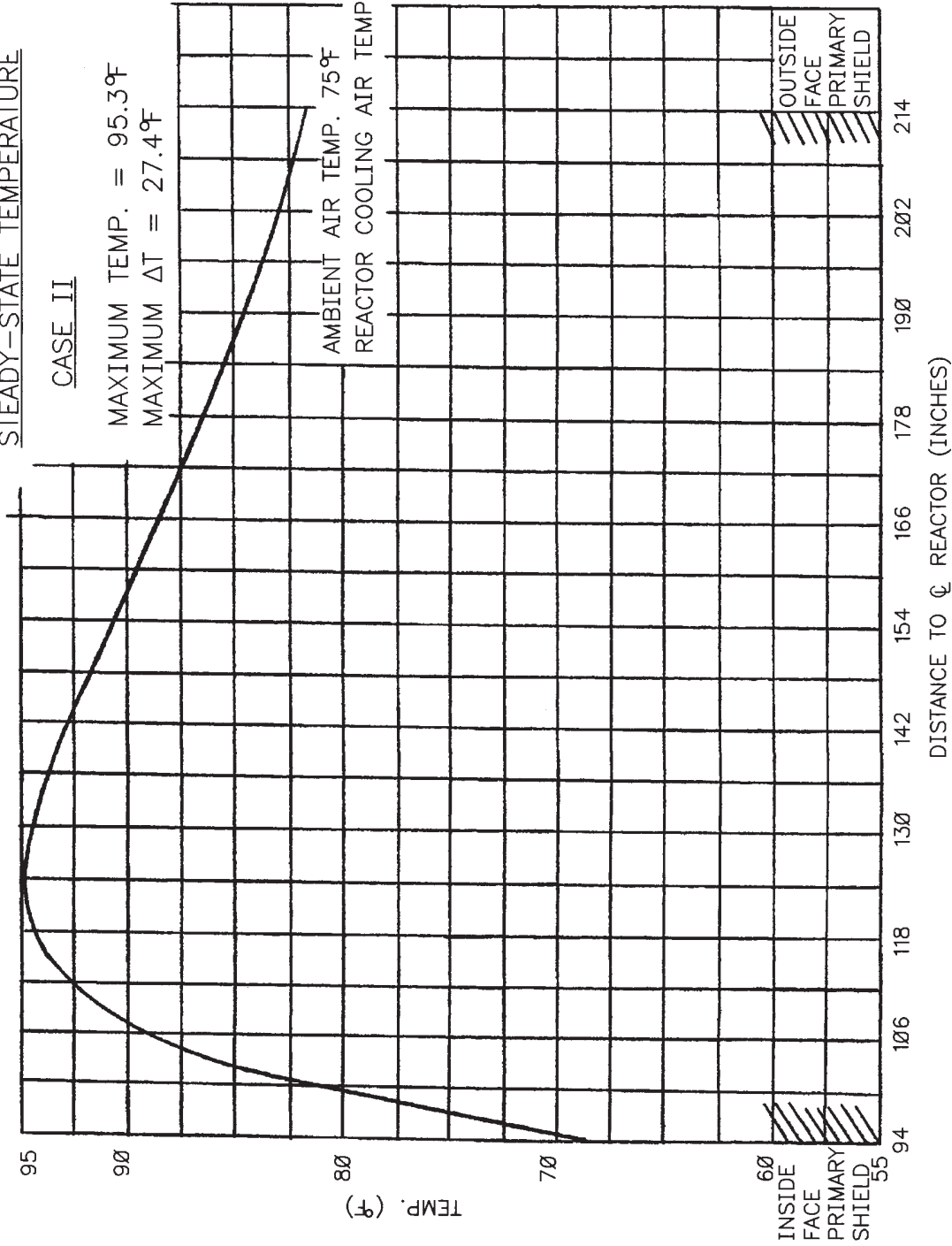
STEADY-STATE TEMPERATURE

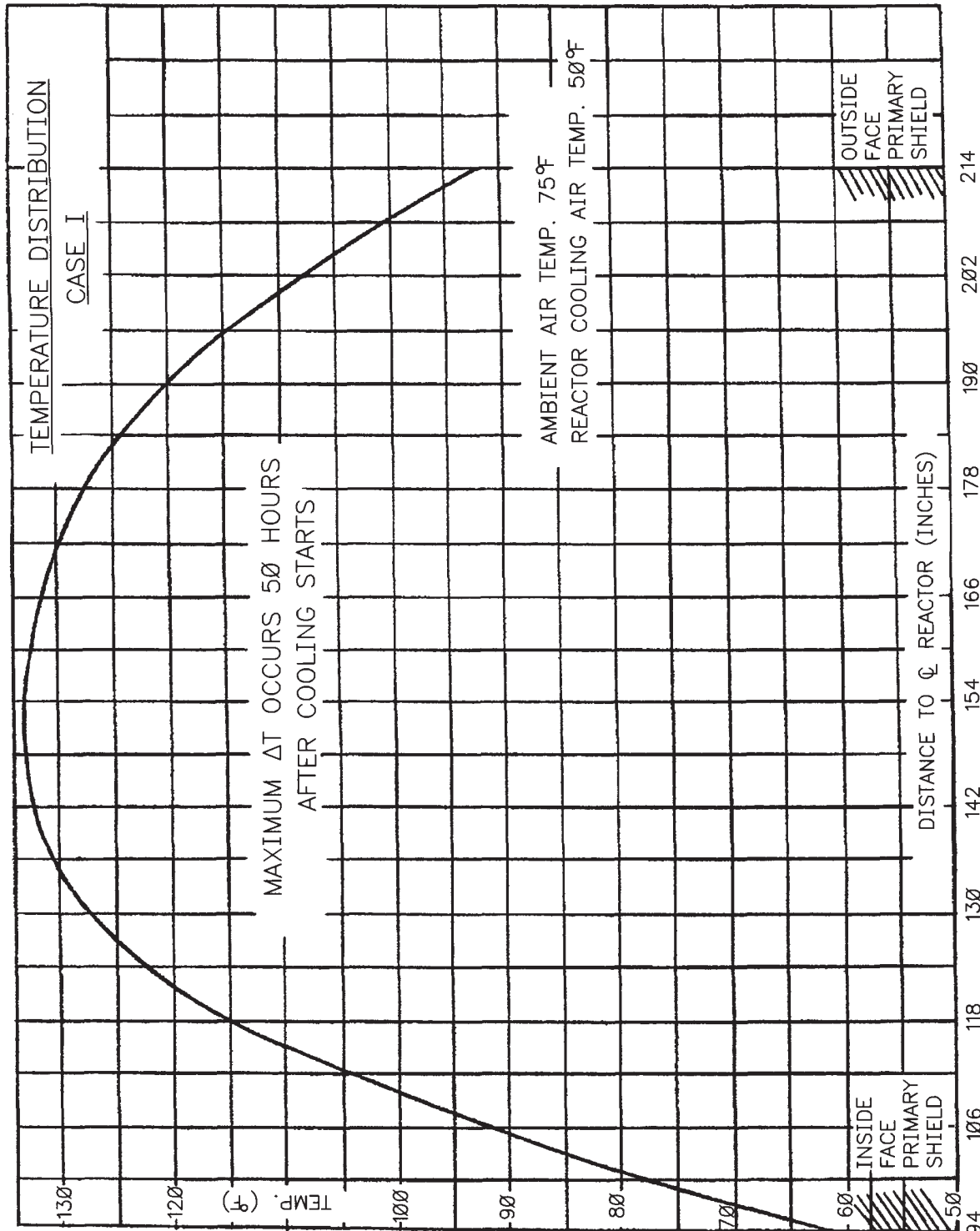
CASE II

MAXIMUM TEMP. = 95.3°F
MAXIMUM ΔT = 27.4°F

AMBIENT AIR TEMP. 75°F

REACTOR COOLING AIR TEMP. 55°F





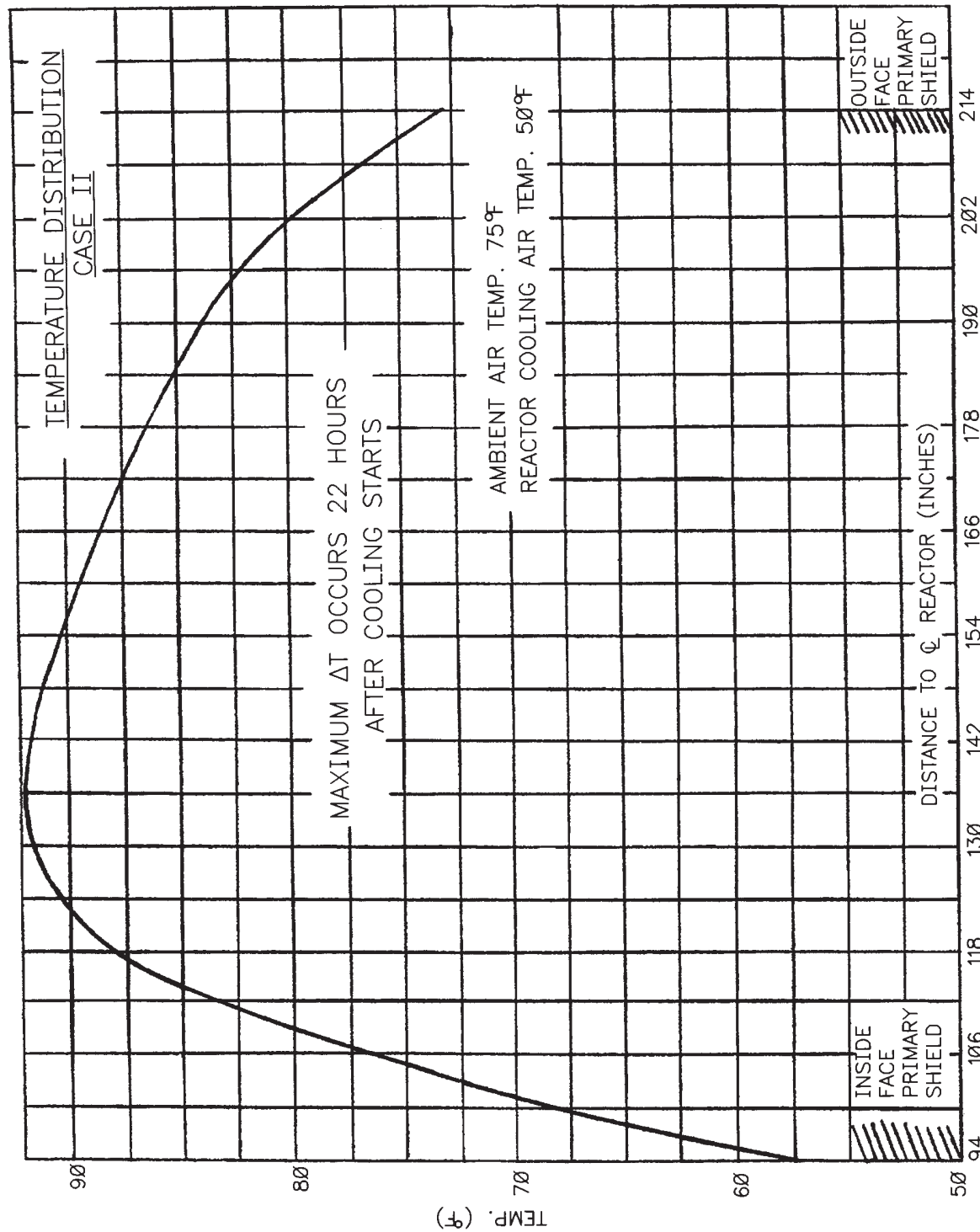
NOTE: POWER UPRATE TO 2,339 MWT RESULTED IN LESS THAN A 1°F INCREASE IN THE TEMPERATURE PROFILE AND MAXIMUM ΔT .

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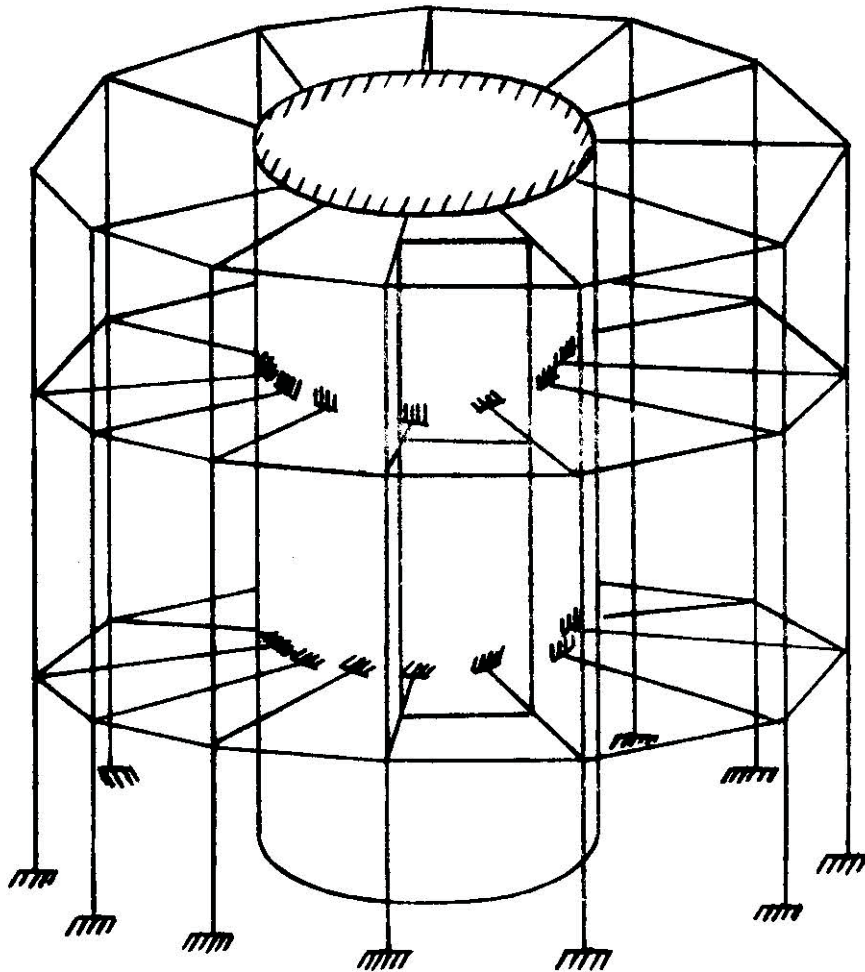
PRIMARY SHIELD TEMPERATURE
DISTRIBUTION CASE I

REVISION 19

FIGURE 3.8.3-3

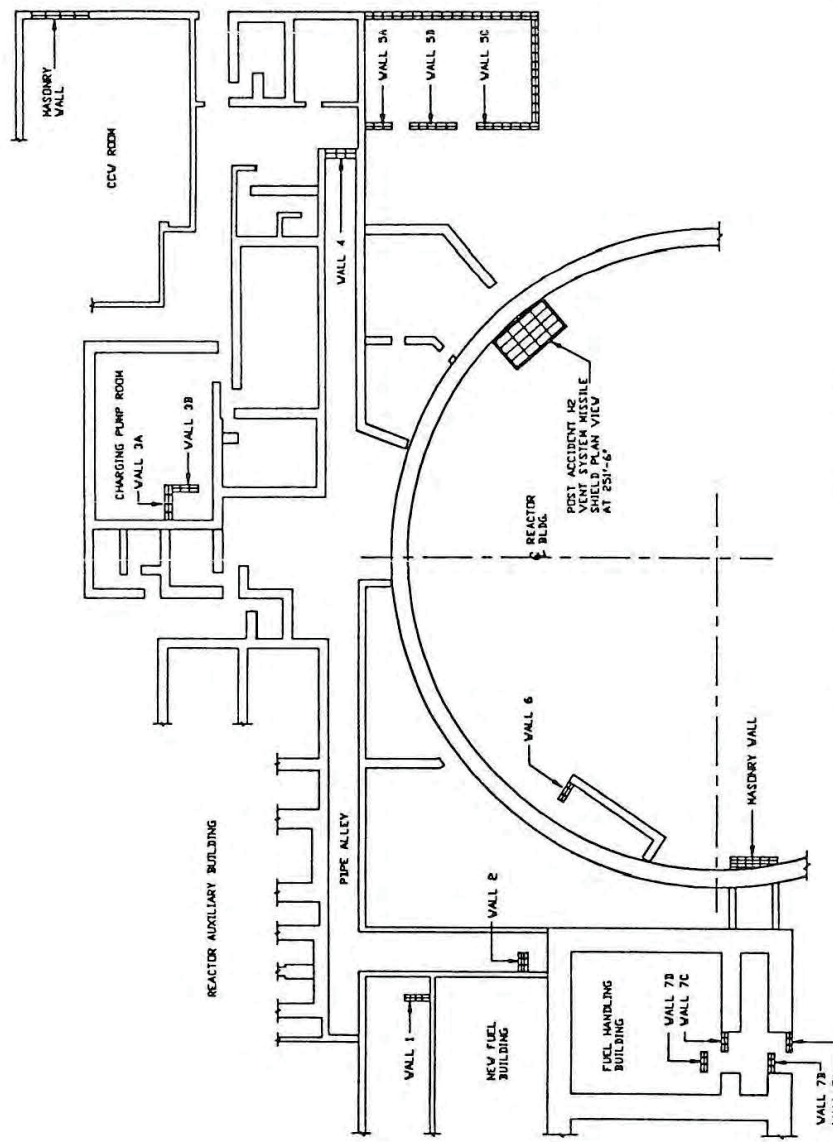


NOTE: POWER UPRATE TO 2,339 MWT RESULTED IN LESS THAN A 1°F INCREASE IN THE TEMPERATURE PROFILE AND MAXIMUM ΔT .

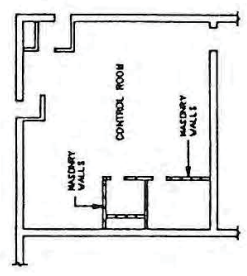


SPACE FRAME
IDEALIZED MODEL

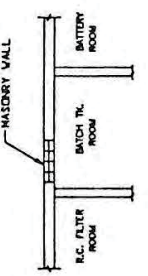
3.8.4 - 1



PLAN VIEW AT EL. 226'-0" (U.D.N.)



PLAN VIEW
R.A.B. ELEV. 254'-0"



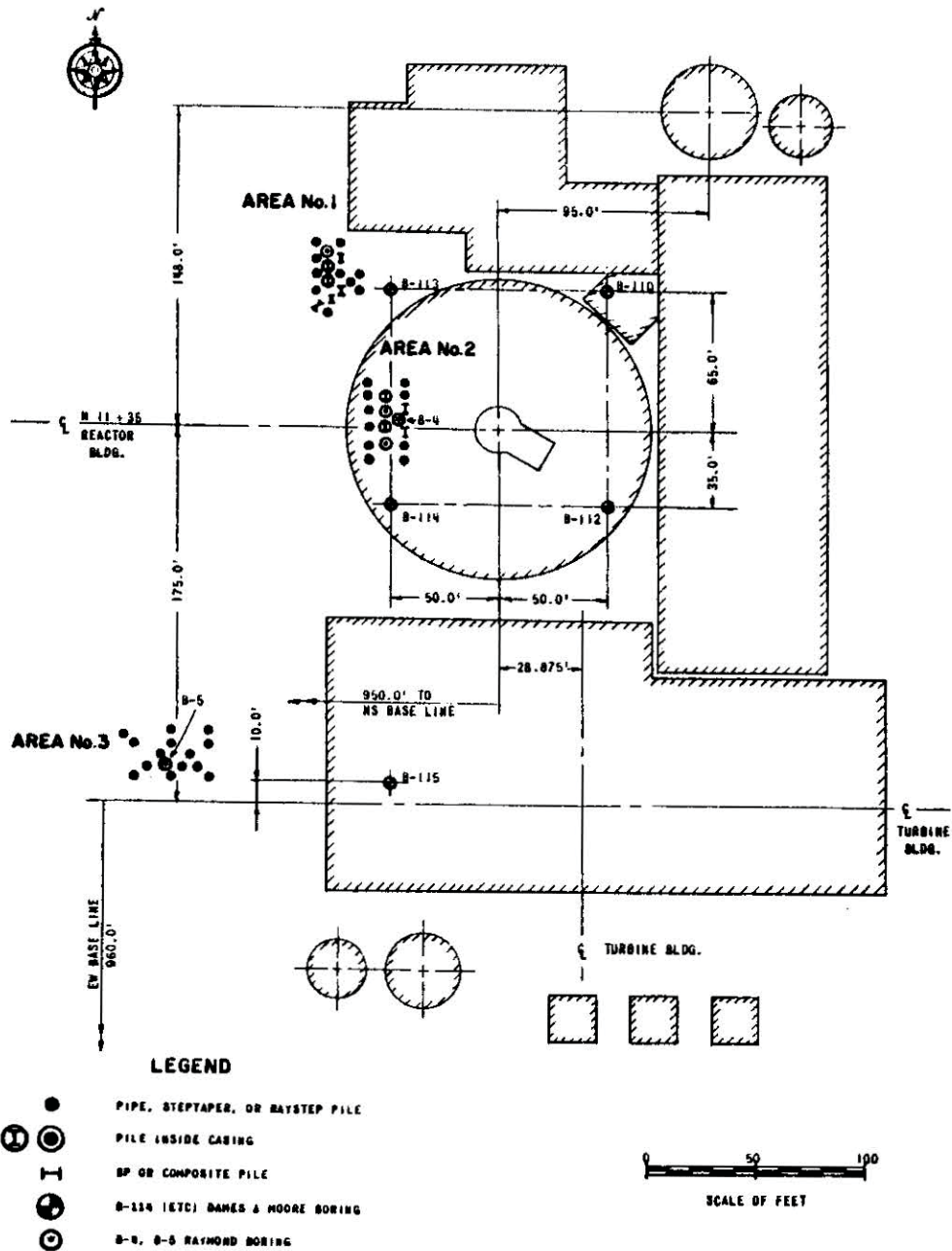
PLAN VIEW
R.A.B. ELEV. 246'-0"

AMENDMENT NO. 11

H.B. Robinson Unit 2 Carolina Power & Light Company UPDATED FINAL SAFETY ANALYSIS REPORT	I. E. BULLETIN 80-11 BLOCK WALLS - LOCATION PLAN
---------------------------------------------------------------------------------------------------	-----------------------------------------------------

FIGURE 3.8.4-2

KEY PLAN OF TEST PILE AREAS

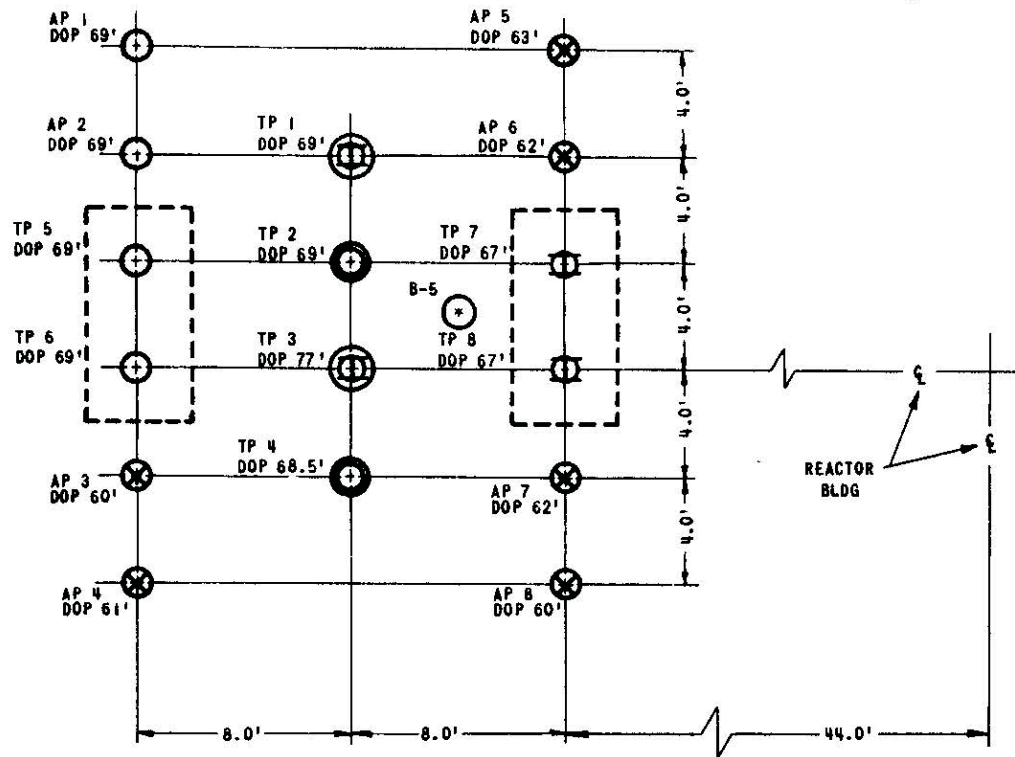


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KEY PLAN OF TEST PILE AREA

FIGURE
3.8.5 - 1

AREA No.2



LEGEND

- ⊗ 12"Ø - 7/16" THICK PIPE PILE NOT MANDREL DRIVEN
- 12"Ø - 7/16" THICK PIPE PILE MANDREL DRIVEN
- ⊞ COMPOSITE PILE MANDREL DRIVEN
- ⊞ } PILES DRIVEN INSIDE CASING
- } PILES DRIVEN INSIDE CASING
- DOP = DEPTH OF PENETRATION
- ⊙ B-5 RAYMOND BORING

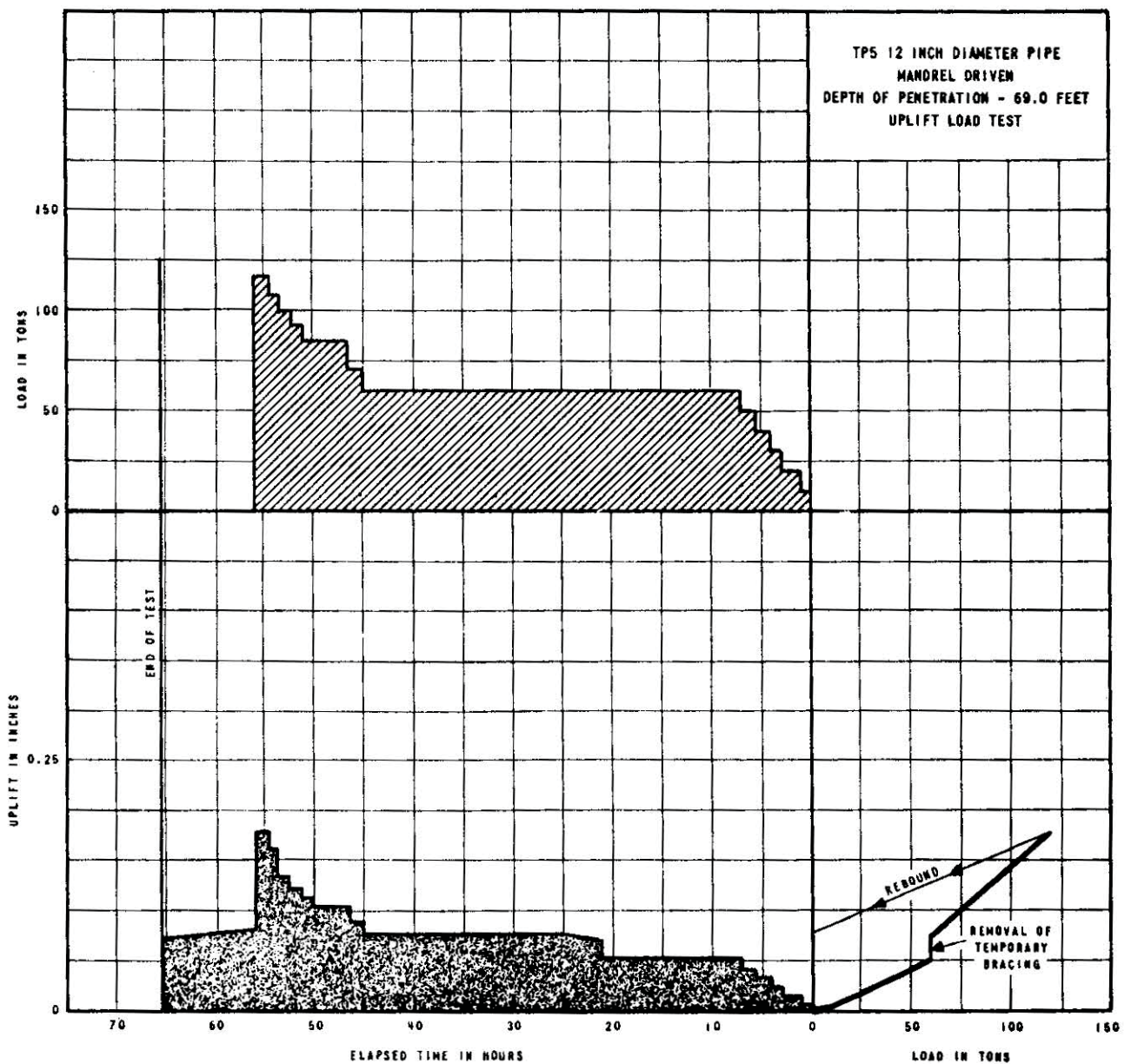
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PILE AREA NO. 2

FIGURE

3.8.5 - 2

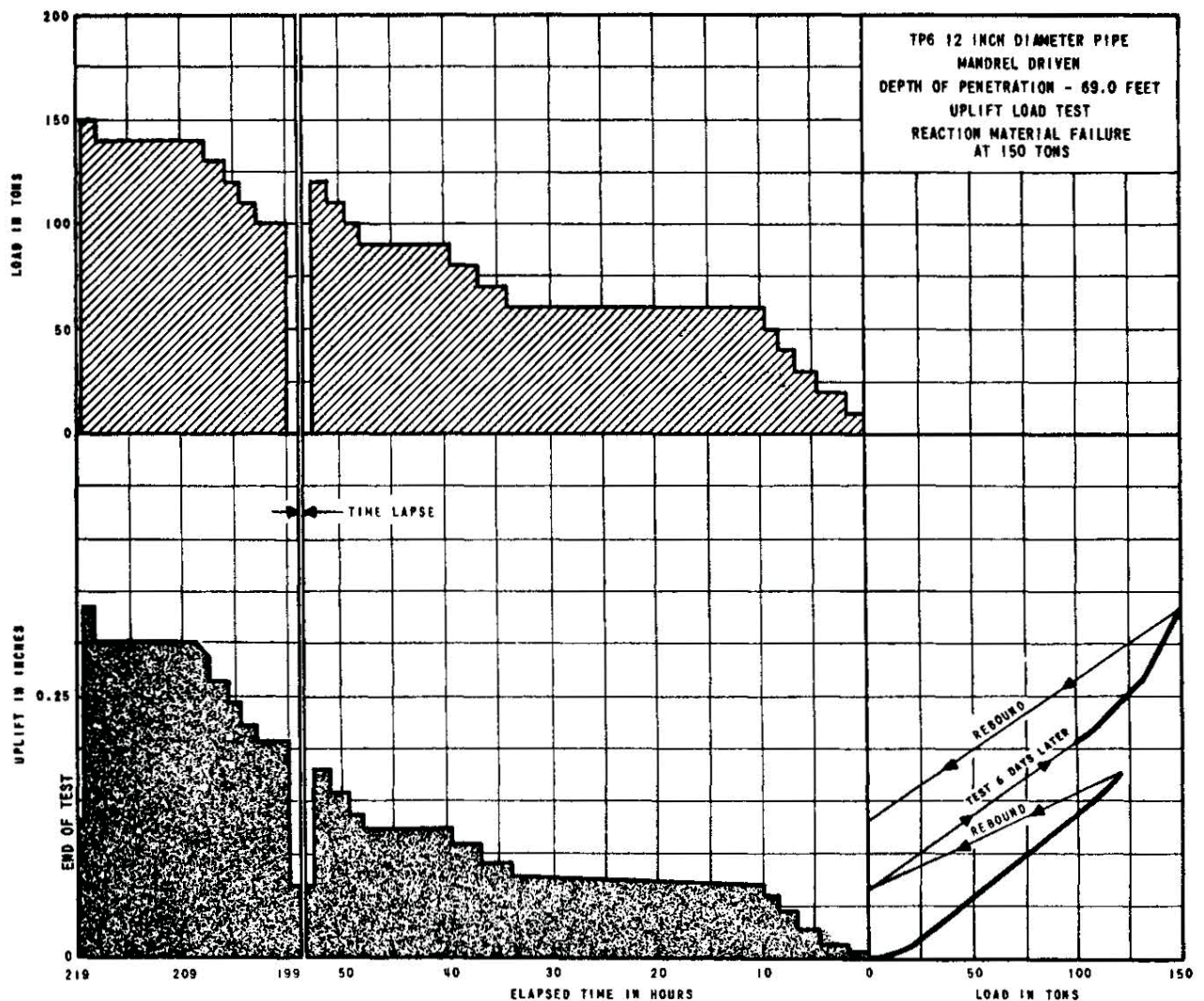


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TP 5 12 INCH DIAMETER PIPE
UPLIFT LOAD TEST

FIGURE

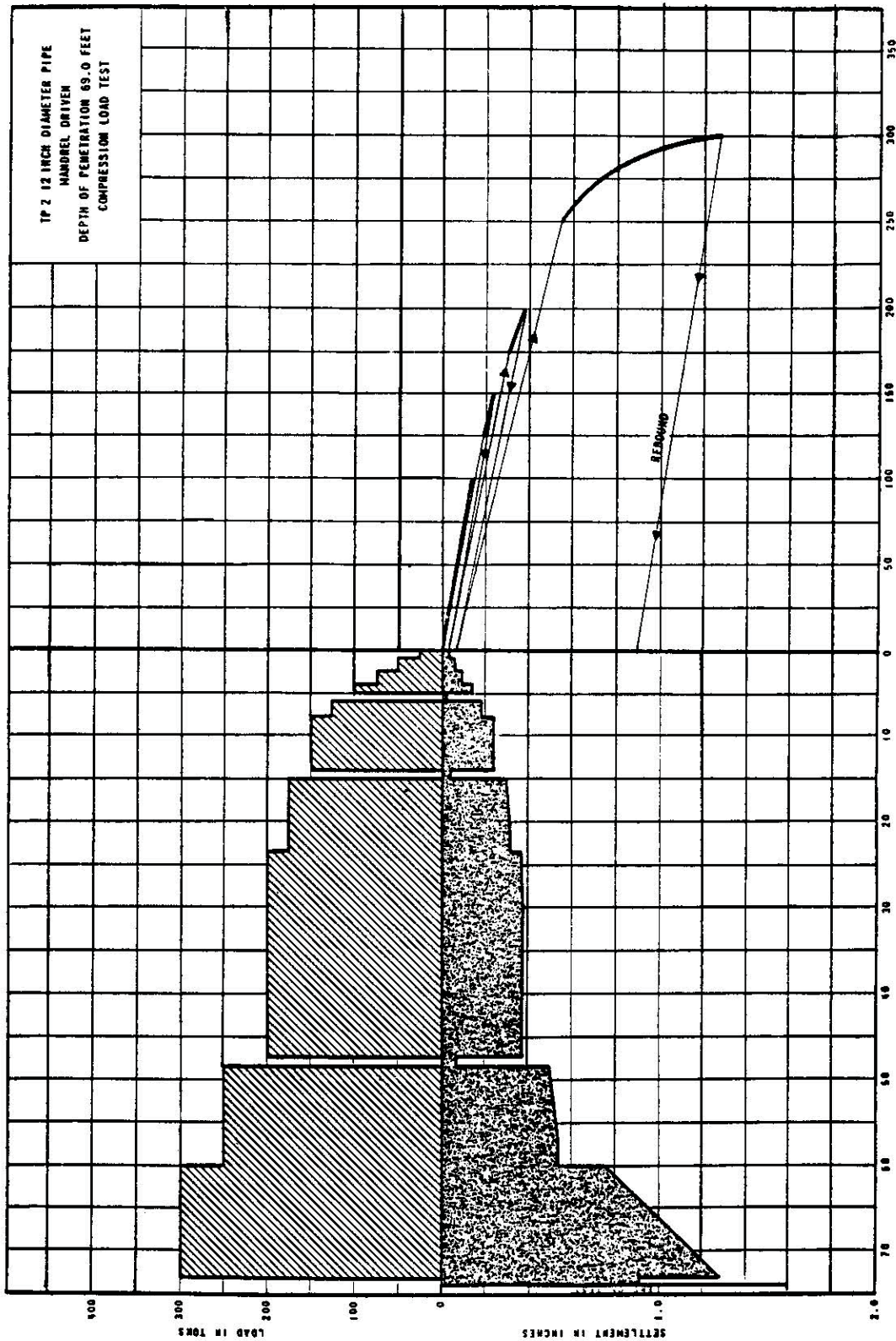
3.8.5 - 3



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TP 6 12 INCH DIAMETER PIPE
UPLIFT LOAD TEST

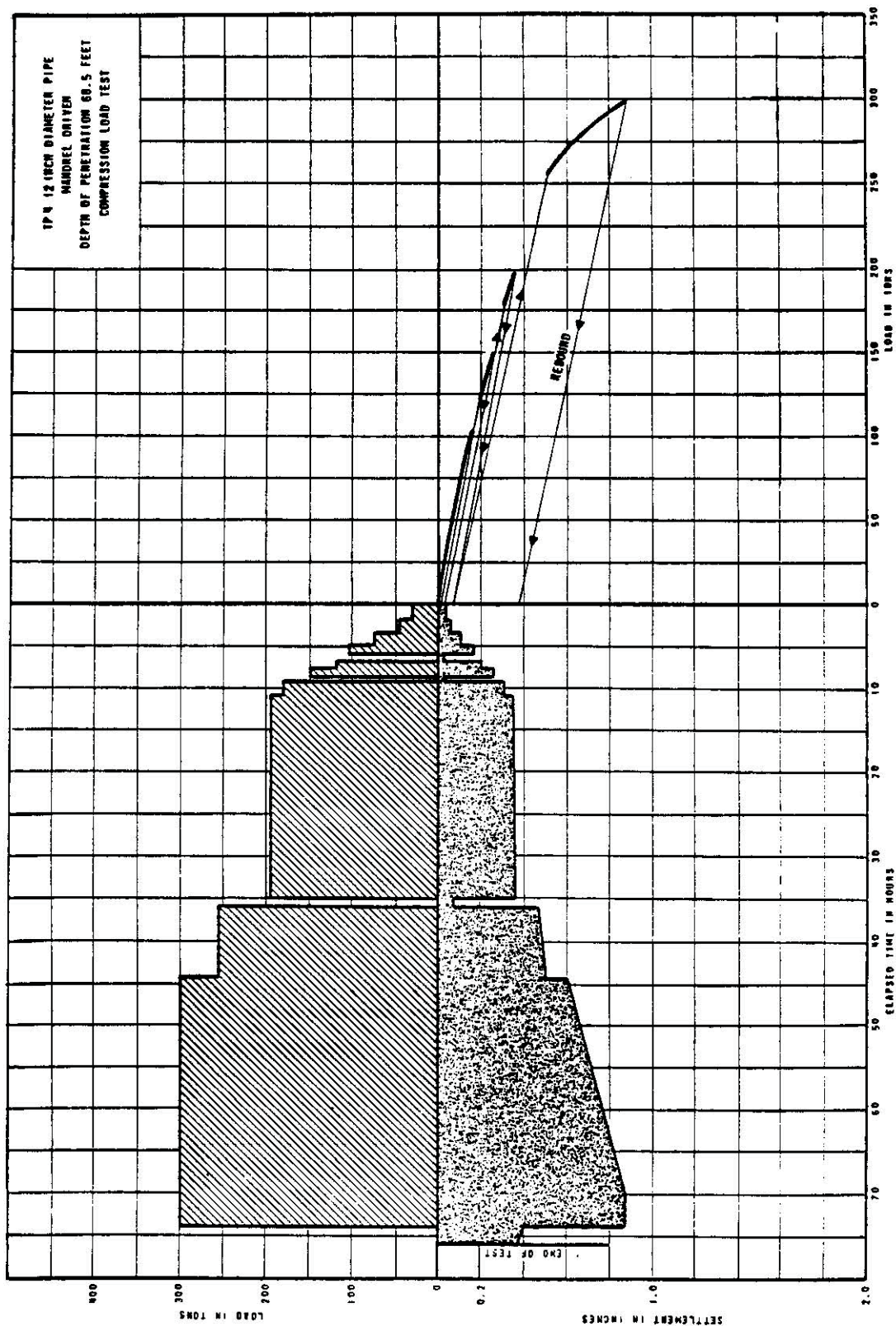
FIGURE
3.8.5 - 4



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TP 2 12 INCH DIAMETER PIPE
COMPRESSION LOAD TEST

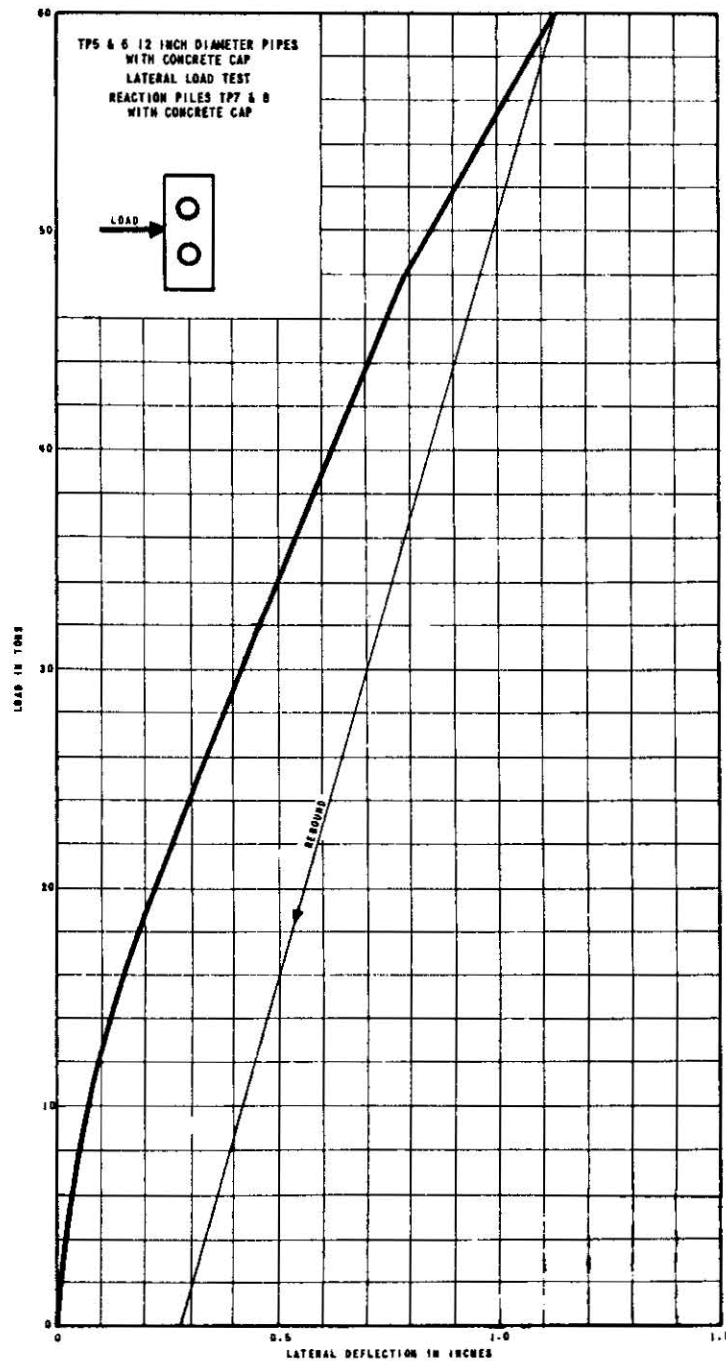
FIGURE
3.8.5 - 5



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TP 4 12 INCH DIAMETER PIPE
COMPRESSION LOAD TEST

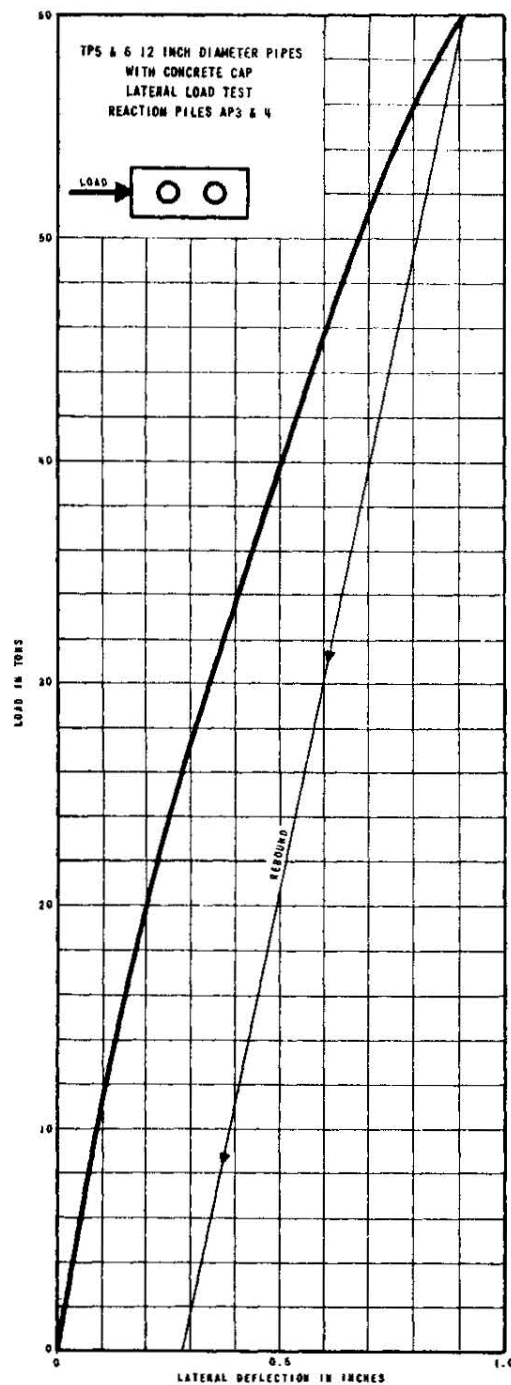
FIGURE
3.8.5 - 6



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TP 5 & 6 12 INCH DIAMETER PIPES
LATERAL LOAD TEST

FIGURE
3.8.5 - 7

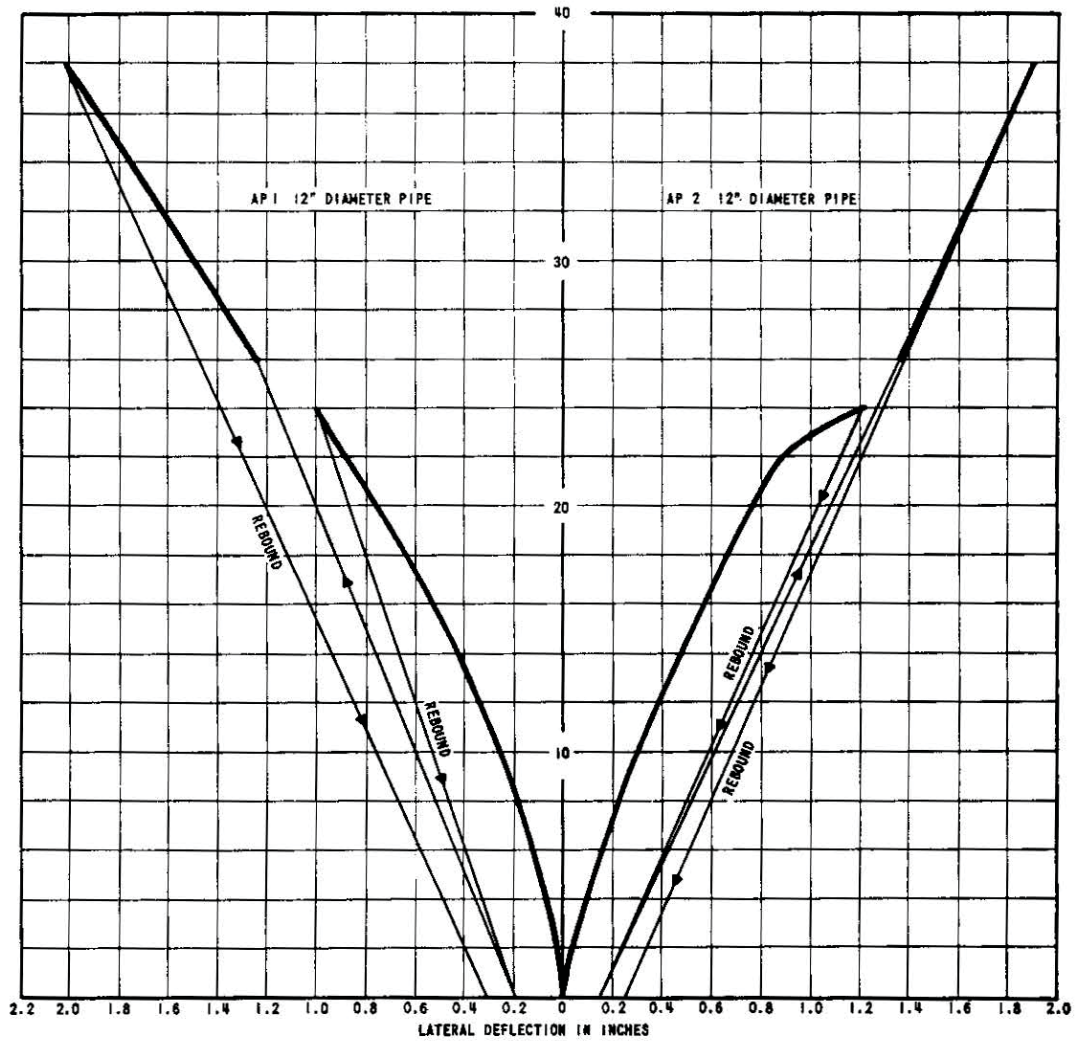


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TP 5 & 6 12 INCH DIAMETER PIPES
LATERAL LOAD TEST

FIGURE
3.8.5 - 8

LATERAL LOAD TEST
JACKING AGAINST EACH OTHER

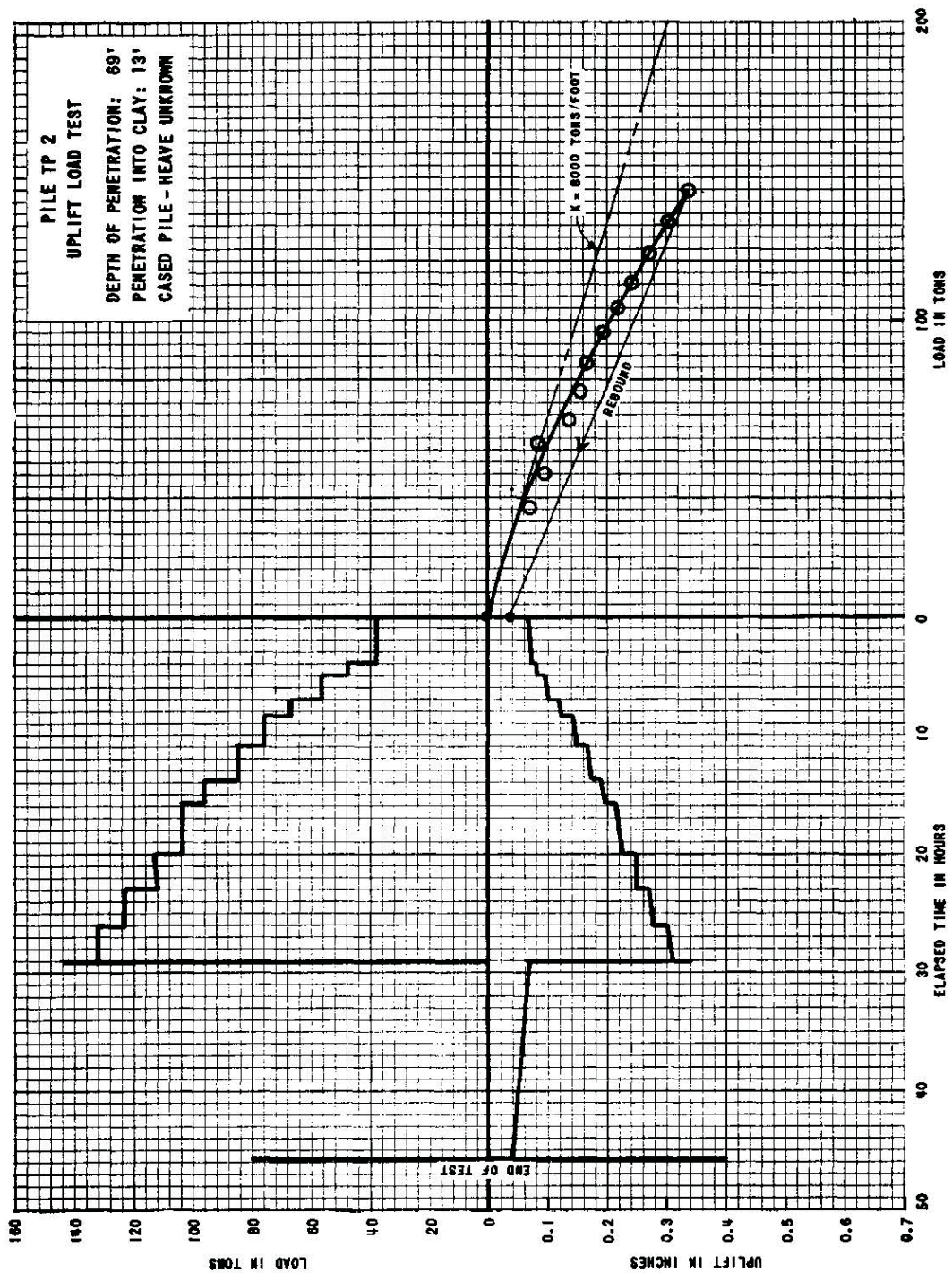


NOTE:
TEST HALTED AT 24 TONS UNTIL
EXCAVATION ADJACENT TO TEST
AREA WAS COMPLETED.

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LATERAL LOAD TEST
JACKING AGAINST EACH OTHER

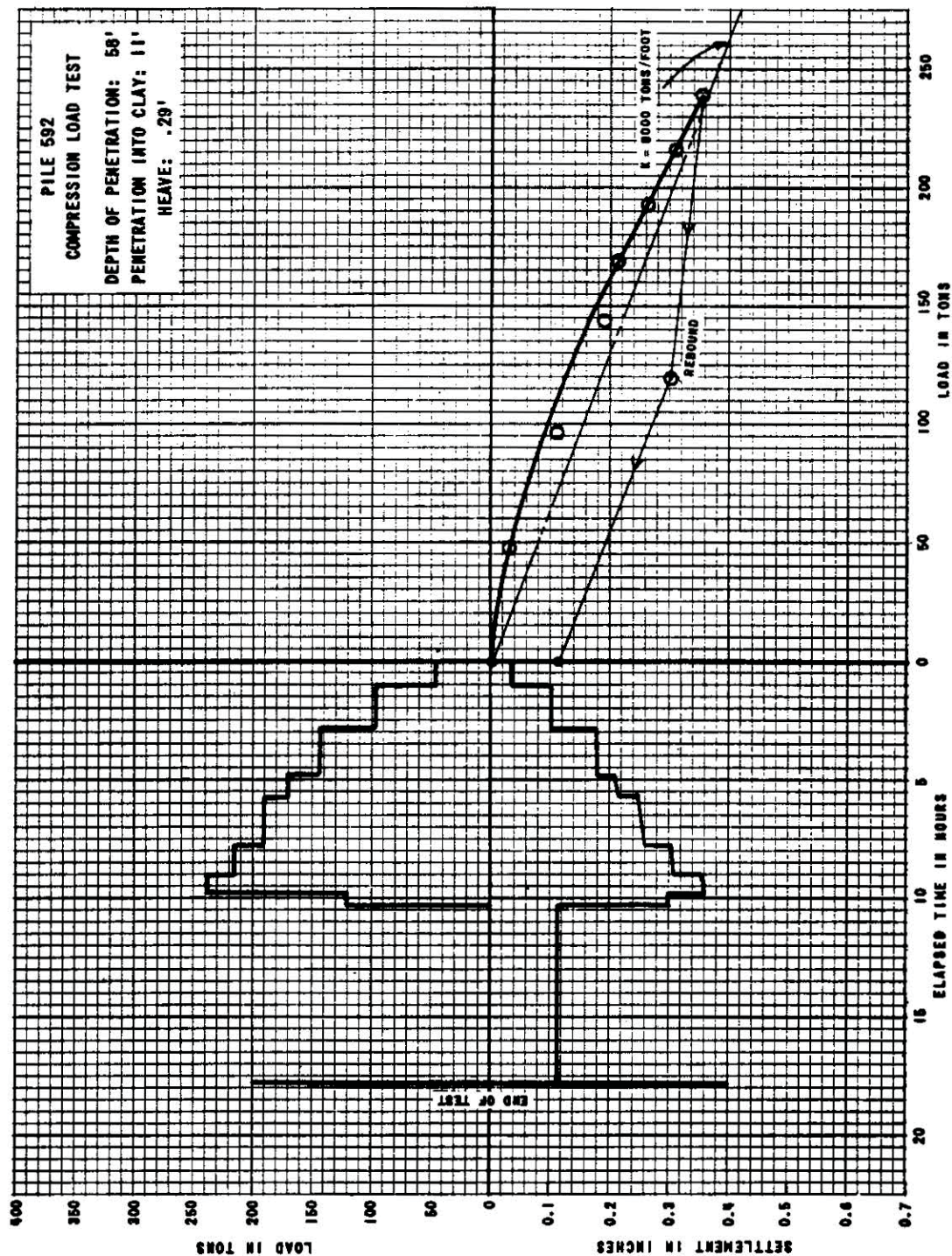
FIGURE
3.8.5 - 9



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PILE TP 2
UPLIFT LOAD TEST

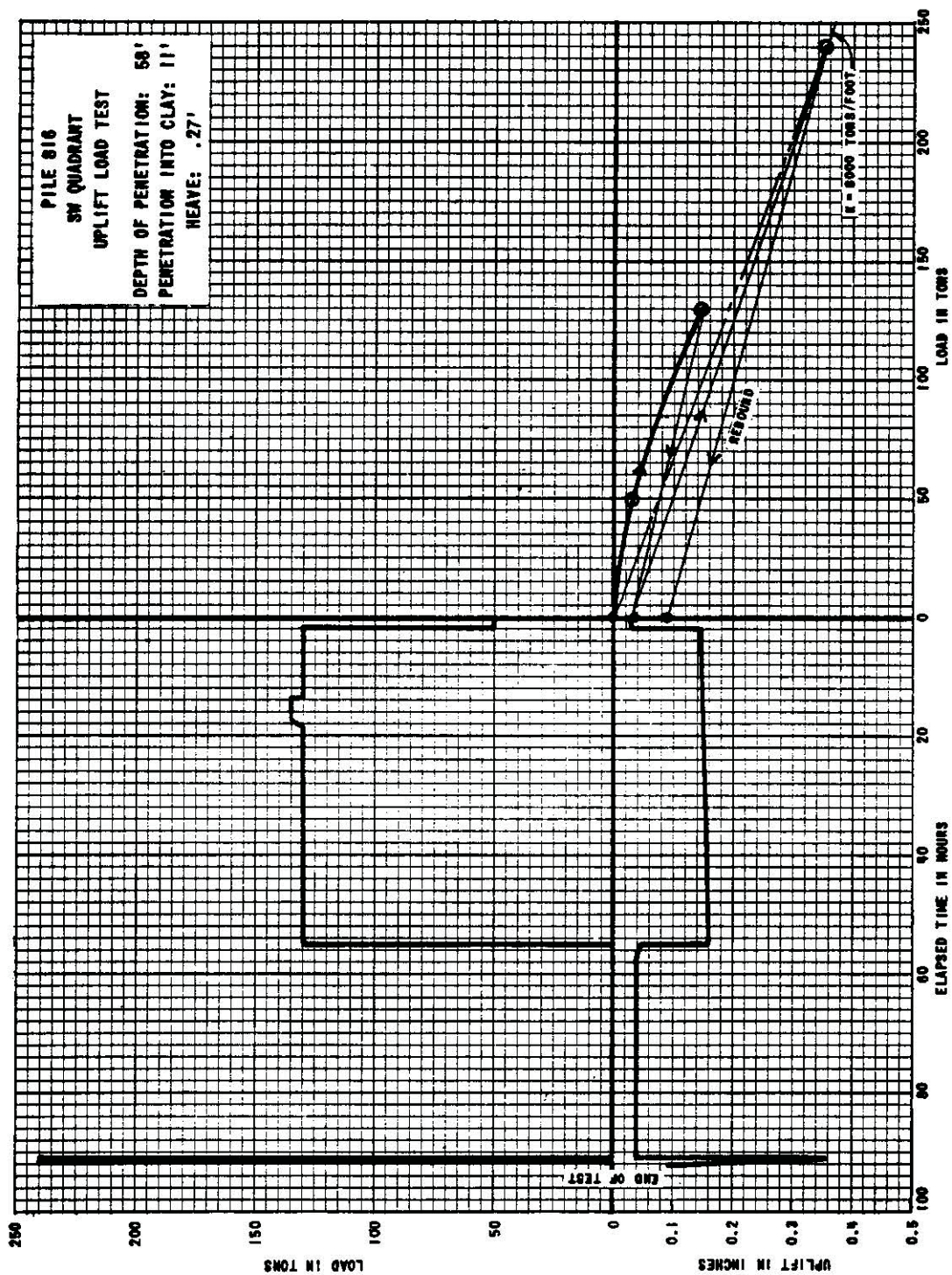
FIGURE
3.8.5 - 10



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PILE 592
COMPRESSION LOAD TEST

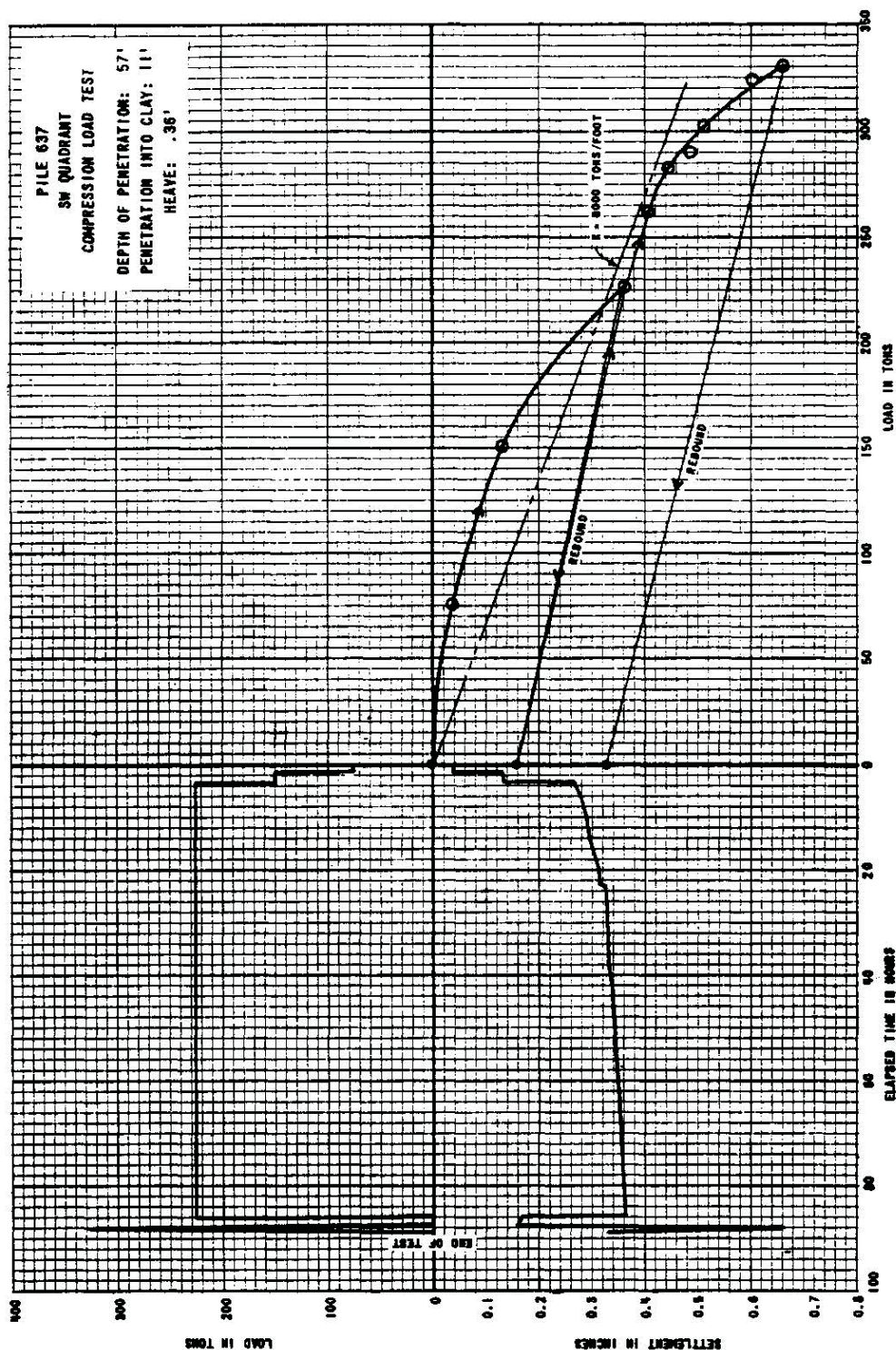
FIGURE
3.8.5 - 11



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PILE 816
UPLIFT LOAD TEST

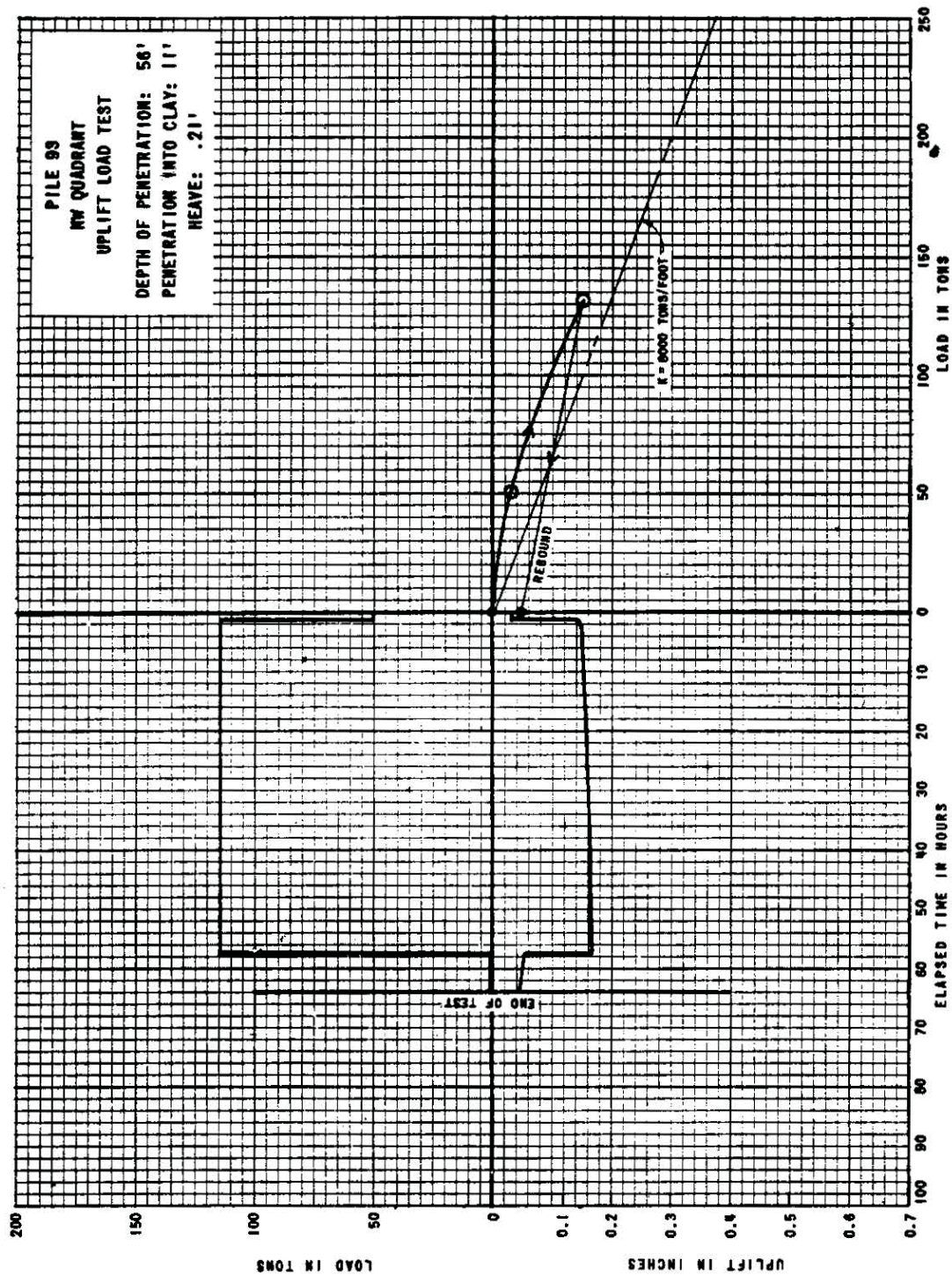
FIGURE
3.8.5 - 12



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PILE 637
COMPRESSION LOAD TEST

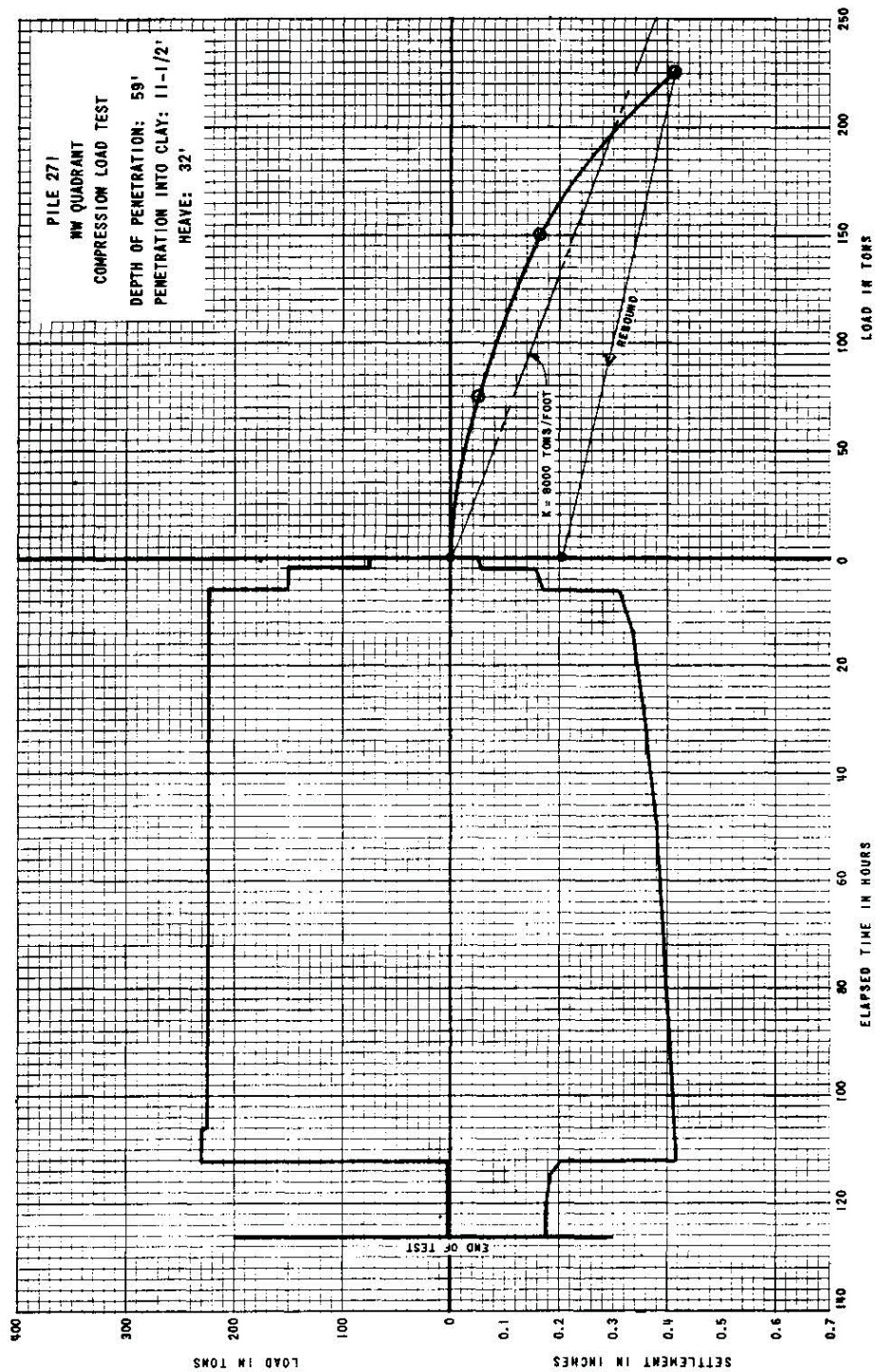
FIGURE
3.8.5 - 13



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PILE 93
UPLIFT LOAD TEST

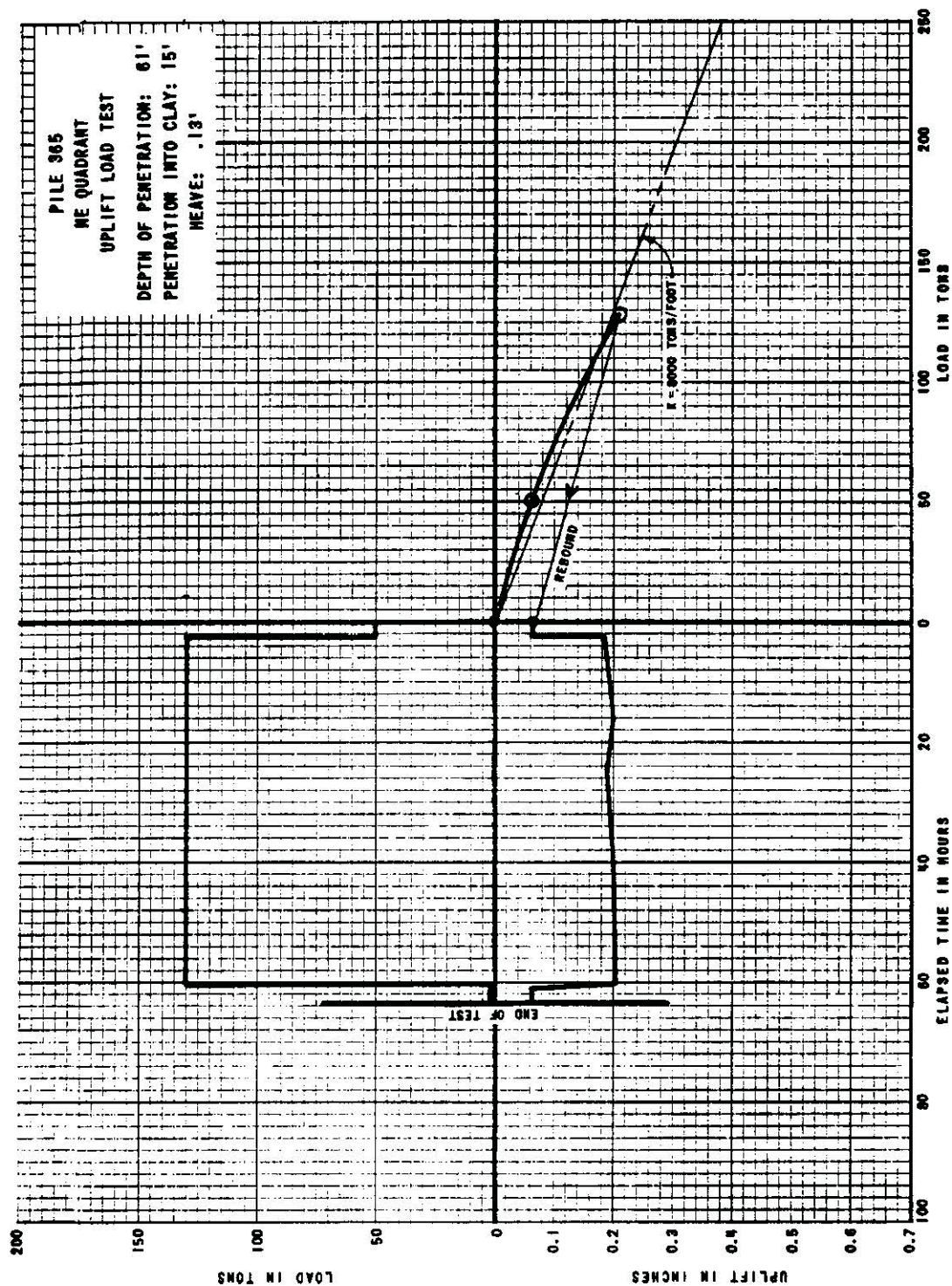
FIGURE
3.8.5 - 14



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PILE 271
COMPRESSION LOAD TEST

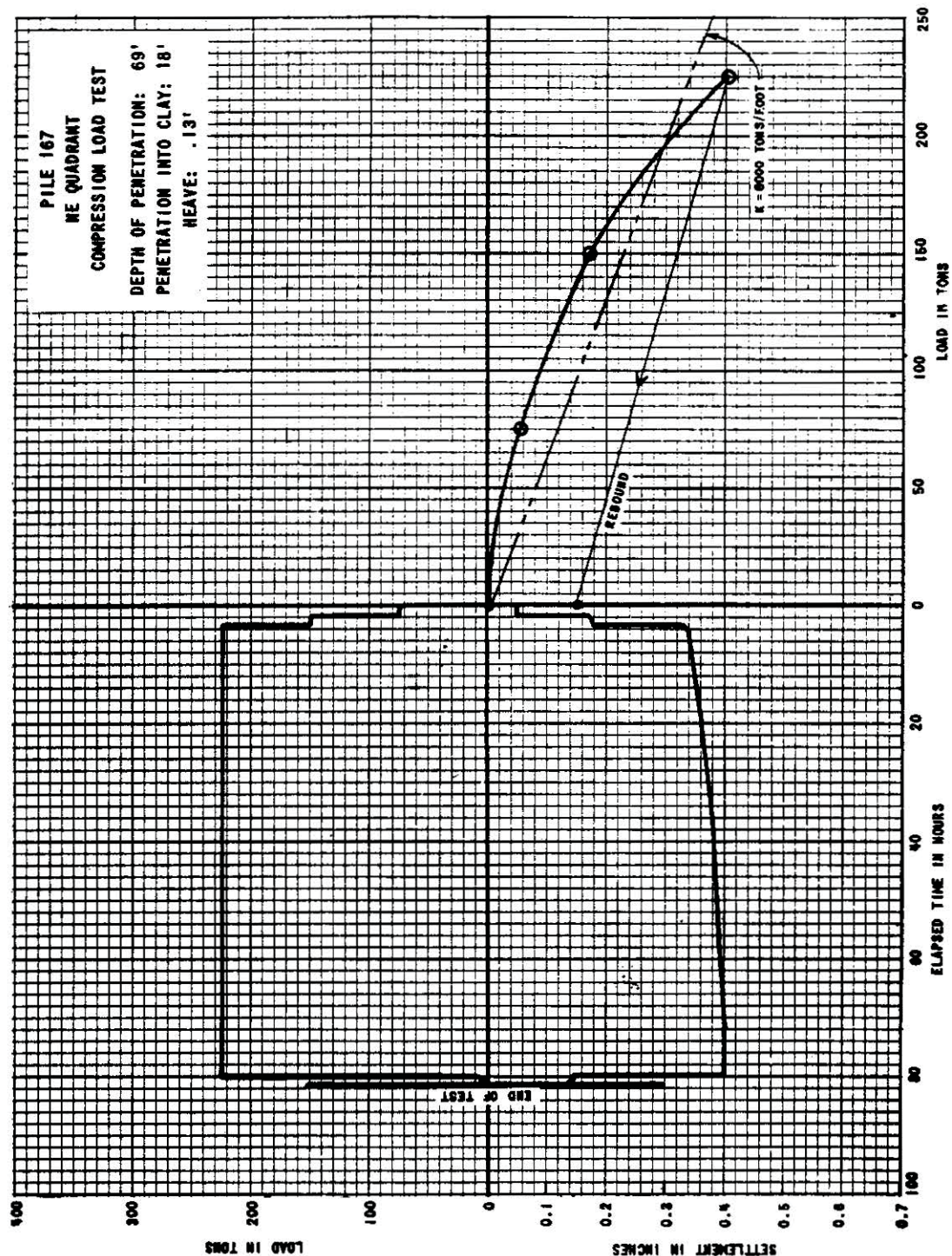
FIGURE
3.8.5 - 15

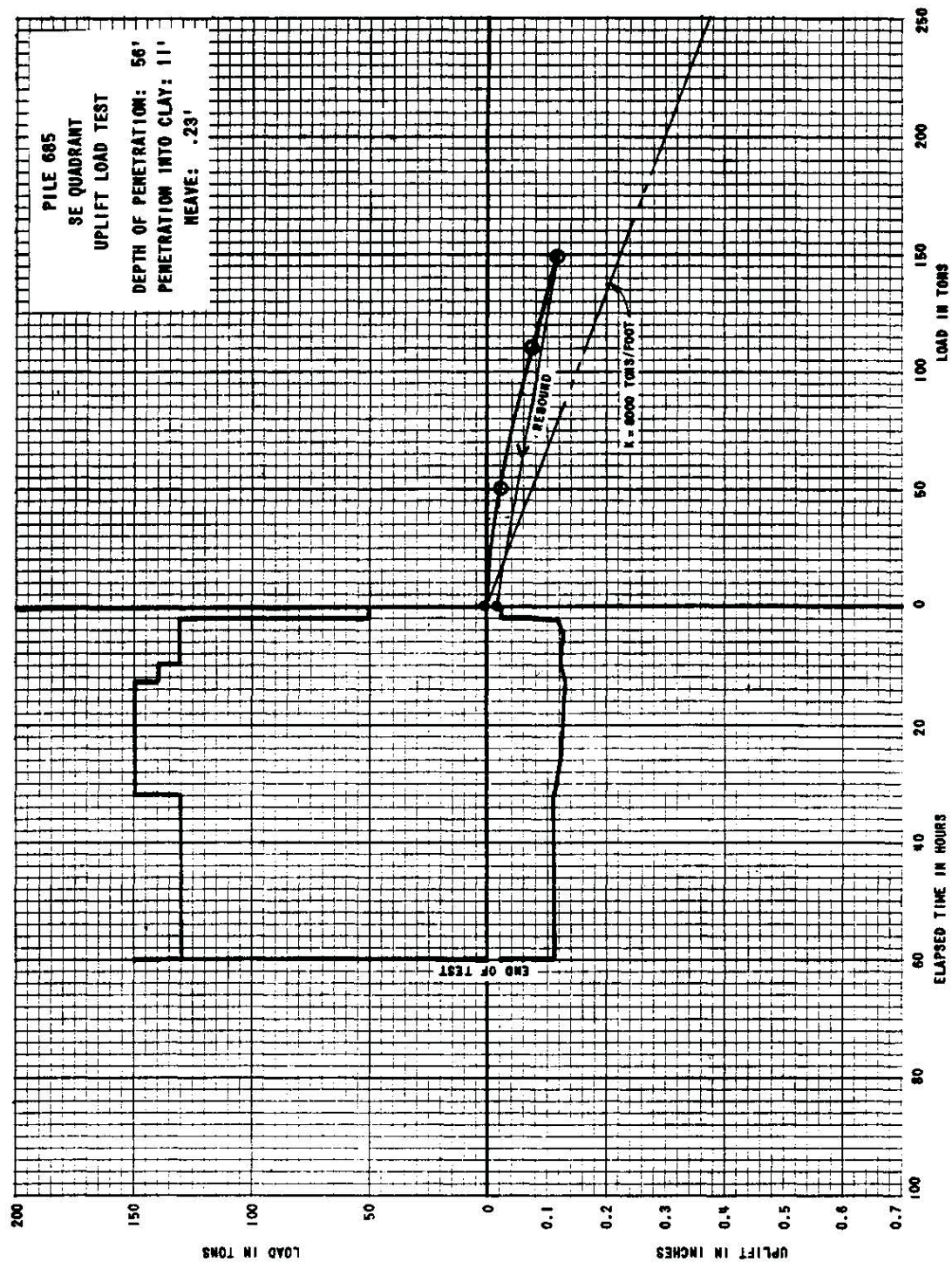


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PILE 365
UPLIFT LOAD TEST

FIGURE
3.8.5 - 16

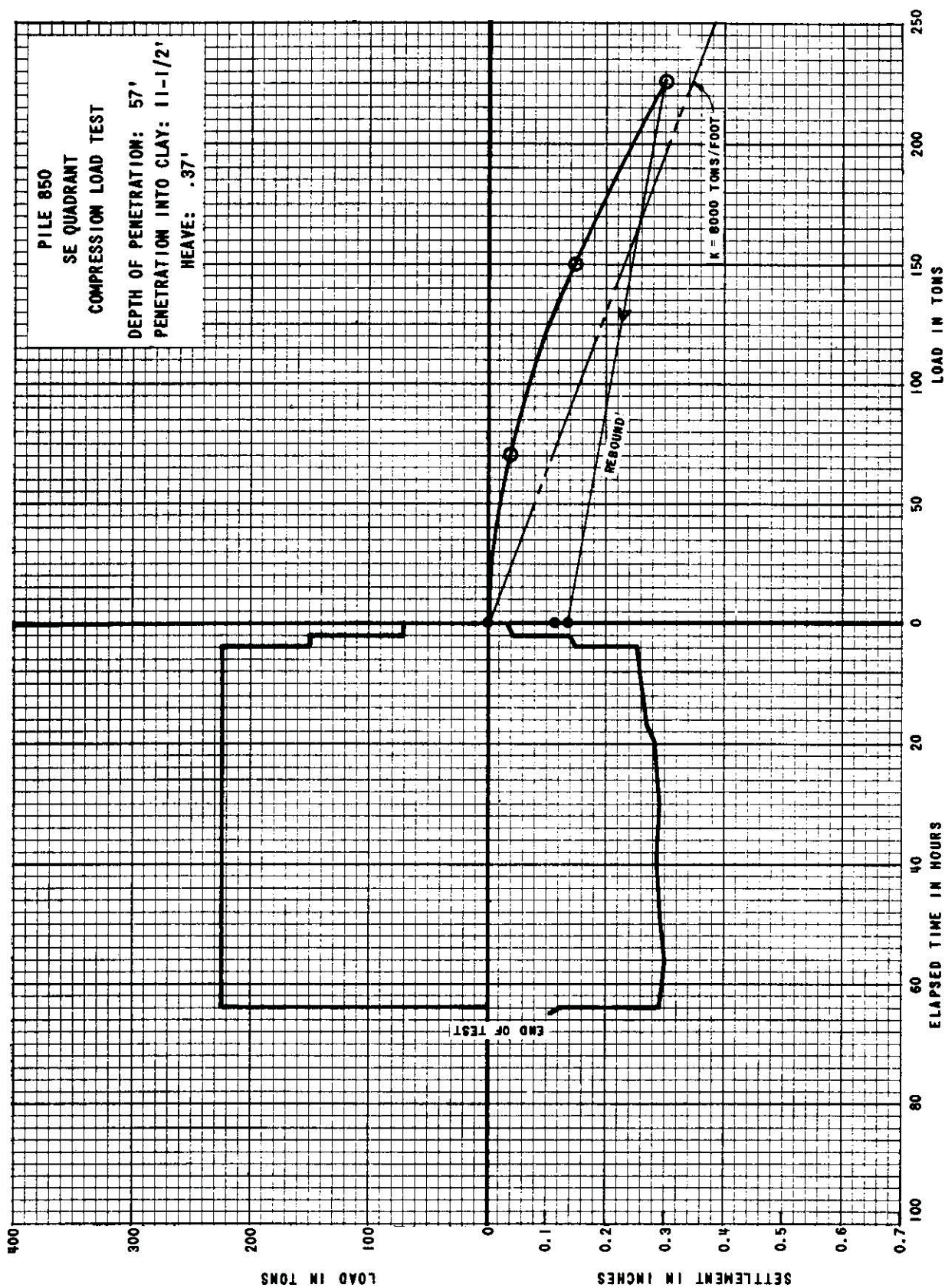




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PILE 685
UPLIFT LOAD TEST

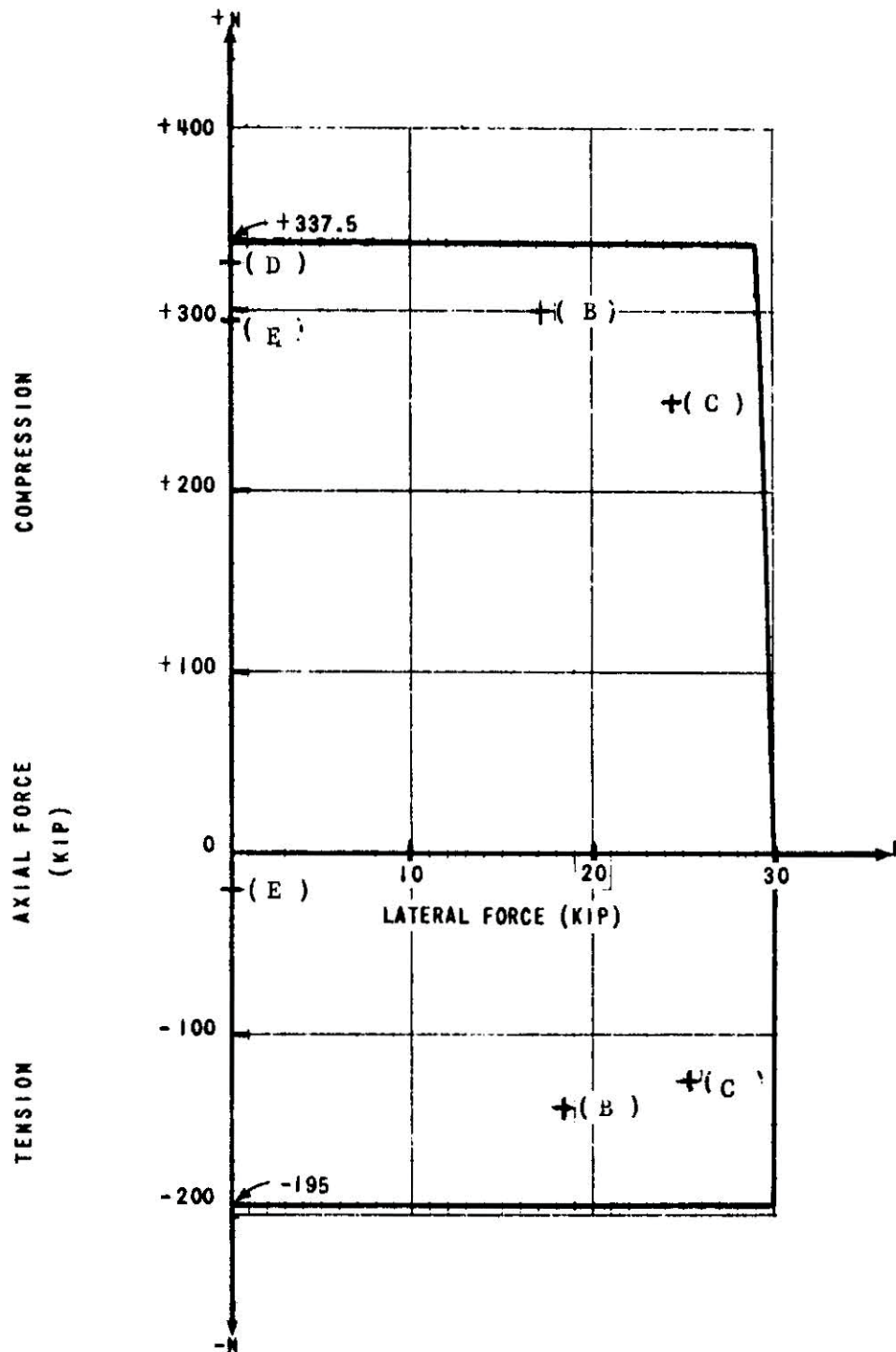
FIGURE
3.8.5 - 18



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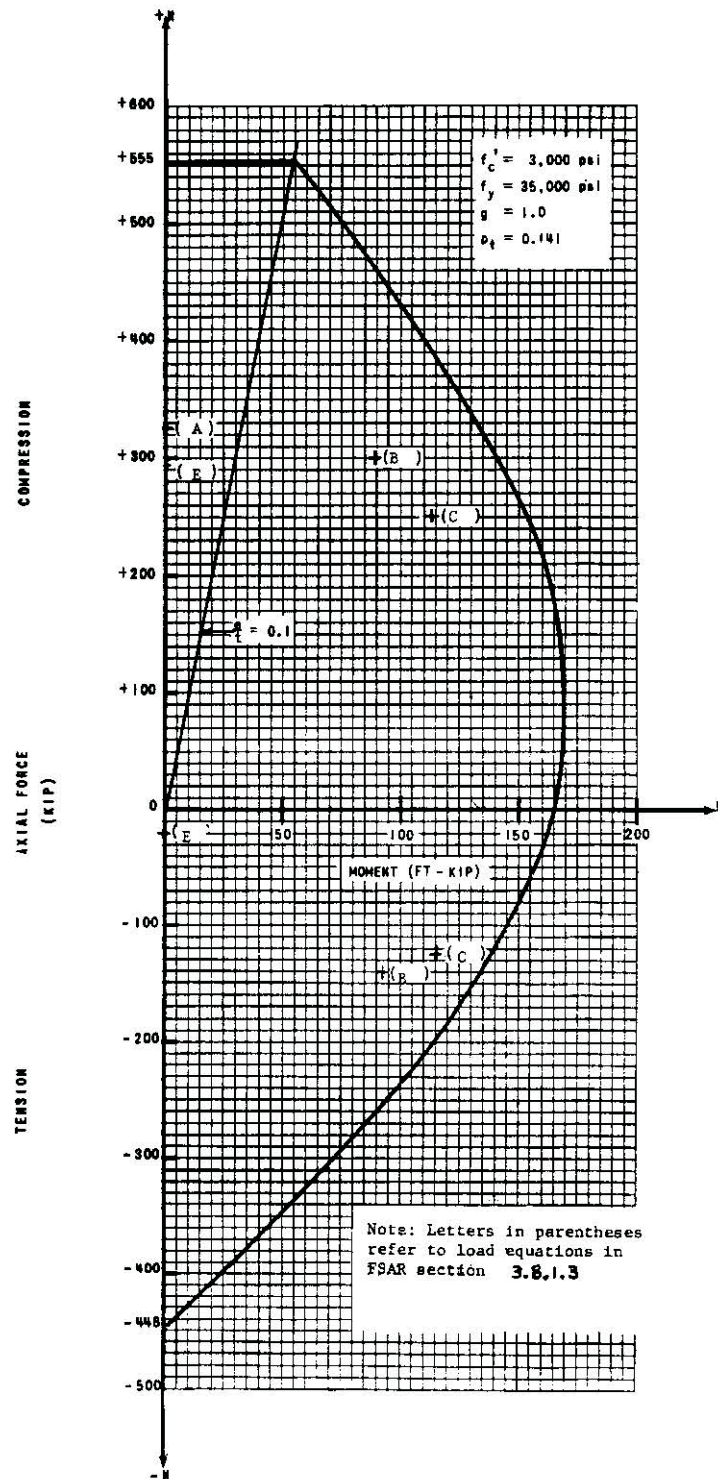
PILE 850
COMPRESSION LOAD TEST

FIGURE
3.8.5 - 19

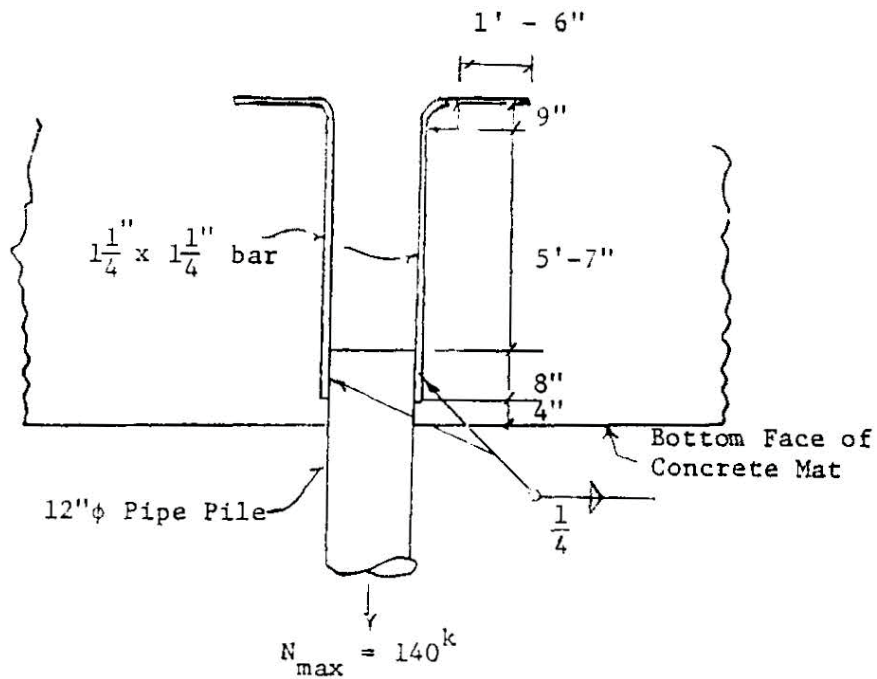


Note: Letters in parentheses refer to load equations in FSAR section 3.8.1.3

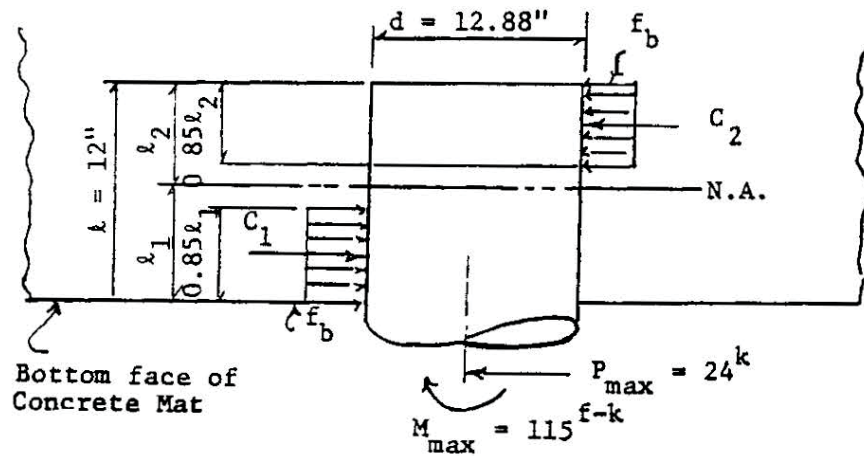
AXIAL FORCES VS. LATERAL FORCES
INTERACTION DIAGRAM



1. Anchor Straps - Type 6 Piles



2. Bearing interface - Type 5 and Type 6 piles



3.9 MECHANICAL SYSTEMS AND COMPONENTS

The seismic classification and industrial codes used for systems, supports, and components are presented in Section 3.2. The seismic analysis of Class I piping and equipment is presented in Section 3.7.3.

3.9.1 SPECIAL TOPICS FOR MECHANICAL SYSTEMS

All components in the Reactor Coolant System (RCS) were designed to withstand the effects of cyclic loads due to reactor system temperature and pressure changes. These cyclic loads are introduced by normal power changes, reactor trip, and startup and shutdown operation. The number of thermal and loading cycles used for design purposes and their bases are given in Table 3.9.1-1. During unit startup and shutdown, the rates of temperature and pressure changes are limited as indicated in Section 5.3.2. The cycles were estimated for equipment design purposes (40-year life), and were not intended to be an accurate representation of actual transients or actual operating experience. For example, the number of cycles for plant heatup and cooldown at 100°F per hour 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 was 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 Table 3.9.1-1, since the tabulation is only intended to represent normal design transients, the effect of these transients has been analytically evaluated and is included in the fatigue analysis for primary system components.

Over the range from 15 percent full power up to 95 percent of full power, the control system for the RCS and its components was designed to accommodate 10 percent of full power step changes in plant load and 5 percent of full power per minute ramp changes without reactor trip. However, under nominal operating conditions, the control system allows the plant to accept step load changes of 19 percent and ramp load changes of 14 percent per minute over the load range of 15 to 95 percent power. The plant will accept an approximately 60 percent loss of load from full power without reactor trip. The steam dumps to the condenser and the main steam PORVs will accommodate approximately 50 percent of this loss and the reactor coolant system will accommodate the remainder of approximately 10 percent.

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TABLE 3.9.1-1

THERMAL AND LOADING CYCLES

<u>TRANSIENT CONDITION</u>	<u>DESIGN CYCLES ⁽¹⁾</u>
1. Plant heatup at 100°F per hour	200 ⁽²⁾
2. Plant cooldown at 100°F per hour	200
3. Plant loading at 15% of full power per minute	29,000 ⁽³⁾
4. Plant unloading at 15% of full power per minute	29,000 ⁽³⁾
5. Step load increase of 20% of full power (but not to exceed full power)	2,000
6. Step load decrease of 20% of full power	2,000
7. Step load decrease of 95% of full power	80
8. Reactor trip	400
9. Hydrostatic test ⁽⁴⁾ pressure 3110 psig temperature 100°F	1 (pre-operational)
10. Hydrostatic test ⁽⁴⁾ pressure 2485 psig temperature 400°F	40 (post-operational)
11. Steady state fluctuations - the reactor coolant average temperature for purposes of design was assumed to increase and decrease a maximum of 5°F in one minute. The corresponding reactor coolant pressure variation is less than 100 psig. It was assumed that an infinite number of such fluctuations will occur.	
⁽¹⁾ Estimated for equipment design purposes and not intended to be an accurate representation of actual transients nor to reflect actual operating experience.	
⁽²⁾ This transient includes pressurizer to 10 percent above the operating pressure.	
⁽³⁾ This value has been limited to 18,560 based on fatigue considerations for pressurizer components.	
⁽⁴⁾ The steam generator lower assemblies, replaced in 1984, were designed to meet these transient conditions with a variation in the temperatures.	

3.9.2 DYNAMIC TESTING AND ANALYSIS

The seismic analysis of the reactor coolant loops, Class I equipment, and other Class I piping is discussed in Section 3.7.3. This section discusses flow and reactor coolant pump motor induced vibrations for the reactor coolant loop and the reactor internals. Also, the seismic analysis of the reactor internals is presented.

Operating experience of prior plants showed that no significant vibration occurred in the primary loops. Although thermal shield vibration was a problem in an early design, the rest of the components performed satisfactorily. The new thermal shield design, based on experience, analysis, and tests, solved this problem as demonstrated by the operation performance of Beznau 1 (NOK), Ginna, Zorita, Connecticut Yankee, and San Onofre 1.

Each of the reactor coolant pumps is equipped with two vibration pickups mounted at the top of the motor support stand. They are mounted in a horizontal plane and pick up radial vibrations of the pump. One is aligned parallel to the pump discharge; the other is aligned perpendicular to the pump discharge. Their signals are taken to a multi-point selector switch mounted outside the reactor containment. The signals from all the reactor coolant pumps are taken to this selector switch. The signals are read on a vibration meter which shows amplitude or velocity of vibration. The normal vibration level is less than 0.001 in. (double amplitude) at running speed (1200 rpm approx). A maximum level of 0.002 in. was specified.

As long as the primary coolant pumps are within specified limits, the other primary system components are considered to be satisfactory as demonstrated by previous plant operating experience. Instead of preoperational vibration tests, the primary coolant pumps were checked to determine if the vibration was within limits.

For the seismic analysis of the reactor internals, the maximum stresses were obtained by combining the contributions from the horizontal and vertical earthquake motion in the most conservative manner. The following paragraphs describe the horizontal and vertical contributions.

The Reactor Building with the reactor vessel support, the reactor vessel, and the reactor internals were included in this analysis. The mathematical model of the building is identical to that used to evaluate the building structure (Section 3.7). The reactor internals were mathematically modeled by beams, concentrated masses, and linear springs.

All masses, water, and metal were included in the mathematical model. All beam elements had the component weight or mass distributed uniformly, e.g., the fuel assembly mass and barrel mass. Additionally, wherever components were attached somewhat uniformly, their mass was included as an additional uniform mass, e.g., baffles and formers acting on the core barrel. The water near and about the beam elements was included as a distributed mass.

Horizontal components were considered as concentrated masses acting on the barrel. These concentrated masses also included components attached to the horizontal members since this was the media through which the reaction was transmitted. The water near and about these separated components was considered as being additive at these concentrated mass points.

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The concentrated masses attached to the barrel represent the following:

- a) The upper core support structure, including the upper vessel head and one-half the upper internals
- b) The upper core plate, including one-half the thermal shield and the other half of the upper internals
- c) The lower core plate, including one-half of the lower core support columns
- d) The lower one-half of the thermal shield
- e) The lower core support, including the lower instrumentation and the remaining half of the lower core support columns.

The modulus of elasticity was chosen at the highest design temperature for the three major materials found in the vessel, internals, and fuel assemblies. In considering shear deformation, the appropriate cross-sectional areas were selected along with a value for Poisson's ratio. The fuel assembly moment of inertia was derived from experimental results by static and dynamic tests performed on fuel assembly models. These tests provided stiffness values for use in this analysis.

The fuel assemblies were assumed to act together and were represented by a single beam. The following assumptions were made in regard to connection restraints. The vessel was pinned to the vessel support which was the surrounding concrete structure and part of the Containment Building. The barrel was clamped to the vessel at the barrel flange and spring-connected to the vessel at the lower core barrel radial support. This spring corresponded to the radial support stiffness for two opposite supports acting together. The beam representing the fuel assemblies was pinned to the barrel at the locations of the upper and lower core plates.

Modal analysis, along with the response spectrum method (Reference 3.9.2-1) was used in this analysis. The modal analysis was studied by the use of a transfer matrix method.

The maximum deflection, acceleration, etc., was determined at each particular point by summing the absolute values obtained for all modes. With the shear forces and bending moments determined, the individual earthquake stresses were then calculated.

Figure 3.9.2-1 shows the mathematical model used.

The reactor internals were modeled as a single degree of freedom system for vertical earthquake analysis. The maximum acceleration at the vessel support was increased by the amplification due to the building soil interaction.

The technique used to determine flow-induced vibrational effects in reactor internals was essentially a combination of test experience and analysis. Operating reactors had been instrumented with strain gages, accelerometers, pressure transducers, and deflection indicators in order to establish the level and trends of component responses. With test information as a

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reference, a straightforward stress analysis was used to determine the stresses at particular locations. These stresses (primary, secondary, and fatigue) were then compared against allowable values for the material.

The principal means of limiting the vibration effects was by the design. Conscientious efforts were made to increase stiffness of components and to avoid resonance conditions. In addition, operating reactors had instrumented and these results verified that vibration levels would be acceptable.

Allowable stress amplitudes for reactor internals vibration were established on the basis of the material fatigue properties for infinite cycles (endurance limit). Since infinite cycle fatigue was a criterion, no limits were necessary for frequency.

Displacements amplitudes for internals vibration were not governing. Stress limits were more restrictive.

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3.9.3 AMERICAN SOCIETY OF MECHANICAL ENGINEERS (ASME) CODE CLASS 1, 2, AND 3 COMPONENTS, COMPONENT SUPPORTS, AND CORE SUPPORT STRUCTURES

All components of the Reactor Coolant System (RCS) and associated systems were designed to the standards of the applicable ASME Code or USAS Code (Section 3.2). The loading combinations which were employed in the design of Class I components of these systems, i.e., vessels, piping, supports, vessel internals, and other applicable components, are given in Table 3.9.3-1.

This Table also indicates the stress limits which 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 must satisfy deformation limits which are more restrictive than the stress limits shown in Table 3.9.3-1. For this reason the reactor vessel internals are treated separately (Section 3.9.5).

3.9.3.1 Piping, Vessels, and Supports

The reasoning for selection of the load combinations and stress limits given in Table 3.9.3-1 is as follows. For the design earthquake, the Nuclear Steam Supply System (NSSS) was designed to be capable of continued safe operation, i.e., for the combination of normal loads and design earthquake loading. Critical equipment and supports needed for this purpose were required to operate within normal design limits.

In the case of the assumed hypothetical earthquake, it was only necessary to ensure that critical components do not lose their capability to perform their safety function, i.e., shut the plant down and maintain it in a safe condition. This capability was ensured by maintaining the stress limits as shown in Table 3.9.3-1. No rupture of a Class I pipe can be caused by the occurrence of the assumed hypothetical earthquake.

The design earthquake has the largest probable ground motion based on the site seismic history. The maximum potential earthquake has the largest potential ground motion based on seismic and geological factors and their uncertainties.

Careful design and thorough quality control during manufacture and construction, and periodic inspection during plant life, ensure that the independent occurrence of a reactor coolant pipe rupture is extremely remote. However, assuming a reactor coolant pipe ruptures, the stresses in the unbroken leg would be as noted in Table 3.9.3-1.

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 0.067g acting in the vertical and 0.1g acting in the horizontal planes simultaneously, are maintained within the allowable stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards (Section 3.2.2).

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- b) Primary steady state stresses, when combined with the seismic stress resulting from the response to a ground acceleration of 0.133g acting in the vertical and 0.2g acting in the 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.

Under the hypothetical earthquake of 0.2g ground acceleration, the following criteria will apply to Class I components:

- a) Stresses in steel supports and reinforced concrete supports will not exceed the yield strength
- b) In pressure vessels and piping, stresses will be limited to a value which provides margins against the development of a plastic hinge without considering the beneficial effects of strain hardening, the stress limit shall be 120 percent of the yield strength
- c) In equipment where translatory or rotational motion is essential to no loss of function, components are deflection limited, operating entirely within the elastic region.

All Class II structures and components are designed on the basis of a static analysis for a ground acceleration of 0.067g acting in the vertical and 0.1g acting in the horizontal directions simultaneously.

The coolant loop supports were designed to restrict the motion to about one-tenth of an inch, whereas the attached safety injection piping can sustain a 3 in. displacement without exceeding the working stress range.

All hangers, stops, and anchors were designed in accordance with USAS B31.1 Code for Pressure Piping and ACI 318 Building Code Requirements for Reinforced Concrete which provide minimum requirements on materials, design, and fabrication with ample safety margins for both dead and operational dynamic loads over the life of the equipment. In addition to the normal load conditions, the requirements of Table 3.9.3-1 for the loading combinations shown are used in design of supports except, the supports for the pressurizer safety and relief valve piping were designed for load combinations described in section 3.9.3.5.2. Specifically, these standards required the following:

- a) All materials used are in accordance with American Society for Testing and Materials (ASTM) specifications which establish quality levels for the manufacturing process, minimum strength properties, and for test requirements which ensure compliance with the specifications
- b) Qualification of welding processes and welders for each class of material welded and for types and positions of welds was required
- c) Maximum allowable stress values were established which provided an ample safety margin on yield strength for normal loads and ultimate strength for design basis accident (DBA) or maximum hypothetical seismic loads.

An extensive review of the steam generator and reactor coolant pump supports was performed during the latter part of 1977 as a result of Nuclear Regulatory Commission (NRC) questions (Reference 3.9.3-1).

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The following subjects are detailed in Reference 3.9.3-1 for these supports:

- a) Materials and material properties
- b) Design loads and loading conditions (normal, upset, emergency, and faulted)
- c) Load combinations and allowable stresses
- d) Welding procedures, and
- e) Inspections and nondestructive tests.

The worst load condition was the faulted condition (dead weight plus thermal load plus pipe rupture load). For this load condition the maximum stresses were below the allowable (Reference 3.9.3-1).

In response to IE Bulletin 79-02, Carolina Power & Light (CP&L) and Ebasco Services, Inc. is performing a verification, inspection, and testing program for pipe support base plates that use concrete expansion anchor bolts. This ongoing program covers the investigation of design concepts, calculations, and the inspection and testing of existing concrete expansion anchor bolts on Seismic Class I pipe support base plates. The initial results of this program showed that 92.6 percent of the Class I anchor bolts were satisfactory (Reference 3.9.3-2). More details are given by References 3.9.3-2 and 3.9.3-3. Further analysis is being conducted to resolve this issue.

As a result of seismic reanalysis of piping and pipe supports in accordance with the requirements of IE Bulletin 79-14, pipe supports in the Residual Heat Removal System and the Containment Spray System were modified to prevent a potential overstressed condition during a design basis earthquake (DBE) 0.2g ground acceleration (Reference 3.9.3-4 through 3.9.3-6). The deviations were the result of improper installation, i.e., the restraints as installed differed from those specified and analyzed originally. Further analysis is being conducted for safety related piping systems to resolve this issue.

The In-Service Inspection Program for Class 1, 2, and 3 components was submitted to the NRC by Reference 3.9.3-7. This program presents weld, support, component, and bolting inspection requirements for piping and component welds, supports, and bolting within the boundaries of safety-related systems. The component inspection program was developed in accordance with Subsections IWB, IWC, and IWD of ASME Section XI.

3.9.3.2 Reactor Vessel

The criteria for movement of the reactor vessel, under the worst combination of loads, i.e., normal plus the assumed hypothetical earthquake or normal plus reactor coolant pipe rupture loads, assure that the radial movement of the reactor vessel will not exceed the clearance between the reactor coolant piping and the surrounding concrete.

The relative motions between RCS components are controlled by the structures which are used to support the reactor vessel, the steam generators, the pressurizer, and the reactor coolant pumps.

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The supports are designed to limit the stresses in the pipe to the stress limits given in Table 3.9.3-1.

A stress evaluation of the reactor vessel was carried out in accordance with the rules of Section III of the ASME Nuclear Vessel Code. The original analysis, CENC-1111 assumed a T-cold head. Over time, it was found the head experienced T-hot conditions and a reanalysis of the affected vessels sections was performed to address the issue (Ref. 3.9.3-13). The evaluation demonstrated that stress levels are within the stress limits of the Code. Table 3.9.3-2 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 3.9.3-3.

The cycles specified for the fatigue analysis are the results of an evaluation of the expected plant operation coupled with experience from nuclear power plants in service, such as Yankee-Rowe. These cycles include five heatup and cooldown cycles per year. During the vessel head replacement project, the transient definitions in the E-Spec 676367 and the stress report, CENC-1111, were clarified when Westinghouse provided the PAR 32 document in letter LTR-RCDA-04-1418 Rev. 1, dated 06 January 2005. The document defined the actual design transients used in the reactor vessel stress report, CENC-1111. The vessel design is discussed further in Section 5.3.

3.9.3.3 Steam Generators

The lower shell assembly in each steam generator was replaced in 1984 and incorporates design improvements over the original Westinghouse Model 44 units fabricated in the late 1960s.

Calculations confirmed that the steam generator tube sheet will withstand the loading (which would be a quasi-static rather than a shock loading) caused by a loss of reactor coolant. The maximum primary membrane plus primary bending stress in the tube sheet under these conditions is 23,600 psi. This is well below ASME Section III yield strength of 41,400 psi at 650°F. Because the pressure in the primary channel head would drop to zero under the condition postulated, no damage would result to the channel head.

The rupture of primary or secondary piping was assumed to impose a maximum pressure differential of 2250 psi across the tubes and tube sheet from the primary side or a maximum pressure differential of 1100 psi across the tubes and tube sheet from the secondary side, respectively. Under these conditions there is no rupture of the primary to secondary boundary (tubes and tube sheet). This criterion prevented any violation of the containment boundary.

To meet this criterion it was established that under the postulated accident conditions (where a primary to secondary side differential pressure of 2250 psia exists) the primary membrane stresses in the tube sheet ligaments averaged across the ligament and through the tube sheet thickness did not exceed 90 percent of the material yield stress at the operating temperature. Also, the primary membrane plus primary bending stress in the tube sheet ligaments, averaged across the ligament width at the tube sheet surface location giving maximum stress, did not exceed 135 percent of the material yield stress at the operating temperature.

This criterion is applicable to abnormal operating circumstances in that it is

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consistent with the ASME, Nuclear Pressure Vessel Code, Section III rules, Paragraph N-714, 2 for hydrotest limitations.

An examination of stresses under these conditions shows that for the case of a 2485 psig maximum tube sheet pressure differential the stresses were within acceptable limits. These stresses together with the corresponding stress limits are given in Table 3.9.3-4.

The tubes were designed to the requirements (including stress limitations) of Section III for normal operation, assuming 2485 psig as the normal operating pressure differential. Hence, the secondary pressure loss accident condition imposes no extraordinary stress on the tubes beyond that normally expected and considered in Section III requirements.

When the plant was designed, no significant corrosion of the Inconel tubing was expected during the lifetime of the plant. The corrosion rate reported in Reference 3.9.3-8 shows "worst case" rates of 15.9 mg/dm² in the 2000 hr test under steam generator operating conditions. Conversion of this rate to a 40 year plant life gives a corrosion loss of less than 1.5×10^{-3} in., which is insignificant compared to the nominal tube wall thickness of 0.050 in. Tube plugging is used to isolate a defective tube to preclude primary-to-secondary leakage.

In the case of a primary pressure loss accident, the secondary to primary pressure differential can reach 1100 psig. This pressure differential is less than the primary to secondary design pressure differential (1550 psi) for normal operating conditions. Hence, no stresses in excess of those covered in Section III rules for normal operation are experienced on the tube sheet for this accident case. Actual pressure tests of 3/4 in. OD/.058 in. wall Inconel tubing show collapse under external pressure of 5700-5900 psi. Extrapolating this data to 7/8 in. OD/.050 in. wall tubes, collapse would occur at about 2630 psi at 650°F. This gives a factor of safety of 2.4 against collapse under the 1100 psig accidental application of external pressure to tubes. A check of the ASME Section VIII design curves for Iron-Chromium-Nickel Steel cylinders under external pressure indicates a predicted collapse pressure for the tubes of 2310 psi, which is close to the extrapolated value for the experimental results.

Consideration has been given to the superimposed effects of secondary side pressure loss and the maximum potential earthquake loading. The fluid dynamic forces on the internal components affecting the primary to secondary boundary (tubes) has been considered as well. For this condition, the criterion is that no rupture of primary to secondary boundary (tubes and tube sheet) occurs.

For the case of the tube sheet, the maximum hypothetical earthquake loading will contribute an equivalent static pressure loading over the tube sheet of less than 10 psi (for vertical shock). Such an increase is small when compared to the pressure differentials (up to 2485 psig) for which the tube sheet is designed. Under horizontal shock loading of the maximum hypothetical earthquake, the stresses are less than those for 1.0g gravity loading experienced in a horizontal position, which the design can readily accept.

The fluid dynamic forces on the internals under secondary steam break accident conditions indicate, in the most severe case, that the tubes are adequate to

constrain the motion of the baffle plates with some plastic deformation, but boundary integrity is maintained.

The ratio of the allowable stresses (based on an allowable membrane stress of 0.9 of the nominal yield stress of the material) to the computed stresses is summarized in Table 3.9.3-5.

3.9.3.4 System Integrity

A complete stress analysis which reflects consideration of all design loadings detailed in the design specification was prepared by the manufacturer. The analysis showed that the reactor vessel, steam generator, pump casing, and pressurizer comply with the stress limits of Section III of the ASME Code. A similar analysis of the reactor coolant piping showed that it complies with the stress limits of the applicable sections of USAS B31.1.

As part of the design control on materials, Charpy V-notch toughness test curves were run on all ferritic material used in fabricating pressure parts of the reactor vessel, steam generator, and pressurizer to provide assurance for hydrotesting and operation in the ductile region at all times. In addition, drop-weight tests were performed on the reactor vessel plate material.

As an assurance of system integrity, all original components in the system were hydrotested at 3110 psig prior to initial operation.

Following the replacement of the steam generator lower assemblies in 1984, the new primary welds were also hydrostatically tested prior to startup.

3.9.3.5 Design and Installation Details for Mounting of Pressure-Relief Devices

3.9.3.5.1 Modifications to Safety Valve SVI-4-C

In April, 1970 during hot functional testing of secondary safety valve lifting setpoints, a 6 in., Schedule 80 connection pipe to safety valve SVI-4-C failed allowing the valve to be blown completely off the connecting pipe (Reference 3.9.3-9). The failure was caused by an overload condition at a thickness change location; however, the actual failure scenario was not identified. The inlet nozzles to 12 main steamline safety valves were redesigned to increase the conservatism of the stress level and to decrease the possible stress intensification. The nozzle size was increased to an 8 in., Schedule 160 pipe. The stress calculations are shown in Reference 3.9.3-9.

3.9.3.5.2 Pressurizer Safety and Relief Piping Evaluation

In response to NUREG-0737, Item II.D.1, Reference 3.9.3-10 "Pressurizer Safety and Relief Line Evaluation Summary Report" was prepared by Westinghouse. That report demonstrates the functional ability and structural integrity of safety and relief valve discharge piping and supports. The following paragraphs summarize the evaluation.

The pressurizer safety and relief valves provide overpressure protection for the reactor coolant system, as described in Section 5.2.2. To prevent a steam interface at the valve seat and thereby eliminate the possibility of valve leakage, a liquid seal consisting of a small amount of water is maintained upstream of each pressurizer safety and relief valve. Upon actuation of a

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valve, its water seal is discharged through the valve and its discharge piping, generating hydraulic shock loads on the piping supports.

The piping between the pressurizer nozzles and the pressurizer relief tank was analyzed according to the requirements of the appropriate equations of the

ANSI B31.1-1967 code. These equations establish limits for stresses from sustained loads, sustained plus occasional loads (including earthquake), thermal expansion loads, and sustained plus thermal expansion loads. Appropriate load combinations and acceptance criteria were developed (See Reference 3.9.3-11 for load combination criteria and Reference 3.9.3-12 for correct loads.) The load combinations and acceptance criteria are identical to those recommended by the piping subcommittee of the PWR safety and relief valve test program, sponsored by the Electric Power Research Institute (EPRI).

The thermal-hydraulic analysis used computer programs which have been shown to match the results of the EPRI Test Program. Hydraulic forcing functions were generated assuming either the simultaneous opening of all safety valves or the simultaneous opening of both relief valves since these represent the worst applicable loading cases for the piping and supports of this specific layout.

The analysis demonstrated that, contingent on support adequacy, the operability and structural integrity of the pressurizer safety and relief valve piping is ensured for all applicable loadings and load combinations.

The piping supports were structurally modified to provide adequate support, for all the loading combinations developed to consider the maximum possible loading which may be imposed on the supports from the piping analysis. All supports satisfy the requirements of the original criteria.

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TABLE 3.9.3-1

LOADING COMBINATIONS AND STRESS LIMITS

<u>LOADING COMBINATIONS</u>	<u>VESSELS</u>	<u>PIPING</u>	<u>SUPPORTS</u>
1. Normal Loads	$P_m \leq S_m$ $P_m \leq S$ $P_L + P_B \leq 1.5 S_m$	Working Stresses or $P_L + P_B \leq S$	Applicable Factored Load Design Values
2. Normal + Design Earthquake Loads	$P_m \leq S_m$ $P_m \leq 1.2 S$ $P_L + P_B \leq 1.5 S_m$	1-1/3 Working Stresses or $P_L + P_B \leq 1.2 S$	Applicable Factored Load Design Values
3. Normal + Assumed Hypothetical Earth- quake Loads	$P_m \leq 1.2 S_m$ $P_L + P_B \leq 1.2 (1.5 S_m)$	$P_m \leq 1.2 S$ $P_L + P_B \leq 1.2 (1.5 S)$	Deflections and Stresses of Supports Limited to Maintain Supported Equipment Within their Stress Limits
4. Normal + Pipe Rupture Loads	$P_m \leq 1.2 S_m$ $P_L + P_B \leq 1.2 (1.5 S_m)$	$P_m \leq 1.2 S$ $P_L + P_B \leq 1.2 (1.5 S)$	Deflections and Stresses of Supports Limited to Maintain Supported Equipment Within Their Stress Limits

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

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TABLE 3.9.3-2

SUMMARY OF ESTIMATED PRIMARY PLUS SECONDARY STRESS INTENSITY
FOR COMPONENTS OF THE REACTOR VESSEL

<u>AREA</u> <u>TEMPERATURE)</u>	<u>STRESS INTENSITY (psi)</u>	ALLOWABLE STRESS 3 S _m (psi) <u>(OPERATING</u>
Control Rod Housing	49,240 (1)	50,100 (1)
Head Flange	57,530 (2)	80,100 (2)
Vessel Flange	59,440 (3)	80,100 (3)
Closure Studs	71,730 (3)	99,600 (3)
Outlet Nozzles	73,600	80,000
Inlet Nozzles & Vessel Supports	46,800	80,000
Core Support Pad	66,000	69,900
Bottom Head to Shell Juncture	29,900	74,700*
Bottom Instrumentation	51,300	69,900
Vessel Wall Transition	31,600	74,700*

* Allowable value for SA 302 Grade A

(1) WCAP-16400-P Table 8-4 Page 8-13 (3S_m, based on 650 degF, SA-182 Gr F316).

(2) WCAP-16369-P Table 8-4 Page 8-14 (3S_m, based on 650 degF, SA-508).

(3) Reference 3.9.3-13.

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TABLE 3.9.3-3

SUMMARY OF ESTIMATED CUMULATIVE FATIGUE USAGE FACTORS FOR
COMPONENTS OF THE REACTOR VESSEL

<u>ITEM</u>	<u>USAGE FACTOR*</u>
Control Rod Housing	0.479 (1)
Head Flange	0.17 (2)
Vessel Flange	0.037 (3)
Stud Bolts	0.693 (3)
Outlet Nozzles	0.630
Inlet Nozzles & Vessel Support	0.375
Core Support Pad	0.230
Bottom Head-to-Shell Juncture	0.001
Bottom Instrumentation	0.119
Vessel Wall Transition	0.001

*As defined in Section III of the ASME Boiler and Pressure Vessel Code, Nuclear Vessels.

(1) WCAP 16400-P Table 8-4 Page 8-13 (CRDM).

(2) WCAP 16369-P Table 8-4 Page 8-14 (Reactor-Head).

(3) Reference 3.9.3-13.

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TABLE 3.9.3-4

STRESSES DUE TO MAXIMUM STEAM GENERATOR TUBE
SHEET PRESSURE DIFFERENTIAL (2485 PSIG)

<u>STRESS</u>	(668°F) <u>COMPUTED VALUE</u>	<u>ALLOWABLE VALUE</u>
Primary Membrane Stress	23,300 psi	37,000 psi (.9 Sy)
Primary Membrane plus Primary Bending Stress	53,000 psi	55,600 psi (1.35 Sy)

In addition to the foregoing evaluation elasto-plastic limit analysis of the tube sheet-head-shell combination indicates a limit pressure of 3400 psi at operating conditions, giving a safety factor of 1.36 for the abnormal condition.

The steam generator lower assemblies were replaced in 1984. The stress values above were developed for the original steam generators. This table remains unchanged due to the original channel head and steam domes (with certain modifications) being reused with the new lower assemblies.

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TABLE 3.9.3-5

RATIO OF ALLOWABLE STRESSES TO COMPUTED STRESSES
FOR A STEAM GENERATOR TUBE
SHEET PRESSURE DIFFERENTIAL OF 2485 PSIG

<u>COMPONENT PART RATIO</u>	<u>STRESS</u>
Channel Head	1.35
Channel Head-Tube Sheet Joint	1.63
Tubes	1.20
Tube Sheet	
Maximum Average Ligament	1.04
Effective Ligament	1.58

3.9.4 CONTROL ROD DRIVE SYSTEM (CRDS)

The control rod drive mechanisms (CRDM) are used for withdrawal and insertion of the rod cluster control assemblies (RCCA) 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 Figure 3.9.4-1, consists of the internal (latch) assembly, the pressure vessel, the operating coil stack, the drive shaft assembly, and the position indicator coil stack. Each of these components is described below.

Each assembly is an independent unit. Each drive is welded onto an adaptor on top of the reactor pressure vessel (RPV) 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.

Reactor coolant fills the pressure containing parts of the drive mechanism. All working components and the shaft are immersed in the reactor 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 magnets are turned on and off in a fixed sequence by solid-state switches for the full-length rod 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. The mechanisms were designed to operate in water at 650°F and 2485 psig. The temperature at the mechanism head adaptor is 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 RPV. A multi-conductor cable connects the mechanism operating coils to the 125 volt DC power supply. The power supply is described in Chapter 7.0.

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.

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The upper set of latches moves up or down to raise or lower the drive rod by 5/8 in. The lower set of latches has 1/16 in. axial movement to shift the weight of the control rod from the upper to the lower latches. The housings are designed in accordance with the requirements for Class A vessels of the American Society of Mechanical Engineers (ASME) Nuclear Vessel Code.

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.

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 can be removed or installed while the reactor is pressurized.

The operator coils (A, B, and C) are made of round copper wire which is insulated with a double layer of filament type glass yarn.

The design operating temperature of the coils is 232°C (450°F). 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 (392°F).

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° 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. At its upper end, there is a disconnect assembly for remote disconnection of the drive shaft assembly from the 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.

The position indicator coil stack slides over the rod travel housing section of the pressure vessel. It detects drive rod position by means of cylindrically-wound differential transformers which span the normal length of the rod travel (144 in.).

Section 4.5 discusses CRDS materials.

For operation of the CRDS, the drive mechanisms shown schematically in Figure 3.9.4-2 withdraw and insert their respective control rods as electrical pulses are received by the operator coils.

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ON and OFF sequence, repeated by switches in the power programmer, causes either withdrawal or insertion of the control rod. Position of the control rod is indicated by the differential transformer action of the position indicator coil stack surrounding the rod travel housing. The differential transformer output changes as the top of the ferromagnetic drive shaft assembly moves up the rod travel housing.

Generally, during plant operation, the drive mechanisms hold the control rods withdrawn from the core in a static position, and only one coil (either the movable gripper coil, or the stationary gripper coil) is energized on each mechanism.

The control rod is withdrawn by repeating the following sequence:

- a) Movable Gripper Coil - ON - The movable gripper armature raises and swings the movable gripper latches into the drive shaft groove.
- b) Stationary Gripper Coil - OFF - Gravity causes the stationary gripper latches and armature to move downward until the load of the drive shaft is transferred to the movable gripper latches. Simultaneously, the stationary gripper latches swing out of the shaft groove.
- c) Lift Coil - ON - The 5/8 in. gap between the lift armature and the lift magnet pole closes and the drive rod raises one step length.
- d) Stationary Gripper Coil - ON - The stationary gripper armature raises and closes the gap below the stationary gripper magnetic pole, and swings the stationary gripper latches into a drive shaft groove. The latches contact the shaft and lift it 0.047 in. The load is so transferred from the movable to the stationary gripper latches.
- e) Movable Gripper Coil - OFF - The movable gripper armature separates from the lift armature under the force of three springs and gravity. Three links, pinned to the movable gripper armature, swing the three movable gripper latches out of the groove.
- f) Lift Coil - OFF - The gap between the lift armature and the lift magnet pole opens. The movable gripper latches drop 5/8 in. to a position adjacent to the next groove.

The sequence for control rod insertion is similar to that for control rod withdrawal:

- a) Lift Coil - ON - The movable gripper latches are raised to a position adjacent to a shaft groove
- b) Movable Gripper Coil - ON - The movable gripper armature raises and swings the movable gripper latches into a groove
- c) Stationary Gripper Coil - OFF - The stationary gripper armature moves downward and swings the stationary gripper latches out of the groove
- d) Lift Coil - OFF - Gravity separates the lift armature from the lift magnet pole and the control rod drops down 5/8 in

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- e) Stationary Gripper Coil - ON
- f) Movable Gripper Coil - OFF

The sequences described above are termed as one step or one cycle and the control rod moves 5/8 in. for each cycle. Each sequence can be repeated at a design rate of up to 72 steps per minute and the control rods can therefore be withdrawn or inserted at a design rate of up to 45 in./min.

If power to the movable gripper coil is cut off, as for tripping, the combined weight of the drive shaft and the RCCA is sufficient to move the latches out of the shaft groove. The control rod falls by gravity into the core. The tripping occurs as the magnetic field, holding the movable gripper armature against the lift magnet, collapses and the movable gripper armature is forced down by the weight acting upon the latches.

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3.9.5 REACTOR PRESSURE VESSEL INTERNALS

3.9.5.1 Design Arrangement

The reactor core and reactor vessel internals are shown in cross-section in Figure 4.1.1-2. and in elevation in Figure 4.1.1-1. The core, consisting of the fuel assemblies, control rods, source rods, and guide thimble plugging devices, provides and controls the heat source for the reactor operation. 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 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 low 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.

The reactor internals are designed to support and orient the reactor core fuel assemblies and control rod assemblies, absorb the control rod dynamic loads, and transmit these and other loads to the reactor vessel flange, provide a passageway for the reactor coolant, and support in-core instrumentation. The reactor internals are shown in Figure 4.1.1-1.

The reactor internals are equipped with bottom-mounted in-core instrumentation supports. These supports are designed to sustain the applicable loads in Section 3.9.5.2.

In the event of downward vertical displacement of the internals, energy absorbing devices limit the displacement by contacting the vessel bottom head. The load is transferred through the energy devices to the vessel. The energy absorbers, cylindrical in shape, are contoured on their bottom surface to the reactor vessel bottom head geometry. Their number and design are determined so as to limit the forces imposed to a safe fraction of yield strength. Assuming a downward vertical displacement, the potential energy of the system is absorbed mostly by the strain energy of the energy absorbing devices.

The free fall in the hot condition is on the order of 1/2 in., and there is an additional strain displacement in the energy absorbing devices of approximately 3/4 in. Alignment features in the internals prevent cocking of the internals structure during this postulated drop. The control rods are designed to provide assurance of control rod insertion capabilities under these assumed drop of internals conditions. The drop distance of about 1 1/4 in. is not enough to cause the tips of the shutdown group of rod cluster control assembly (RCCA) to come out of the guide tubes in the fuel assemblies.

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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 incore instrumentation support structure.

3.9.5.1.1 Lower Core Support Structure

The major containment and support member of the reactor internals is the lower core support structure, shown in Figure 3.9.5-1. This support structure assembly consists of the core barrel, core baffle, lower core plate, support columns, thermal shield, intermediate diffuser plate, and 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.

The lower core plate provides the necessary flow distributor holes for each fuel assembly. 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.

The one piece thermal shield is supported from the core barrel by lugs positioned at the bottom and top of the shield. 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 one in. long. This type of attachment has an extremely high natural frequency. The specimens are held in position within the baskets by a stop on 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.

The lower core support structure and principally 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.

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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 exit through the vessel outlet nozzles.

Vertically 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 core barrel shell and partially through the lower support columns to the bottom support plate, and thence, 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 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 ID. 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 expansion 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 American Society of Mechanical Engineers (ASME) Section III limits.

3.9.5.1.2 Upper Core Support Assembly

The upper core support assembly, shown in Figure 3.9.5-2, 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, shown on Figure 3.9.5-3, sheath and guide the control rod drive (CRD) shafts and control rods, and provide no other mechanical functions. 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 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

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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 thereby assured by this system of locating pins and guidance arrangements. 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 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 a lower system to convey and support flux thimbles penetrating the vessel through the bottom.

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 table. The minimum bend radii are about 12 ft. and the trailing ends of the thimbles (at the seal table) are extracted approximately 13-23 ft during refueling of the reactor in order to avoid interference within the core. The thimbles are closed at the leading (reactor) 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 table. During normal operation, the retractable thimbles are stationary and move only during refueling or for maintenance, at which time a space of approximately 23 ft above the seal line is cleared for the retraction operation. Section 7.2 contains more information on the layout of the in-core instrumentation system.

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The incore 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 refueling sequence.

3.9.5.1.4 Evaluation of Core Barrel and Thermal Shield

The internals design was based on analysis, test, and operational information. Troubles in previous Westinghouse pressurized water reactor (PWR) were evaluated and information derived was considered in this design. For example, the new Westinghouse design used a one-piece thermal shield which is attached rigidly to the core barrel at one end and flexured at the other. The earlier designs that malfunctioned were multi-piece thermal shields that rested on the vessel lugs and were not rigidly attached at the top.

Early core barrel designs that have malfunctioned in service, now abandoned, employed threaded connections such as tie rods, joining the bottom support to the bottom of the core barrel, and a bolted connection that tied the core barrel to the upper barrel. The malfunctioning of core barrel designs in earlier service was believed to have been caused by the thermal shield which was oscillating, thus creating forces on the core barrel. Other forces were induced by unbalanced flow in the lower plenum of the reactor. In today's RCCA design there are no fuel followers to necessitate a large bottom plenum in the reactor. The elimination of these fuel followers enabled Westinghouse to build a shorter core barrel.

The Connecticut Yankee reactor and the Zorita reactor core barrels are of the same construction as the H.B. Robinson reactor core barrel. Deflection measuring devices employed in the Connecticut Yankee reactor during the hot-functional test, and deflection and strain gages employed in the Zorita reactor during the hot-functional test provided important information that was used in the design of the Robinson Vintage internals, including that for Indian Point. When the Connecticut Yankee thermal shield was modified to the same design as for Southern California Edison, it, too, operated satisfactorily as was evidenced by the examination after the hot-functional test. After these hot-functional tests on all of these reactors, a careful inspection of the internals was provided. All the main structural welds were examined, nozzle interfaces were examined for any differential movement, upper core plate inside supports were examined, the thermal shield attachments to the core barrel including all lockwelds on the devices used to lock the bolt were checked, and no malfunctions were found.

Substantial scale model testing was performed at Westinghouse Atomic Power Division. This included tests which involved a complete full scale fuel assembly which was operated at reactor flow, temperature, and pressure conditions. Tests were run on a 1/7th scale model of the Indian Point reactor. Measurements taken from these tests indicated very little shield movement, on the order of a few mils when scaled up to H. B. Robinson. Strain gage measurements taken on the core barrel also indicated very low stresses. Testing to determine thermal shield excitation due to inlet flow disturbances was included. Information gathered from these tests was used in the design of the thermal shield and core barrel. It was concluded from the testing program and the analyses with the experience gained that the design as employed on the H. B. Robinson Plant is adequate.

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3.9.5.2 Loading Conditions

The internals were designed to withstand the forces due to weight, preload of fuel assemblies, control rod dynamic loading, vibration, and earthquake acceleration. These internals were analyzed in a manner similar to Connecticut Yankee, San Onofre, Zorita, Saxton, and Yankee. Under the loading conditions, including conservative effects of design earthquake loading, the structure satisfies stress values prescribed in Section III, ASME Nuclear Vessel Code. The dynamic criteria for design and the stress levels of the internals in this plant are similar to those in Connecticut Yankee.

Tabulation of the design and predicted blowdown and seismic loads on components was not made because of the dynamic analysis methods used. With these methods, response of systems were studied without using fictitious static forces. As a result of a double ended pipe break in a very short period of time, depressurization waves would be transmitted through the primary loop into the reactor vessel. These waves would be amplified, attenuated, superimposed, and slowed down, when passing through the system. The result would be a complicated transient distribution of pressures and coolant velocities inside the reactor. Loadings on components would be the result of these pressures and water velocities, changing rapidly (high frequency oscillations) during the first 0.250 seconds (for the 0.001 seconds time break).

The loads on components were obtained by integrating the resulting pressure and drag force distributions as a function of time. These loads were then applied to the structures via structural codes which gave the response of the components also as a function of time. Consequently, the reactor internals were not studied for a given set of blowdown forces that could be tabulated, but for time variable loads. Amplitudes, phase, and frequency of the exciting loads were equally important factors for the internals excitation. The limits for deflection for abnormal conditions are discussed in Chapter 15.0.

3.9.5.3 Design Bases

3.9.5.3.1 Design Criteria for Normal Operation

The internals and core were designed for normal operation conditions and subjected to loads of mechanical, hydraulic, and thermal origin. The response of the structure under the design earthquake is included in this category.

The stress criteria established in Section III of the ASME Boiler and Pressure Vessel Code, Article 4, were adopted as a guide for the design of the internals and core with exception of those fabrication techniques and materials which were combined in the most conservative way and were considered primary stresses.

The members are designed under the basic principles of:

- a) Maintaining distortions within acceptable limits
- b) Keeping the stress levels within acceptable limits
- c) Prevention of fatigue failures

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3.9.5.3.2 Design Criteria for Abnormal Operation

The abnormal design condition assumed blowdown effects were due to a reactor coolant pipe double-ended break.

For this condition the criteria for acceptability were that the reactor be capable of safe shutdown and that the engineered safety features operate as designed. Consequently, the limitations established on the internals for these types of loads were concerned principally with the maximum allowable deflections. The deflection criteria for critical structures under abnormal operation are presented in Table 3.9.5-1.

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TABLE 3.9.5-1

INTERNALS DEFLECTIONS UNDER ABNORMAL OPERATION

OF-	INCHES		
	CALCULATED DEFLECTION (PRELIMINARY)	ALLOWABLE LIMIT	NO LOSS- FUNCTION LIMIT
<u>Upper Barrel</u> , expansion/compression (to assure sufficient inlet flow area/and to prevent the barrel from touching any guide tube to avoid disturbing the RCC guide structure)	0.072	3	6
<u>Upper Package</u> , axial deflection (to maintain the control rod guide structure geometry)	0.005	1	2
<u>RCC Guide Tube</u> , cross section distortion (to avoid interference between the RCC elements and the guides)	0	0.035	0.072
<u>RCC Guide Tube</u> , deflection as a beam (to be consistent with conditions under which ability to trip has been tested)	0.2	1.0	1.5
<u>Fuel Assembly Thimbles</u> , cross section distortion (to avoid interference between the control rods and the guides)	0	0.035	0.072

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3.9.6 In-Service Testing of Pumps and Valves

The Inservice Testing (IST) Program for HBR 2 is in accordance with the applicable rules and requirements of the ASME Section XI Code, Rules for Inservice Inspection of Nuclear Power Plant Components. This program is required by 10 CFR 50, Section 50.55a(f). The IST program complies with the Edition and/or Addenda of the ASME Section XI Code specified in 10 CFR 50, Section 55a(b)(2) or as approved by the Nuclear Regulatory Commission (NRC).

The Edition(s) and/or Addenda(s) of the ASME Section XI Code applicable to the IST program are defined in site administrative procedures.

Written relief requests are granted for deviations to the ASME Section XI Inservice Testing requirements by the Commission pursuant to 10 CFR 50, Section 55a(f)(6)(i). These relief requests are identified in the IST program.

The requirements for Inservice Inspection (ISI) of Class 1 components are described in Section 5.2.4.

The requirements for Inservice Inspection (ISI) of Class 2 and 3 components are described in Section 6.6.

REFERENCES: SECTION 3.9

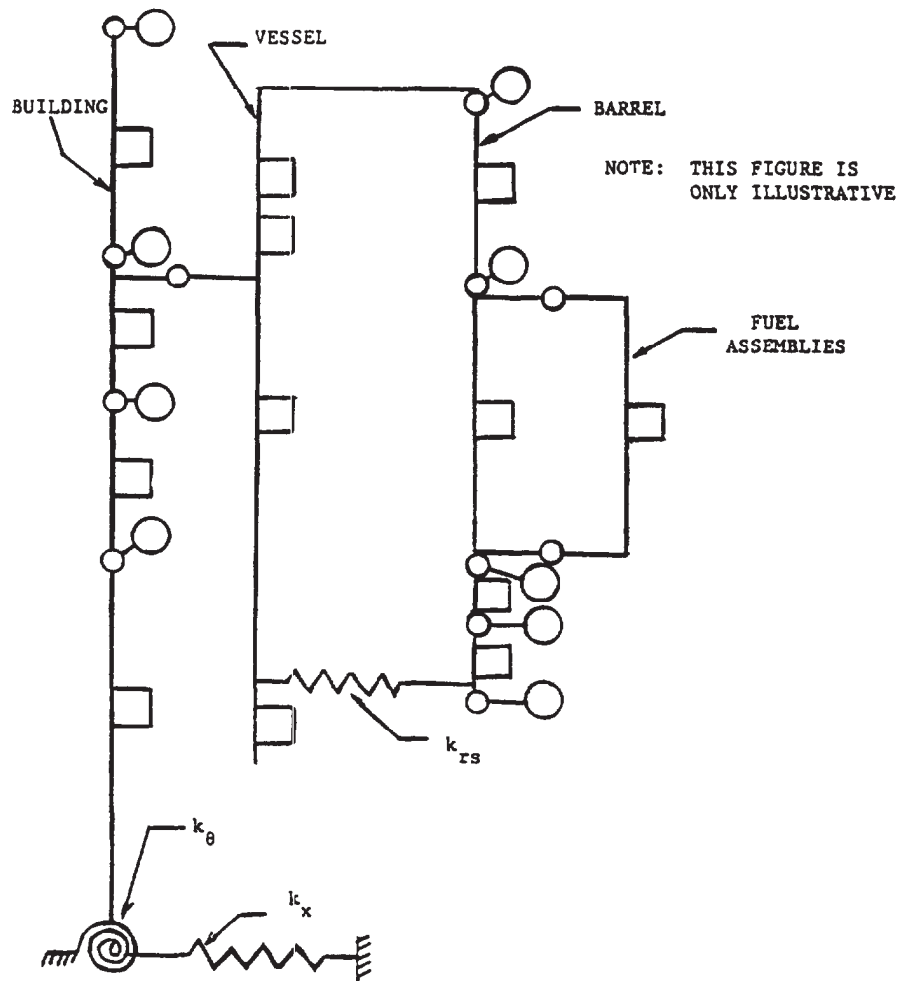
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- 3.9.3-5 License Event Report, LER 79-039, November 7, 1979.
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- 3.9.3-11 Letter, CPL-82-587, received December 20, 1982, R. S. Howard (Westinghouse) to A. B. Cutter (CP&L), Pressurizer Safety and Relief Valve Piping System - Support Loads.
- 3.9.3-12 Letter, CPL-83-531, dated March 14, 1983, R. S. Howard (Westinghouse) to A. B. Cutter (CP&L), Pressurizer Safety and Relief Valve Piping System - Support Loads.
- 3.9.3-13 "Reactor Vessel Closure Flange T-hot Transient Analysis Stress Report," Dominion Engineering, Inc., R-3521-00-1, June 2005.
- 3.9.6-1 Attachment to Letter NLS-84-008, dated January 18, 1984, S. R. Zimmerman (CP&L) to S. Varga (NRC), In-Service Inspection Program - Interval 2, March 7, 1981 to March 7, 1991.
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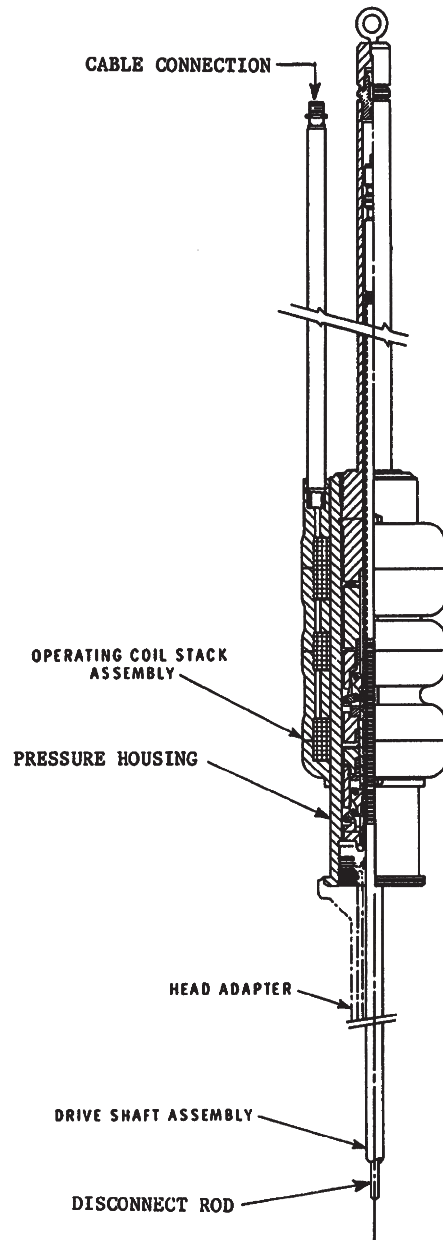
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- 3.9.6-4 NRC Letter, NLU-83-595, dated August 31, 1983, Regarding Safety Evaluation Related to Requests for Relief from Inservice Inspection Requirements.
- 3.9.6-5 NRC Letter, NRC-92-599, dated October 19, 1992, Regarding Relief Requests For Third Ten-Year Interval Inservice Inspection Program Plan.

LEGEND:

- k_{rs} = radial support spring constant
- k_{θ} = rotational ground spring constant
- k_x = translational ground spring constant
- = concentrated masses
- = distributed masses



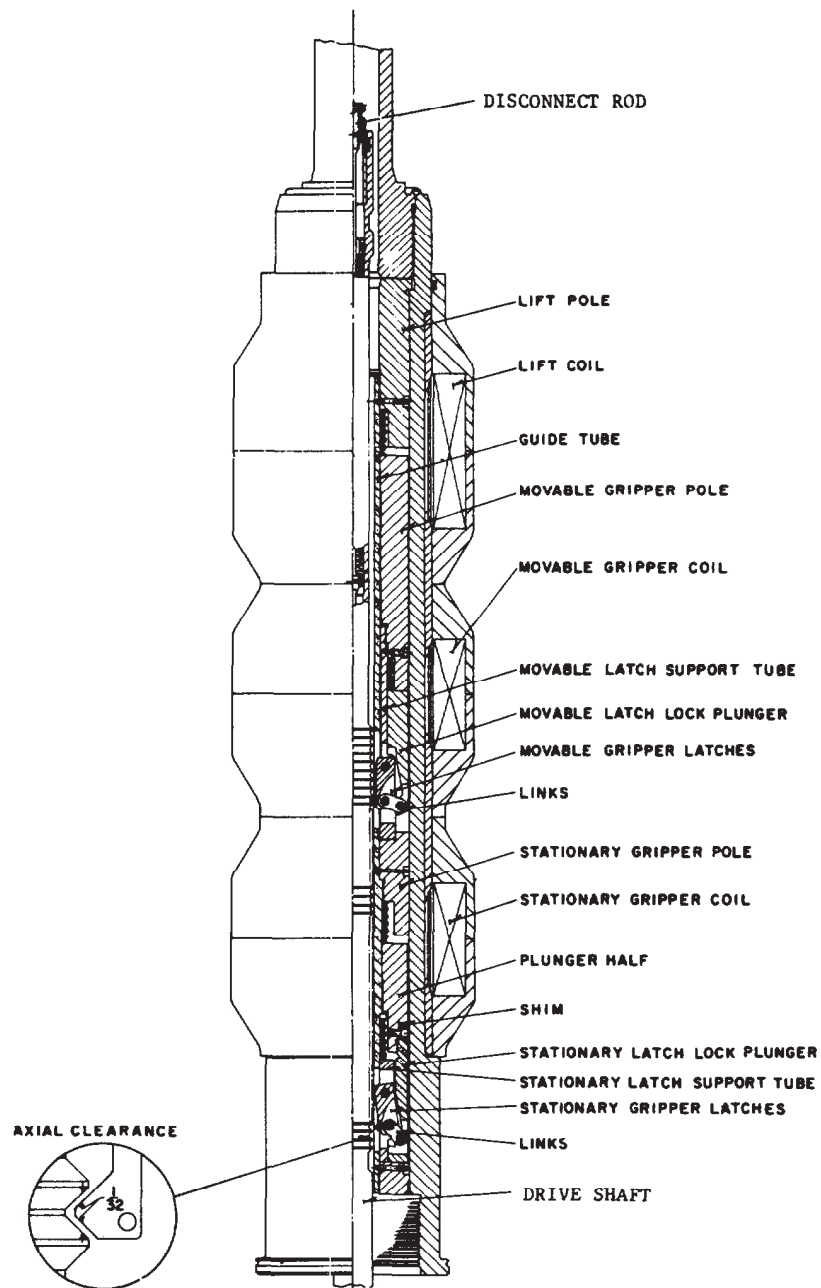


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SAFETY ANALYSIS REPORT

CONTROL ROD DRIVE MECHANISM ASSEMBLY

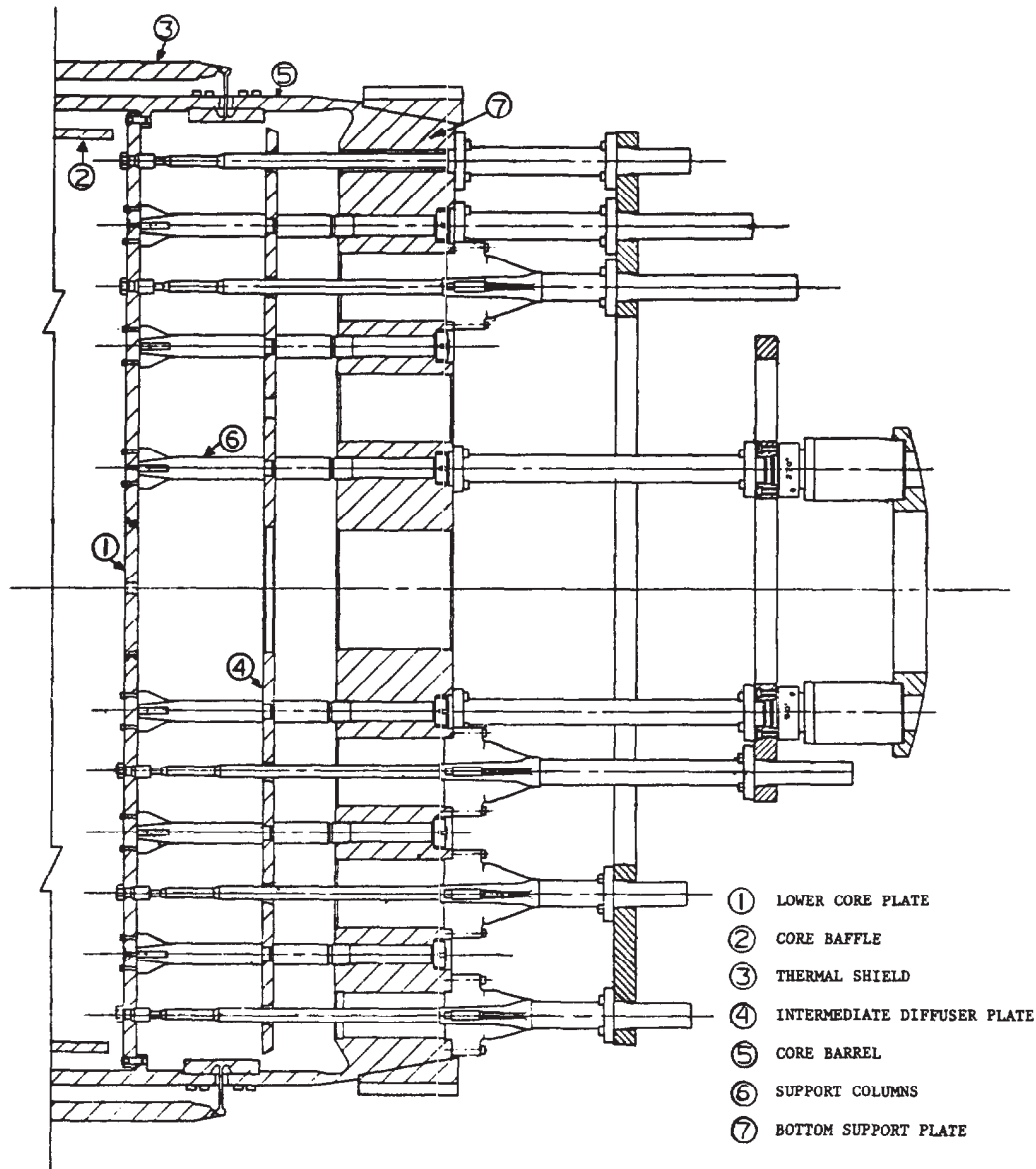
FIGURE
3.9.4 - 1

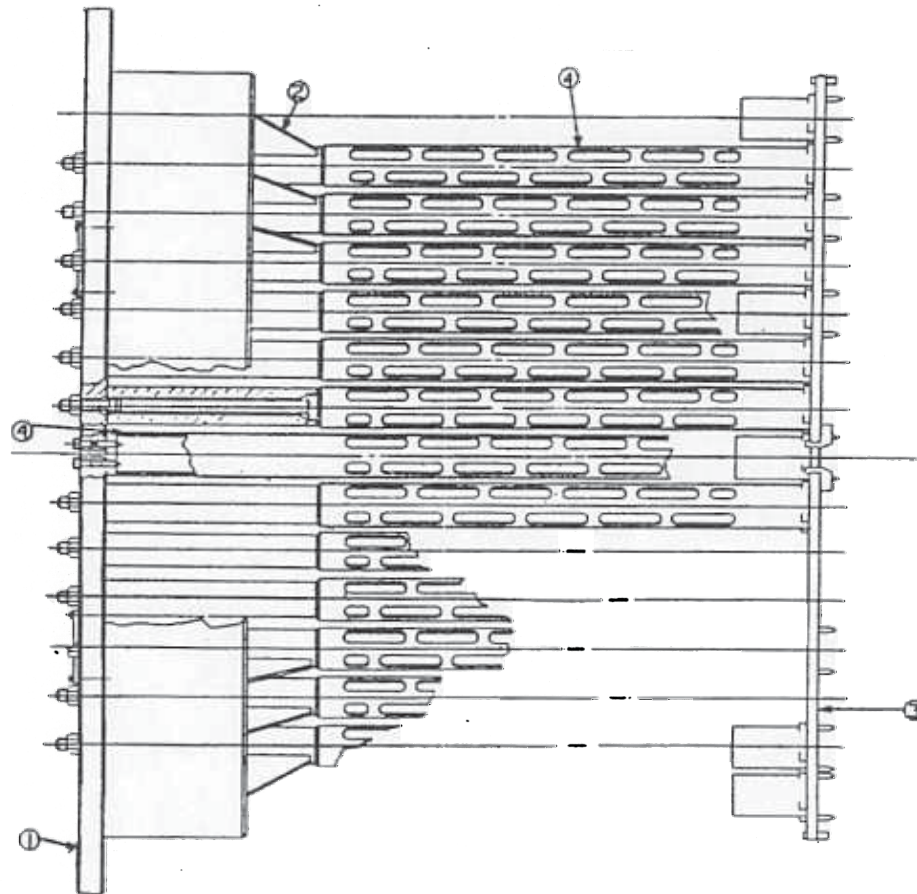


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CONTROL ROD DRIVE MECHANISM SCHEMATIC

FIGURE
3.9.4 - 2





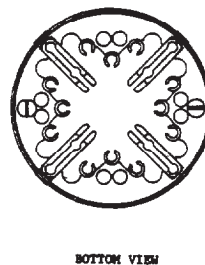
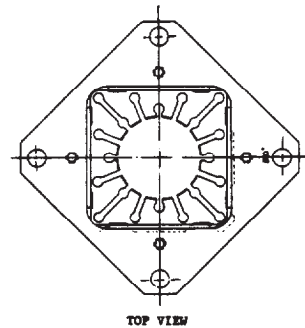
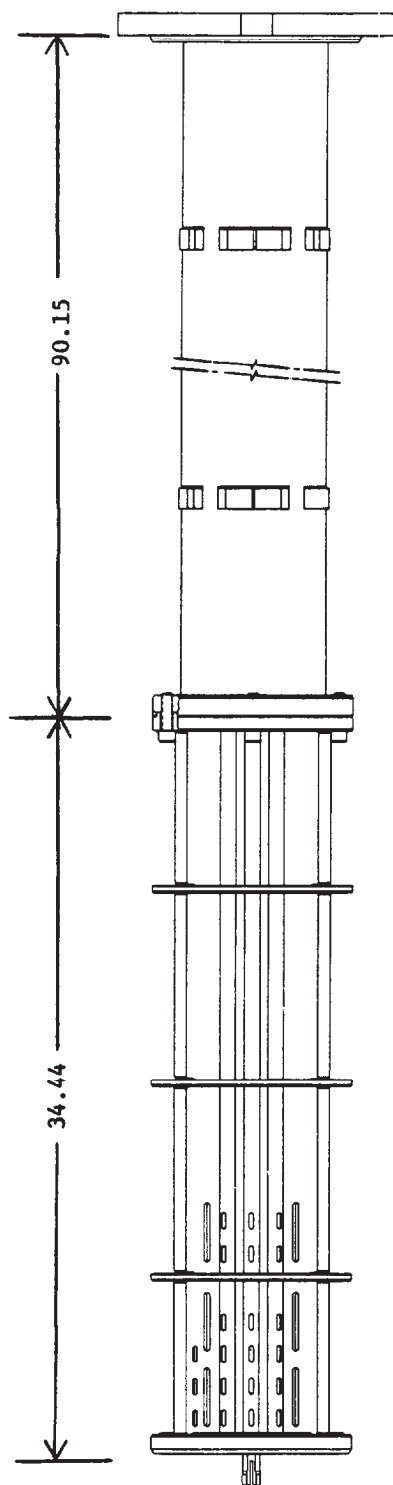
- ① TOP SUPPORT PLATE
- ② DEEP BEAM SECTIONS
- ③ UPPER CORE PLATE
- ④ SUPPORT COLUMNS

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UPPER CORE SUPPORT ASSEMBLY

FIGURE
3.9.5 - 2



3.10 SEISMIC QUALIFICATION OF SEISMIC CATEGORY I INSTRUMENTATION AND ELECTRICAL EQUIPMENT

Electrical and control equipment which initiates reactor trips and/or actuates safeguards systems must be capable of performing its functions during and after an earthquake that has occurred at the plant site. To demonstrate the ability of this equipment to perform under earthquake conditions, selected types of this essential equipment representative of all protection and safeguards circuits and equipment were subjected to vibration tests which simulated the seismic conditions for the low seismic" class of plants. The "low seismic" class consists of those plants having a Design Basis Earthquake (DBE) horizontal acceleration less than or equal to 0.2g.

During the tests, equipment operation was monitored to prove proper performance of functions. The results show that there were no electrical malfunctions for the equipment tested as described in Reference 3.10.0-1. Based on these results, it was concluded that the equipment tested will perform their design functions during, as well as following, a "low seismic" earthquake. Reference 3.10.0-1 applies to H. B. Robinson (HBR) except Section 2.2 (Process Control Equipment). Analysis of seismic test results for the HBR process control equipment indicated that there were no electrical problems.

Dynamic analyses of the buildings for the plant DBE show that the significant horizontal and vertical accelerations of the building floor where the equipment is located are within the specified "low seismic" test envelopes given in Figure B-2 of Reference 3.10.0-1.

In addition to the specific equipment listed in Reference 3.10.0-1, consideration has been given to metal-clad and metal-enclosed switchgear. To provide proper functioning of the safeguards circuits and associated equipment during and following earthquake conditions, this switchgear equipment has been specified and designed to withstand acceleration in excess of 0.2g horizontally and 0.133g vertically. This capability was a matter of procurement specification of Westinghouse and its design agents and design action of the vendors.

The safeguards circuits employ Westinghouse Models DB and DH circuit breakers and associated metal-enclosed or metal-clad switchgear. Review of these switchgear for proof of adequacy of the seismic resistance designs determined that the Model DB breakers mounted in the metal enclosures have been shock tested and proven to remain fully operable for shocks of at least 3g in any direction.

Proof of resistance of the Model DH metal-clad switchgear to a seismic response spectrum established for HBR 2 has been demonstrated by vibration testing of typical, equivalent metal-clad switchgear incorporating the Model DHP circuit breakers. The Model DH circuit breakers installed in HBR 2 are an earlier design than the Model DHP. However, the general configuration weight, distribution and vibration resistant design approach of the Model DH are essentially identical to the Model DHP. When subjected to the seismic testing spectrum, there was no loss of functions of the Model DHP metal-clad switchgear.

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Analyses of selected plant components to demonstrate their ability to withstand a seismic event are described in Section 3.7.3.3.

Unresolved Safety Issue (USI) A-46, "Seismic Qualification of Equipment in Operating Plants," was addressed in accordance with the Seismic Qualification Utility Group (SQUG) developed Generic Implementation Procedure, Revision 2 as corrected on February 14, 1992 (GIP-2). Verification of the seismic adequacy of mechanical and electrical equipment included:

- Training of Seismic Evaluation Personnel
- Identification of Safe Shutdown Equipment
- Screening Verification and Walkdown
- Outlier Identification and Resolution

Evaluations were also performed for the following:

- Relay Functionality Review
- Tanks and Heat Exchangers Review
- Cable and Conduit Raceway Review

Methodology and results of the screening evaluations and walkdowns are included as enclosures to reference 3.10.0-2.

An alternate method of seismic design and verification of modified, new, and replacement equipment (NARE) is the use of revision 3 of the Generic Implementation Procedure (GIP-3) (Reference 3.10.5-5), as modified and supplemented by the U.S. Nuclear Regulatory Commission Supplemental Safety Evaluation Report No. 2 (SSER No. 2) and (SSER No. 3) References 3.10.0-6 and 3.10.0-7). The NARE (Reference 3.10.0-8) acronym is intended to include the new and/or replacement electrical and mechanical equipment, tanks, heat exchangers, relays and electrical raceways covered in the GIP.

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REFERENCES: SECTION 3.10

- 3.10.0-1 Vogeding, E. L. "Seismic Testing of Electrical and Control Equipment", WCAP-7397-L, January, 1970.
- 3.10.0-2 CP&L letter RNP-RA/95-0123 from R. M. Krich to USNRC, "Response to Generic Letter 88-20, 'Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities,' Supplement 4 and Submittal of the Results of the Implementation of the Resolution of Unresolved Safety Issue A-46, 'Verification of Seismic Adequacy of Mechanical and Electrical Equipment in Operating Reactors'," dated June 30, 1995.
- 3.10.0-3 CP&L letter RNP-RA/95-0193 from R. M. Krich to USNRC, "Response to Request for Additional Information Seismic Qualification of Mechanical and Electrical Equipment," dated October 30, 1995.
- 3.10.0-4 CP&L letter RNP-RA/97-0091 from T. M. Wilkerson to USNRC, "Response to Request for Additional Information Regarding Seismic Qualification of Mechanical and Electrical Equipment," dated August 4, 1997.
- 3.10.0-5 Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment, Revision 3. Prepared by SQUG and sent to the NRC by letter dated July 31, 1995.
- 3.10.0-6 Supplement No. 1 to Generic Letter (GL) 87-02 that transmits Supplemental Safety Evaluation Report No. 2 (SSER No. 2) on SQUG Generic Implementation Procedure Revision 2, as corrected on February 14, 1992 (GIP-2), May 22, 1992.
- 3.10.0-7 NRC letter to SQUG dated December 4, 1997, Supplemental Safety Evaluation Report No. 3 (SSER No. 3), on the Review of Revision 3 to Generic Letter Implementation Procedure for Seismic Verification of Nuclear Power Plant Equipment, updated 5/16/97 (GIP-3).
- 3.10.0-8 Implementation Guidelines for Seismic Qualification of New and Replacement Equipment/Parts (NARE) Using Generic Implementation Procedure (GIP), Revision 4, dated July, 2000.

3.11 ENVIRONMENTAL DESIGN OF MECHANICAL AND ELECTRICAL EQUIPMENT

3.11.0 GENERAL

Equipment that is relied on to perform a necessary safety function must be demonstrated to be capable of maintaining functional operability under all service conditions postulated to occur during its installed life for the time it is required to operate. This requirement, which is embodied in General Design Criteria 1, 2, 4 and 23 of Appendix "A" and Sections III and XI of Appendix "B" to 10CFR50, is applicable to equipment located inside and outside containment. More detailed requirements and guidance relating to the methods and procedures for demonstrating this capability have been set forth in 10CFR50.49 (Reference 3.11.0-1). In addition, an on-going surveillance/maintenance and replacement program has been established to maintain environmental qualification of required equipment for the remainder of the plant life.

10CFR50.49 requires that each holder of a license to operate a nuclear power plant establish a program for qualifying electric equipment important to safety. The environmental qualification of electrical equipment at H. B. Robinson - Unit 2 (HBR-2) was established in response to I. E. Bulletin 79-01B (Reference 3.11.0-2). The qualification basis was taken as the Division of Operating Reactors (DOR) Guidelines. For certain equipment purchased as a result of NUREG-0696 or NUREG-0737, the requirements of NUREG-0588 Category I were applied. The issuance of 10CFR50.49 does not require re-qualification of equipment to the provisions of 10CFR50.49 if the equipment was previously required to be qualified to these earlier guidelines. Replacement and new equipment must be qualified to the requirements of 10CFR50.49 unless there are sound reasons to the contrary as stated in Regulatory Guide 1.89.

The purpose of this section is to provide information on the environmental conditions and design bases for which safety related electrical and mechanical equipment is designed to ensure compliance with the above. In addition, this section describes the applicants' environmental qualification program and methodology for compliance with DOR Guidelines and therefore 10CFR50.49.

3.11.1 EQUIPMENT IDENTIFICATION AND ENVIRONMENTAL CONDITIONS

The methodology to determine which equipment is important to safety is to be environmentally qualified is based on the IE Bulletin 79-01B approach of reviewing plant systems which perform safety functions. The equipment within such systems, which are necessary for the performance of the safety functions, are identified (in the master list) and qualified environmentally to demonstrate acceptable performance throughout its installed life.

The master list submitted with the HBR-2 10CFR50.49 response was based on previous submittals, reviews of the FSAR, Technical Specifications, etc. To ensure that the rationale for the equipment's inclusion or exclusion from the list is available, a detailed review of the Chapter 15 events for HBR-2 has been performed and the results reported as the EQ Master Equipment List (EQMEL).

The EQMEL includes the minimum set of electrical components required by 10CFR50.49. It includes:

- safety-related equipment which is relied upon to ensure: 1) the integrity of the reactor coolant pressure boundary, 2) the capability to achieve and maintain safe shutdown conditions, and 3) the capability to limit offsite doses within 10CFR100 guidelines.
- non-safety equipment whose failure could jeopardize a safety function described above, and
- Regulatory Guide 1.97 (Reference 3.11.1-1) and certain NUREG-0737 components

The temperature and pressure conditions inside the containment resulting from a design basis accident (DBA) are a function of time until steady state conditions are established. The HBR-2 postulated temperature and pressure profiles for the containment are shown in Figures 3.11.1-1 and 3.11.1-2, respectively.

The HBR-2 containment lower level consists of a reactor vessel sump area and compartmented base floor. The floor level elevation is at 228 feet. The anticipated volume of water available to flood the containment during an accident is 451,000 gallons. This is comprised of Refueling Water Storage Tank, Accumulators, Spray Addition Storage Tank, and Reactor Coolant Loop water volumes emptied within the containment (Reference 3.11.0-2). This produces a floor flood level of approximately 6.09 feet (6 feet, 1 inch) or a flood elevation of 234.09 feet within containment. All equipment identified in the HBR-2 EQDPs have been analyzed to determine the impact of submergence.

3.11.2 QUALIFICATION TESTS AND ANALYSIS

Environmental qualification testing and/or analysis based on tests are performed on safety related equipment located in a harsh environment. The results are evaluated for compliance with the DOR Guidelines.

Specifically, all reviews consider but are not limited to the following:

- a) Assurance that the test report is applicable to HBR-2. This is accomplished by assuring that the project name, purchase order, and equipment specification as a minimum are identified on or are traceable to the report.
- b) A comparison of the test sample is made to assure that the equipment tested is identical to or representative of the purchased equipment.
- c) The aging (temperature, radiation, humidity, electro-mechanical cycling, etc., as required) simulation is evaluated to determine if the test equipment has been subjected to an environment which simulates its expected end-of-qualified-life environment prior to the design basis accident testing. Process temperatures, when applicable, are addressed.
- d) The design basis accident environments (temperature, pressure, humidity, chemical spray radiation, etc.) are evaluated to determine if they envelope the HBR-2 postulated environments in the unlikely event of a design basis accident.
- e) Anomalies observed during qualification testing are evaluated.

In addition, other items such as test sequence, margin, and interfaces are also addressed during the environmental qualification report review process.

Compliance with the NRC Regulatory Guides and General Design Criteria is described in UFSAR Sections 1.8 and 3.1, respectively.

Pages 3.11.2-2 through 3.11.2-6 were deleted by Amendment No. 10.

3.11.3 QUALIFICATION TESTS RESULTS

A summary of the harsh environment qualification test results for each type of qualified safety related equipment is provided in the HBR-2 EQDPs. The EQDP substantiates the qualification in detail.

Typical documents which are included in the EQDP are:

- a) List of Affected Equipment
- b) Qualification Analysis which provide the following:
 - Qualification Summary
 - Qualification Requirements (i.e., Normal and Accident Environmental and Functional Parameters)
 - Qualification Documentation Assessment
 - Qualification Justification Analysis
- c) Responses to Applicable NRC/IE Bulletins and Notices
- d) Identification of Required Maintenance to Maintain Qualification
- e) General Information (i.e., drawings, memos, vendor supplied documents, etc.)

A central file documenting the EQ of electrical equipment is maintained and physically stored as a permanent QA record (i.e., an auditable file) on site. The EQ Central File consists of a reference file, a list of equipment requiring EQ, the EQMEL, and a set of EQDPs. The EQDP provides auditable proof of the equipment's ability to perform required safety functions under normal and postulated accident conditions. This is achieved by comparison between the HBR-2 plant specific requirements and the environmental qualification test parameters.

3.11.4 Loss Of Ventilation

The effects of a loss of ventilation on plant operation is discussed in Section 9.4.1.2.

3.11.5 ESTIMATED CHEMICAL AND RADIATION ENVIRONMENT

3.11.5.1 CHEMICAL ENVIRONMENT

Safety related systems are designed to perform their safety related functions in the temperature, pressure, and humidity environments discussed in Section 3.11.1 and in Section 6.2. In addition, components of the ESF systems inside the containment are designed to perform their safety related functions in a long-term contact with boric acid and sodium hydroxide solutions, recirculated through the Safety Injection System (SIS) and the Containment Spray System (CSS).

The pH time history of the water both in the containment spray and in the containment sump, as well as the boron concentration in the Reactor Coolant System, is discussed in Sections 6.1.1.2 and 6.5.2.

The CVCS, SIS, and CSS are designed for both the maximum and long-term boron concentration of 2000 - 2200 ppm at a pH of 8.5 to 11.0.

3.11.5.2 RADIATION ENVIRONMENT

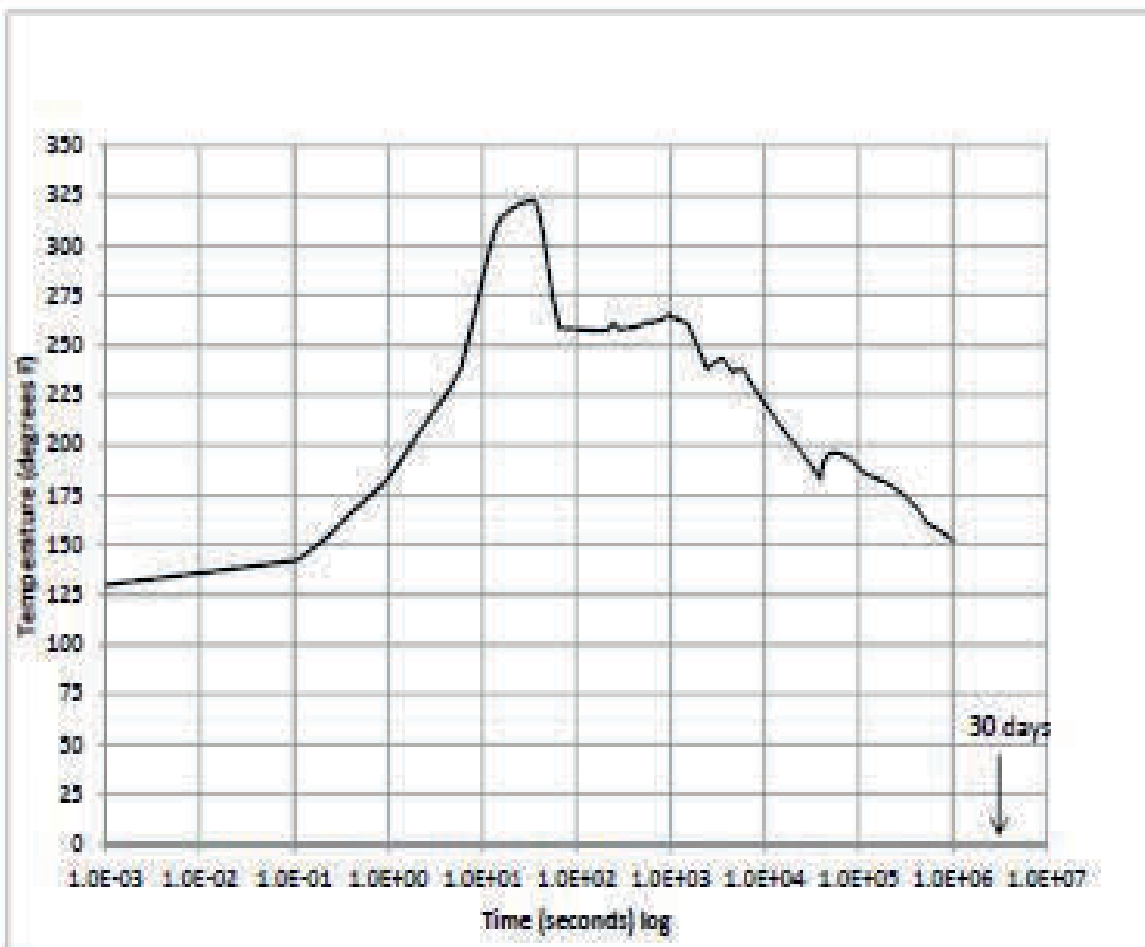
Safety related systems and components are designed to perform their safety related functions after the normal operational exposure plus one accident exposure. The normal operation exposure is based on the design source terms presented in Section 11.1 and Section 12.2. Post-accident system and component radiation exposures are dependent on equipment location. Source terms and other accident parameters are presented in Section 12.2 and Chapter 15.

The design radiation exposures are based on gamma and beta radiation. The effects of beta radiation are effectively attenuated by small amounts of shielding, such as conduits for cable and enclosures for equipment. Organic materials which are located inside the containment are identified in Section 6.1.2.

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REFERENCES: SECTION 3.11

- 3.11.0-1 "Response to Amended Regulation on Environmental Qualification of Electrical Equipment Important to Safety 10CFR50.49 (2-22-83) May, 1983," submitted with letter LAP-83-185, E. E. Utley (CP&L) to S. A. Varga (NRC) dated May 20, 1983.
- 3.11.0-2 "Environmental Qualification of Electrical Equipment," H. B. Robinson E. G. Plant - Unit 2, NRC I. E. Bulletin 79-01B (90-Day Report), Rev. 3 dated February 1, 1981.
- 3.11.0-3 "Environmental Qualification," E. E. Utley (CP&L) to S. A. Varga (NRC), dated July 31, 1981, NO-81-1285.
- 3.11.0-4 "Environmental Qualification of Safety-Related Electrical Equipment," E. E. Utley (CP&L) to S. A. Varga (NRC), dated August 31, 1981, NO-81-1432.
- 3.11.0-5 "Environmental Qualification of Safety-Related Electrical Equipment," S.R. Zimmerman (CP&L) to S. A. Varga (NRC), dated July 1, 1983, LAP-83-271.
- 3.11.0-6 "Resolution of Safety Evaluation Reports for Environmental Qualification of Safety-Related Electrical Equipment," A. B. Cutter (CP&L) to S. A. Varga (NRC), dated March 2, 1984, NLS-84-083.
- 3.11.1-1 "Instrumentation for Light Water-Cooled Nuclear Power Plants to Assess Plant and Environs Conditions During and Following an Accident," CP&L Letter NLS-87-136, Rev. 4 to Regulatory Guide 1.97 Submittal dated October 9, 1987.

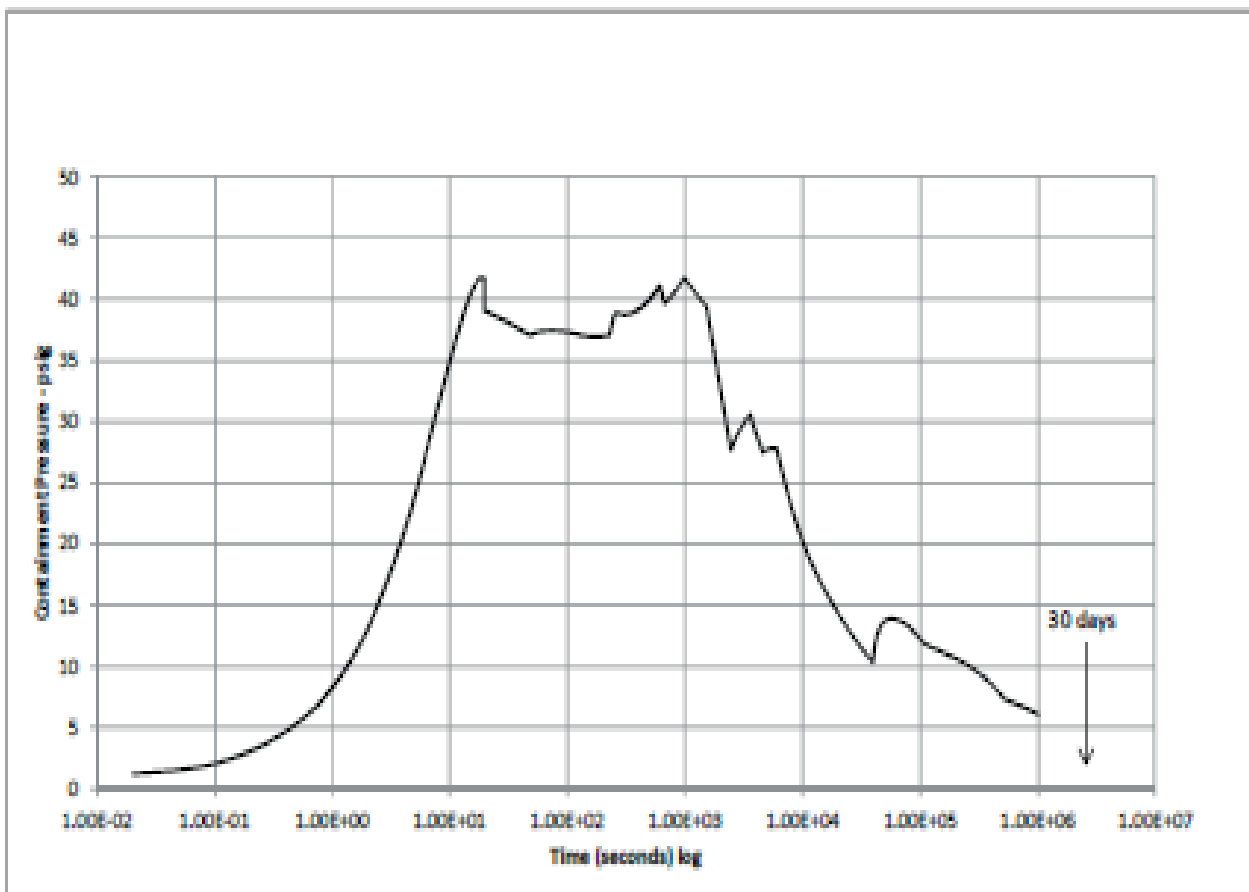


REVISION NO. 25

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ANALYSIS REPORT

ENVIRONMENTAL
CONDITIONS FOR
EQUIPMENT TESTING
TEMPERATURE VS. TIME

FIGURE
3.11.1-1



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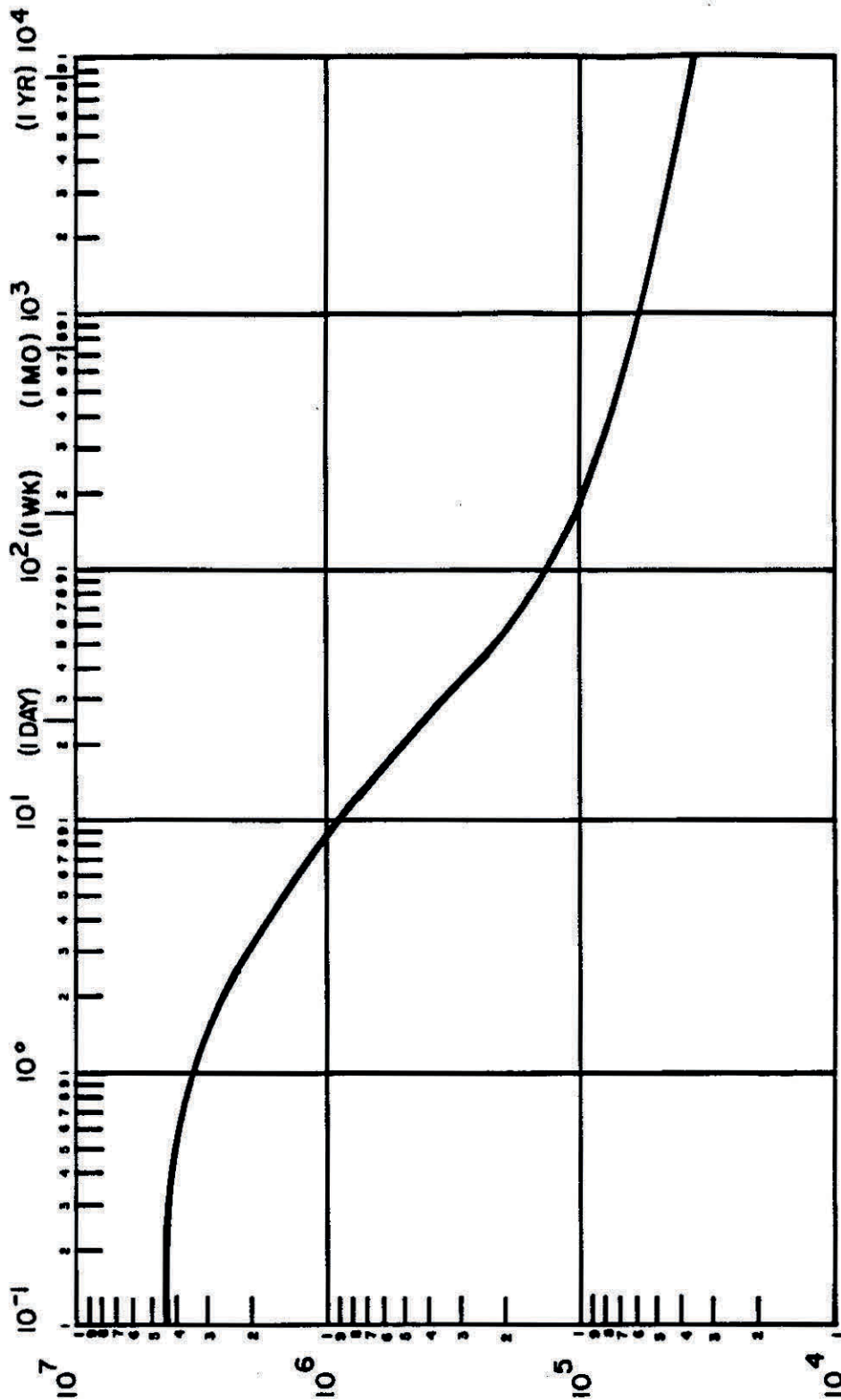
ENVIRONMENTAL
CONDITIONS FOR
EQUIPMENT TESTING
PRESSURE VS. TIME

FIGURE
3.11.1-2

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Figures 3.11.2-1, 3.11.2-2, 3.11.2-3 and 3.11.2-4 were deleted by Amendment No. 10

TIME AFTER RELEASE - HR.



DIRECT GAMMA DOSE RATE - RAD. / HR.

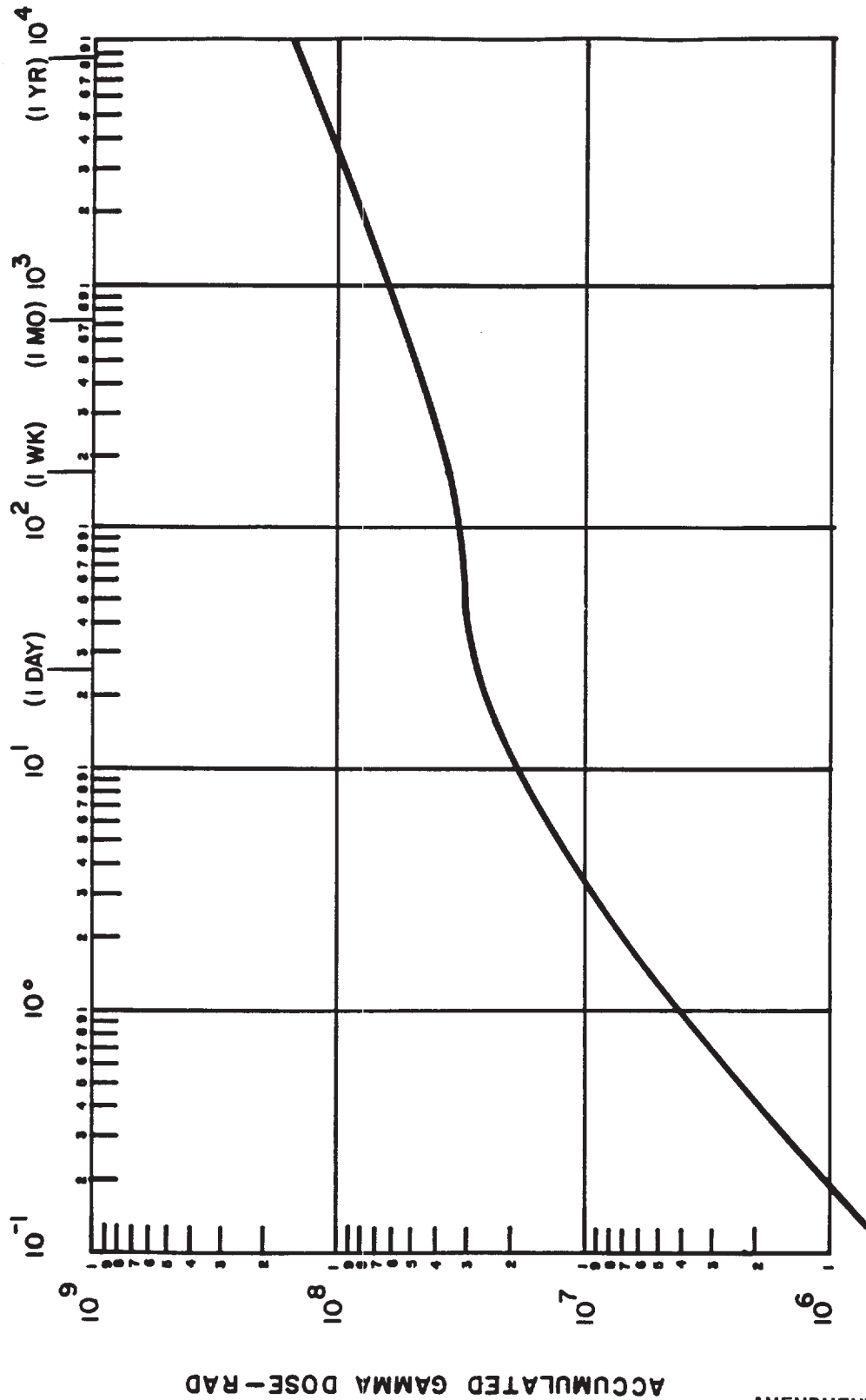
AMENDMENT NO. 10

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INSTANTANEOUS GAMMA DOSE RATE
INSIDE CONTAINMENT AS A FUNCTION
OF TIME AFTER RELEASE

FIGURE
3.11.5-1

TIME AFTER RELEASE - HR.



ACCUMULATED GAMMA DOSE - RAD

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INTEGRATED GAMMA DOSE LEVEL INSIDE
CONTAINMENT AS A FUNCTION OF
TIME AFTER RELEASE

FIGURE
3.11.5-2