

XII - STRUCTURES AND SHIELDING

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XII - STRUCTURES AND SHIELDING1.0 SUMMARY DESCRIPTION

The principal structures of the station consist of the Reactor Building, Turbine Building (including service area appendages), Control Building, Controlled Corridor, Radwaste Building, Augmented Radwaste Building, Intake Structure, Off-Gas Filter Building, Elevated Release Point, Diesel Generator Building, Multi-Purpose Facility, Railroad Airlock, Drywell and Suppression Chamber, Miscellaneous Circulating Water System Structures (e.g. Circulating Water Conduits, Seal Well, etc.), Optimum Water Chemistry Gas Generator Building and Office Building. These structures are founded upon suitable material for their intended application and are designed as a minimum to be within the limits of applicable codes as identified in Chapter I. In addition, critical structures are designed to withstand more severe loading conditions than normally considered in conventional design practice.

Location and orientation of the buildings on the site are shown in Burns and Roe Drawing 4003. The general arrangement of the structures is shown in Burns and Roe Drawings 2050, 2051, 2052, 2053, 2054, 2056, 2059, 2060, 2061, 2062, 2063, 2064, 2065, 2066, 2067, 2068, 2069, 2072, and 2073.

Shielding and access control are provided for radiation protection which is in accordance with the limits and guidelines of appropriate regulations.

2.0 STRUCTURAL DESIGN

2.1 Classification of Structures and Equipment

2.1.1 General

The two classes of structures and equipment applicable to the structural design requirements are as follows:

Class I - This class includes those structures, equipment, and components whose failure or malfunction might cause or increase the severity of an accident which would endanger the public health and safety. This category includes those structures, equipment, and components required for safe shutdown and isolation of the reactor.

Class II - This class includes those structures, equipment, and components which are important to reactor operation, but are not essential for preventing an accident which would endanger the public health and safety, and are not required for the mitigation of the consequences of these accidents. A Class II designated item shall not degrade the integrity of any item designated Class I.

2.1.2 Class I Structures and Equipment

Delineated below are CNS Class I structures and equipment (some of which are non-safety related).

2.1.2.1 Principal Class I Structures Required for Safe Shutdown

Reactor Building
Control Building
Intake Structure (except steel superstructure)
Diesel Generator Building
Controlled Corridor
Drywell and Suppression Chamber

2.1.2.2 Principal Class I Structures Not Required for Safe Shutdown

Elevated Release Point
Radwaste Building (below grade)

2.1.2.3 Class I Equipment

Nuclear Steam Supply System
 Reactor Pressure Vessel
 Reactor Pressure Vessel Supports
 Control Rods and Drive System including equipment necessary
 for scram operation
 Control Rod Drive Housing Supports
 Fuel Assemblies
 Core Shroud
 Core Supports
 Steam Separator
 Steam Dryer
Reactor Recirculation System piping including valves and pumps
Piping connections from the Reactor Pressure Vessel up to and
 including the first isolation valve external to the Drywell

USAR

Reactor Core Cooling and Station Standby Systems
 High Pressure Coolant Injection System (including Emergency
 Condensate Storage Tanks)
 Reactor Building Floor Drain Sump Pumps
 Reactor Core Isolation Cooling System
 Standby Liquid Control System
 Core Spray System
 Reactor Equipment Cooling System Critical Loop Piping
 Residual Heat Removal System and its associated Service Water
 System
 Radwaste Storage Tanks
 Reactor Water Cleanup Phase Separators
Portion of Instrument and Service Air Systems associated with
 essential systems^[1]
Station Standby Gas Treatment System and the supporting Sump Z
 System (Refer to USAR Section XII-2.3.5.1.9)
Portion of Station Service Water System associated with essential
 systems
Fuel Storage Facilities, to include spent fuel and new fuel storage
 equipment (excludes ISFSI)
Standby Electrical Power Systems
 Station Battery System
 Standby Diesel Generator System and Auxiliaries
 Emergency Buses and other electrical gear and power to
 critical equipment
Instrumentation and controls required for operation of Class I
 equipment

2.1.3 Class II Structures and Equipment

The following structures and equipment have been designated as
Class II.

2.1.3.1 Principal Class II Structures

Turbine Building (including service area appendages)
Off-Gas Filter Building
Office Building
Circulating Water System Structures (except Class I portion of
 Intake Structure)
Multi-Purpose Facility
Augmented Radwaste Building
Radwaste Building (above grade)
Railroad Airlock
OWCGG Building

An evaluation of the Turbine Building concrete structure was performed to confirm that it is capable of remaining structurally intact without gross structural failure following a postulated SSE. This allows crediting the dose consequence mitigation assumptions related to leakage holdup and the resulting iodine plateout within the Main Condenser following a postulated Loss-of-Coolant Accident. (Refer to USAR Section XII-2.2.2).

Additionally, structures which support the Sump Z power supply (Machine Shops, Heating Boiler and Exhaust Fan Rooms, Water Treatment Area, and Off-Gas Filter Building), although designated as Class II, have been

evaluated for the Class I Operating Basis Earthquake loading requirements (Refer to USAR Section XII-2.3.5.1.9).

2.1.3.2 Class II Equipment

Turbine Generator System
Main Condenser System
Circulating Water System
Reactor Feedwater and Condensate Systems
Station Auxiliary Power Buses
Electrical controls and instrumentation (For Class II Systems)
Reactor Water Cleanup Demineralizer System, except Phase Separators
Augmented Off-Gas Treatment System
Radwaste System, except storage tanks
Turbine System Moisture Separators
Startup and Auxiliary Transformers
Condensate Makeup Demineralizer System
Other piping and equipment not listed under Class I
Optimum Water Chemistry System

2.2 Description of Principal Structures

2.2.1 Reactor Building

The Reactor Building (refer to Burns and Roe Drawings 2059, 2060, 2061, 2062, 2063, 2064, 2065, and 2066) encloses the Nuclear Steam Supply System and Reactor auxiliary and cooling systems, including the Spent Fuel Storage Pool. The building provides secondary containment for the Reactor and primary containment for auxiliary systems and refueling operations, as required (see Section V-3). The Reactor Building also houses the Primary Containment which is a radioactive material barrier consisting of the Drywell, in which the Reactor Pressure Vessel is located, the Suppression Chamber, and process lines out to the first isolation valve outside the containment wall.

The Reactor Building is a reinforced concrete structure with structural steel framing above the refueling floor. All interior walls are reinforced concrete. The Reactor Cavity, Dryer-Separator Pit, and Spent Fuel Pool are lined with stainless steel. The Reactor Pedestal is reinforced concrete and supports both the Reactor Pressure Vessel and the Sacrificial Shield Wall. The structural steel framing above the refueling floor is enclosed with insulated metal siding. This siding is designed to blow off under tornado pressures (refer to USAR Section XII-2.3.3.2.4). Heavy diagonal bracing is used between columns to resist lateral loads. The roof is an insulated metal deck supported by structural steel framing.

There are no rigid structural elements connecting the Reactor Building to other buildings with different dynamic characteristics. All major structures have been separated by gaps. Total movements within the gap separations vary between 1.06 and 3.95 inches. Actual gap spaces of two to four inches are provided to accommodate these movements.^[2] There are some flexible component supports bridging the gaps between the Reactor Building and other buildings which were evaluated and found to have no adverse impact on the building analysis (see USAR Sections XII-2.3.5.1.3).^[55]

There is an air gap between the Drywell and the Drywell Biological Shield Wall that serves the following function:

1. The gap allows the Drywell to act as an independent pressure vessel.
2. The gap dimension has been set so that the Drywell Biological Shield Wall provides backup to the Drywell so it will not fail if deformed by jet forces when the jet loading condition is imposed.
3. The gap allows the Drywell Biological Shield Wall to be an independent structure without secondary loading from the Drywell, except where required from #2 above.^[3]

The drainage of the two inch gap between the Drywell and the Drywell Biological Shield Wall is provided by eight, four inch diameter drain pipes at the sand transition zone where the Drywell is supported by the concrete. These drains are equally spaced around the periphery of the Drywell.

Venting to avoid possible pressurization of the gap is provided by the annular space between embedded sleeves and Drywell penetrations which pass through these sleeves. All embedded sleeves are sized at least four inches larger in diameter than the penetrations and this space extends to the two inch gap around the Drywell. This vent area is available at all sleeves for piping penetrations, the Drywell personnel airlock, the two Drywell equipment hatches and the large vent pipes connecting the Drywell to the Suppression Chamber.^[4]

Personnel airlocks into the Reactor Building (see Burns and Roe Drawing 2060) have been designed in accordance with loading requirements for Class I Structures. Where a possibility of a missile entering the building exists, missile barriers have been provided.

The personnel airlocks have been provided with gasketed self-closing hollow metal doors at the interior and exterior ends.

The doors and accessories have been designed to resist the forces created during a loss of coolant accident when the building is pressurized to positive (+) 7 inches wg.^[8] Note that the Southwest personnel airlock at Elevation 903'-6" is barricaded.^[54]

The Spent Fuel Pool contains the radioactive spent fuel assemblies, control, and instrument rods, and other non-fuel irradiated components originating from nuclear system equipment. A maximum of five feet of concrete thickness is used for radiation protection at the sides and six feet at the bottom of the storage pool. A minimum cover of water above the fuel assemblies will be maintained for shielding of plant personnel during fuel storage and transfer operations. This minimum level is specified in the Technical Specifications.

The ends of the north-south pool walls are supported at the north end by the Drywell Biological Shield Wall and at the south end by the exterior concrete walls of the Reactor Building. These walls are designed as deep beams carrying the dead and live loads of the refueling floor in addition to the loads from the Spent Fuel Pool. The Spent Fuel Pool slab is designed as a two-way slab supported by the Drywell Biological Shield Wall and the enclosing fuel pool walls. Included in the design is the effect of thermal gradients through the walls due to the presence of hot water in the pools.^[5]

The Reactor Building and Primary Containment structures are in scope for License Renewal per 10 CFR 54.4(a)(1), (a)(2), and (a)(3) and were subject to aging management review. Aging effects are managed by the following Aging Management Programs: Containment Inservice Inspection (see USAR Section K-2.1.10), Containment Leak Rate (see USAR Section K-2.1.11), Fire Protection (see USAR Section K-2.1.16), Inservice Inspection - IWF (see USAR Section K-2.1.20), Masonry Wall (see USAR Section K-2.1.21), Periodic Surveillance and Preventive Maintenance (see USAR Section K-2.1.31), Structures Monitoring (see USAR Section K-2.1.36), and Water Chemistry Control - BWR (see USAR Section K-2.1.39). The following Time-Limited Aging Analyses are applicable: Metal Fatigue (see USAR Section K-2.2.2.2).

2.2.2 Turbine Building

The Turbine Building (refer to Burns and Roe Drawings 2050, 2051, 2052, 2053, and 2054) houses the turbine-generator and associated auxiliaries including the Condensing, Feedwater, and Water Treatment Systems. Space is provided in this building for other auxiliary power plant equipment. The Water Treatment Area, Machine Shop, Exhaust Fan Room and Heating Boiler Room are located adjacent to the Turbine Building and are referred to as Turbine Building appendages.

The Turbine Building is a reinforced concrete structure up to the operating floor. Concrete shield walls surround the turbine-generator and structural steel framing rises above the operating floor. The building is enclosed with insulated metal siding and roofing. The interior walls are reinforced concrete with concrete block enclosing smaller areas. The Turbine Pedestal is a massive reinforced concrete structure supported by the same foundation mat as the building.

An evaluation of the Turbine Building concrete structure was performed to confirm that it is capable of remaining structurally intact without gross structural failure following a postulated SSE. This allows crediting the dose consequence mitigation assumptions related to leakage holdup and the resulting iodine plateout within the Main Condenser following a postulated Loss-of-Coolant Accident. Samples of key Turbine Building substructures (e.g., walls, floor slabs, and columns) were evaluated for increased seismic loading resulting from a postulated SSE. The horizontal seismic acceleration input to the operating floor of the Turbine Building at elevation 932 ft-6 in. due to the Turbine Building response was assumed to be 0.3g based on a comparison with Class I structures (Reactor Building and Control Building). The evaluations show that the increase in design loadings from the original seismic Class II criteria to the postulated SSE condition do not result in stresses that exceed the allowable limits applicable to the SSE load case. Therefore, it is concluded that there is sufficient margin in the original design to ensure that the concrete portion of the Turbine Building structure will remain intact during and following an SSE. These results are based primarily on the fact that allowable stresses are increased for the SSE load case and, consequently, the increase in seismic loading is offset by the increase in allowable stresses.

The Turbine Building is in scope for License Renewal per 10 CFR 54.4(a)(1), (a)(2), and (a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), Masonry Wall (see USAR Section K-2.1.21), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.3 Control Building

The Control Building (refer to Burns and Roe Drawings 2050, 2051, and 2052) houses instrumentation and switches required for station operation.

Also located in this building is a computer room, station batteries, and components of the Reactor Protection System. The control building is a reinforced concrete structure.

The Control Building is in scope for License Renewal per 10 CFR 54.4(a)(1), (a)(2), and (a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), Masonry Wall (see USAR Section K-2.1.21), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.4 Radwaste Building

The Radwaste Building (refer to Burns and Roe Drawings 2067, 2068, and 2069) houses the various components of the Radwaste System, as well as the control center for the Radwaste System. The building is a reinforced concrete structure.

The Radwaste Building is in scope for License Renewal per 10 CFR 54.4(a)(1) and (a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), Masonry Wall (see USAR Section K-2.1.21), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.5 Diesel Generator Building

The Diesel Generator Building (refer to Burns and Roe Drawings 2051 and 2052) houses two complete diesel generators with associated equipment including air filters, silencers, exhaust stack, and all necessary electrical equipment.

The Diesel Generator Building is a reinforced concrete structure. The building foundation consists of wall footings which are separated from the diesel generator foundations.

The Diesel Generator Building is in scope for License Renewal per 10 CFR 54.4(a)(1), (a)(2), and (a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.6 Elevated Release Point

The Elevated Release Point discharges gases to the atmosphere from portions of the Reactor Building, Standby Gas Treatment System, and the Off-Gas System. It is designed as a free standing steel tower. The Elevated Release Point is provided with appurtenances such as aviation obstruction lights and radiation monitoring instruments. The Elevated Release Point is located 350 feet southeast of the Reactor Building. The tower is 325 feet above grade with top at Elevation 1,215. The tower is designed in accordance with Class I criteria and includes a dynamic analysis. This dynamic analysis determined that the wind load case bounds the applicable seismic loadings. Note that the Elevated Release Point is not designed for tornado loading since it is not required for Safe Shutdown and its failure would not cause an accident (see USAR Section XII-2.3.3.2.1).

The Elevated Release Point is in scope for License Renewal per 10 CFR 54.4(a)(1) and (a)(2) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.7 Circulating Water System Structures

The Circulating Water System has the following configuration:

Circulating water is drawn into the Intake Structure from the Missouri River. The Intake Structure is located at the river bank and houses four, one-quarter capacity, Circulating Water pumps, four Service Water pumps, a Fire Protection System pump, and other associated accessories. Circulating water is carried by large diameter pipes into the Turbine Building and is distributed to the inlet water boxes of the main condensers. Water discharges from the condensers in a tunnel in a southerly direction to the Seal Well structure located south of the Turbine Building.

2.2.7.1 Intake Structure

The Intake Structure (refer to Burns and Roe Drawings 2050 and 2056) is located at the river bank. The substructure is a reinforced concrete structure and is designated Class I. The operating floor in the area of the Service Water pumps and fire pump is enclosed by a reinforced concrete structure which is also designated Class I. This concrete superstructure and the remainder of the operating floor is enclosed by a Class II steel framed superstructure. The steel superstructure is enclosed by metal siding which is designed to blow off during a tornado (see USAR Section XII-2.3.3.2.4).

The operating floor of the structure, on which the Circulating Water pumps, trash racks, traveling screens, and Service Water pumps are mounted is at Elevation 903.5 feet. The river bed elevation in the navigation channel to the east of the Intake Structure is nominally 860 feet MSL. Normal Summer water level is at Elevation 880.

The intake has eight screen bays at nine feet, eight inches wide inside serving the four Circulating Water pumps, and one screen bay serving the Service Water pumps. The Circulating Water pumps discharge into a concrete tunnel. A valve and expansion joint are provided between the discharge of each pump and the concrete tunnel.

A ten foot deep concrete skirt, plus sheet piling down to bedrock, is installed along the river face as an integral part of the bottom slab. The concrete skirt returns along the north and south walls in a manner which ensures that the skirt will have ten feet of cover below the slope of the bank. Further rip-rap is placed in front of the intake structure along the floor of the forebay and along the adjoining banks to supplement the skirt and sheet piling in providing scour protection. Rip-rap is extended up the river slope of the plant fill to prevent scouring from both high water and ice.

An Ice Deflector is placed in the Missouri River to direct float ice away from the Intake Structure. The Ice Deflector is a non-essential component which enhances plant operations. The presence of the Ice Deflector is scheduled around the Missouri River navigational season.^[6]

The Intake Structure is in scope for License Renewal per 10 CFR 54.4(a)(1), (a)(2), and (a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.7.2 Intake Structure Guide Wall^[7]

A guide wall is located in front of the Intake Structure to reduce sediment buildup in the Intake Structure. A detailed engineering evaluation, environmental evaluation, and a model study has been performed.

The guide wall is physically attached at its upstream end to the circular sheet pile cell which was left in place after the remaining cofferdam structure was removed. It continues downstream starting at a slight divergent angle with the channel line along a flat arc to a point even with the upstream corner of the Intake Structure. From there it continues downstream parallel with the Intake Structure at a distance of approximately 15' riverward therefrom. The downstream end is approximately 40 feet past the downstream corner of the Intake Structure. No connection is made to the shore at the downstream end. The wall is constructed of steel sheet piling with the section upstream of the Intake Structure designed to contain fill by incorporating a tieback system. Downstream of the upstream corner of the Intake Structure the wall is designed as a cantilever. Rip-rap protection at the riverward face of the wall is provided against scour of the river bed. A timber fender that extends 18 inches beyond the wall prevents damage to boats or barge tows that may inadvertently bump the guide wall in passing.

The primary purpose of the guide wall is to reduce the sediment input to the Intake Structure by forcing bed load and other material contained at lower elevations in the river to flow past the intake to a point where inflow to the intake will not influence river behavior. The upper elevations of the river containing relatively finer sediment flow over the submerged weir. A model study indicated a potential reduction of as much as 75% in the amount of sediment to be carried into the Intake Structure.

Design of the wall was reviewed for stability against a potential of ten feet of scour with concurrent buildup of ten feet of silt at the interior face. Lateral loads due to water and/or backfill and a six-inch water differential across the wall due to Circulating Water and Service Water bay operations was considered in the design. Stresses for these conditions were limited such that a minimum factor of safety of 1.5 relative to yield stress was realized.

The Intake Structure Guide Wall is classified as a Class II structure, however its design has been reviewed for the effect of a Class I seismic event to ensure stability against failure which might affect the safe shutdown potential of the Service Water bay and its equipment. The anchored section is designed to resist the dynamic backfill pressure and hydrodynamic water pressure due to a Safe Shutdown Earthquake. The cantilevered section is designed to resist the hydrodynamic effects of water during the seismic disturbance. The cantilevered section is designed as a flexible structure immersed in water. Stresses were determined to be within the yield point of the materials for the seismic analyses. The Intake Structure Guide Wall was also reviewed with respect to barge impact and the structure was determined to remain functional (see USAR Section XII-2.3.7).

A three-foot wide by four-foot high hole has been installed near the far north end of the guide wall. A gate and gate frame assembly has also been provided to allow for opening and/or closing of the hole depending on the forecast river levels. The purpose of the hole is to provide a flow path from the Missouri River to the Service Water pump bay during low river water level conditions needed to ensure Service Water pump operability. Detailed engineering evaluations verify design basis Service Water pump hydraulic requirements can be met and ensure that the guide wall structural requirements are maintained. Failure modes by which the opening could become blocked,

including ice flows of maximum three-foot thickness and/or debris such as logs, were accounted for in the design. The design is such that the effects of siltation from flow through the opening are minimized.

An array of 20 submerged flow turning vanes has been installed east of the guide wall in the river channel. Each vane is constructed of steel sheet piling and driven into the river bed to a top elevation below barge navigation depth. The vane array functions to induce scouring of the river bed adjacent to the guide wall to prevent sediment accumulation. The prevention of sediment accumulations increases the effectiveness of the guide wall.

2.2.7.3 Circulating Water Conduits

Intake to the condensers is through a steel pipe while the discharge from the condenser to the Seal Well is through concrete tunnels. Connections at the pump discharge and at the condensers are steel pipe. Anticipating future units to be constructed at the site, provisions were made for placing future discharge tunnels between the Intake Structure and the Turbine Building.

2.2.7.4 Seal Well - Discharge Structure

A reinforced concrete discharge structure (Seal Well) is located South of the Turbine Building and discharges water into a discharge flume and then into the Missouri River. Water elevation for siphon operation is maintained by a gated weir at a minimum Elevation of 880 feet MSL. Stone rip-rap is used to prevent scours in the vicinity of the discharge structure.

2.2.8 Office Building

The Office Building provides office facilities for station personnel. The structure was originally a two-story reinforced concrete frame consisting of isolated column footings and grade beams supporting precast concrete wall panels. A third floor of similar construction was later added. The first floor is a concrete slab on grade while the second and third floors are concrete slabs supported by concrete beams and girders.

In 1987 and 1988 an additional 20,000 sq. foot addition was constructed in front of the original building. The original front wall was removed so that one complete and continuous building would result. The new superstructure added consists of a structural steel frame work supporting a composite floor system consisting of steel decking and a concrete floor surface. A portion of the 3rd floor extends over the top of the Railroad Airlock.

The Office Building is in scope for License Renewal per 10 CFR 54.4(a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.9 Multi-Purpose Facility

The Multi-Purpose Facility (MPF) houses decontamination and support equipment used in the maintenance and repair of plant components. The foundation and floor of the main area of the MPF were designed to allow for conversion into a low level radioactive waste storage area if needed. The foundation is supported by piles driven into the bedrock. A separation joint between the MPF and the Control and Radwaste Buildings prevents interaction during seismic events. Additionally, the MPF is designed to store processed

Dry Active Waste (DAW) in the High Specific Activity Waste (HSAW) and DAW storage areas. There are no plans currently or in the foreseeable future for long-term storage for low level waste in the MPF. If these plans change, then the appropriate radiation monitoring equipment will be installed in the MPF.

The Multi-Purpose Facility is in scope for License Renewal per 10 CFR 54.4(a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.10 Augmented Radwaste Building

The Augmented Radwaste Building (refer to Burns and Roe Drawings 2072 and 2073) houses the various components of the Augmented Radwaste System as well as the instrumentation and control systems for the Augmented Radwaste System.

The Augmented Radwaste Building is in scope for License Renewal per 10 CFR 54.4(a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.11 Controlled Corridor

The Controlled Corridor is a reinforced concrete structure that is founded at Elevation 888'-6" on structural (Class I) fill placed between the Control Building and the Reactor Building. This Class I structure houses the Cable Expansion Room, through which the majority of the control, power and instrumentation cables are routed between the Cable Spreading Room of the Control Building and the Reactor Building.

The Controlled Corridor is in scope for License Renewal per 10 CFR 54.4(a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.12 Off-Gas Filter Building

The Off-Gas Filter and Fan Building is a prefabricated metal structure that is constructed on a reinforced concrete slab on grade. This Class II building houses the Off-Gas filter pits and the Off-Gas dilution fans and related equipment. This structure was evaluated for the effects of the Class I OBE seismic event to support the requirements applicable for the Sump Z system (see USAR Section XII-2.3.5.1.9)

The Off-Gas Filter and Fan Building is in scope for License Renewal per 10 CFR 54.4(a)(3) and was subject to aging management review. Aging effects are managed by the following Aging Management Programs: Fire Protection (see USAR Section K-2.1.16), and Structures Monitoring (see USAR Section K-2.1.36). There are no Time-Limited Aging Analyses that are applicable.

2.2.13 Railroad Airlock

The Railroad Airlock is a Class II structure with 12 inch thick concrete walls and roof, designed in accordance with Class II structural requirements for buildings. The structure is separated from the Class I Reactor

Building in such a manner as to permit building movements without any interaction between buildings. In addition, a flexible seal has been incorporated in the gap between buildings to assure secondary containment integrity.

The Railroad Airlock has been provided with a pair of gasketed swing doors at the exterior, and a pair of gasketed missile proof swing doors between the airlock and the Reactor Building. Both pairs of doors are interlocked so that neither can be operated unless the other is completely closed, locked and sealed.

The exterior doors of the airlock have been designed to resist a wind load of 30 pounds per square foot of door area acting inward or outward. Maximum deflection of door is limited to 1/240 of the span.^[8]

The interior doors between the airlock and Reactor Building have been designed for tornado loading, including impact from tornado generated missiles, as described in USAR Section XII-2.3.3.2.2.

2.2.14 Drywell and Suppression Chamber

The Drywell is a steel pressure vessel in the shape of an inverted light bulb which forms part of the Primary Containment System. The Drywell houses the Reactor Vessel, the Reactor Recirculation System and other branch connections of the Reactor Coolant System. The Suppression Chamber is a steel pressure vessel in the shape of a torus below and encircling the Drywell. Large vent pipes connect the Drywell and the Suppression Chamber.

2.2.15 Optimum Water Chemistry Gas Generator Building

The Optimum Water Chemistry Gas Generator (OWCGG) Building is a Class II structure, designed in accordance with Class II structural requirements for buildings, located along the north wall of the Turbine Building. The OWCGG building is a single story concrete block structure on a reinforced concrete foundation with a poured concrete roof. The hollow concrete blocks are filled with grout. The OWCGG Building has nominal 12" thick walls and contains four separate rooms. The rooms are: the Hydrogen Room, the Oxygen Room, the Rectifier Room, and the OWC Control Room. The rooms do not communicate with each other internally; instead, each room has a separate door(s) built into the exterior wall for access.

The northeast corner of the OWCGG Building is the OWC Hydrogen Room, and contains both the gas generator and the hydrogen compressor/accumulator skid. The walls of the Hydrogen Room are strengthened with reinforcing bars. The concentration of hydrogen in the Hydrogen Room is very low and the design of the walls and roof around the Hydrogen Room is sufficient to withstand a postulated 70 PSF blast load.

2.3 Loading Considerations

2.3.1 General

Class I and Class II structures are designed for dead, live, seismic, and wind loads in accordance with applicable codes and as described in the following paragraphs. The loading conditions are determined by the function of the structure and its importance in meeting the station safety and power generation objectives. Maximum allowable stresses used for general loading combinations are presented for Class I structures in Table XII-2-1. Detailed descriptions of the various load combinations for critical structures are provided in Appendix C. The allowable stresses and/or acceptance criteria for each load combination are also provided in Appendix C.

TABLE XII-2-1

ALLOWABLE STRESSES FOR CLASS I STRUCTURES

<u>Loading Condition</u>	<u>Reinforcing Max Allowable Stress</u>	<u>Concrete Max Allowable Compression Stress</u>	<u>Concrete Max Allowable Shear Stress</u>	<u>Concrete Max Allowable Bearing</u>	<u>Structural Steel Tension on the Net Section</u>	<u>Structural Steel Shear on Gross Section</u>	<u>Structural Steel Compression on Gross Section</u>	<u>Structural Steel Bending</u>
1. Dead Loads Plus Live Loads Plus Operating Load Plus Seismic Loads (OBE)	$0.5 f_y$	$0.45 f'_c$	$*1.1 (f'_c)^{0.5}$	$0.25 f'_c$	$0.60 f_y$	$0.40 f_y$	Varies with Slenderness Ratio	$0.66 f_y$ to $0.60 f_y$
2. Dead Loads Plus Live Loads Plus Operating Loads Plus Wind Loads	$0.667 f_y$	$0.60 f'_c$	$*1.467 (f'_c)^{0.5}$	$0.33 f'_c$	$0.80 f_y$	$0.53 f_y$	Varies with Slenderness Ratio	$0.88 f_y$ to $0.80 f_y$
3. Dead Loads Plus Live Loads Plus Operating Loads Plus Seismic Loads (SSE) or Tornado Loads	$0.90 f_y$	$0.85 f'_c$	--	--	$0.90 f_y$	--	Varies with slenderness ratio	$0.90 f_y$

NOTES: f_y = Minimum yield point of the material

f'_c = Compressive strength of concrete

Note that this table represents a generalization of allowable stresses for Class I structures. More specific load combinations and allowable stresses for particular Class I structures and more detailed load combinations are described in Appendix C.

* For beams with no web reinforcement. Other conditions in accordance with ACI Table 1002(a).

2.3.2 Vertical Loads

2.3.2.1 Dead Loads

The dead loads include the weight of the framing, roof, floors, walls, platforms, and all permanent equipment and materials.

2.3.2.2 Live Loads

The live loads include all vertical loads except the dead loads. These live loads are generalized in Table XII-2-2.

2.3.3 Lateral Loads

2.3.3.1 Wind Loads

Class I and Class II structures are designed to withstand a minimum wind velocity of 100 mph. They are constructed in accordance with standard codes and good engineering practice. The forces due to wind are calculated in accordance with the methods described in ASCE Paper No. 3269 entitled "Wind Forces on Structures". Applicable pressure and shape coefficients are used. A one-third increase in allowable stress is allowed for the wind loading conditions.

2.3.3.2 Tornado Loads

2.3.3.2.1 General

Class I structures are designed to the following tornado design criteria.

1. A tangential velocity of 300 mph.
2. A transverse velocity of 60 mph.
3. A pressure drop of 3 psi occurring over a three second time interval.

4. Tornado wind loadings (missile excepted) are combined with functional loads and the stresses resulting therefrom shall not exceed 90% of the yield stress in steel nor 85% of the ultimate strength in concrete. Where tornado loads govern the design, wind velocities above 300 mph can occur without stressing material beyond the yield point.

5. Class I structures are also designed to provide protection against tornado generated missiles (see USAR Section XII-2.3.3.2.2 below).

The only exception to this is that a system whose failure or malfunction might increase the severity of an accident is not designed to withstand the effects of a tornado if the failure of the system will not cause an accident. The reason for this exception is that the probability of the occurrence of a design basis loss-of-coolant accident or a design-basis tornado during the life of the plant is small. Therefore, the probability of the simultaneous occurrence of these two independent events is vanishingly small.

TABLE XII-2-2

LIVE VERTICAL LOADS

<u>Location</u>	<u>Load (psf)</u>
Roof - snow	20 minimum
Stairs and corridors	100
Assembly rooms	100
Platforms with grating or checkered plate	100
Storage pools	water plus equipment
Laydown areas	weight of equipment
Light piping and electrical loads	20
Heavy piping and electrical loads	calculated from actual pipe runs. This load is used to limit loads by hanger contractors.
Other Areas	varies depending on intended use and equipment.

2.3.3.2.2 Tornado Generated Missiles

Class I structures are designed to provide protection against tornado generated missiles as follows (except as noted in USAR Section XII-2.3.3.2.1):

1. A 35-foot long utility pole with a 14-inch butt with an impact velocity of 200 miles per hour.
2. A one-ton missile such as compact-type automobile with an impact velocity of 100 miles per hour and a contact area of 25 square feet.
3. A two-inch extra heavy pipe, 12 feet long.
4. Any other missile resulting from failure of a structure or component or one which has potential of being lifted from storage or working areas at the site.

These missiles are of low level origin. The only missiles that could be generated at high elevations are associated with the elevated release point and the superstructure roofing and siding panels. Missile loading investigations are based on the Bates and Swanson paper "Tornado Design Considerations for Nuclear Power Plants", presented to the annual meeting of the American Nuclear Society in November, 1967.

The station elevated release discharge line and its supporting tower are both constructed of steel and yield failure of either or both would result in buckling but no separation. The rupture of structural members or portions thereof in such a manner as to create missiles is not considered credible.

The limited instrumentation supported on the tower has very little mass and, assuming similar velocities, would produce less damage than the critical design missiles described above.

Siding panels would also have considerably less mass than the criteria missiles, and their effect would not be governing.

Exterior concrete walls are designed to prevent penetration by missiles. Local crushing and opposite face spalling of concrete would be expected to occur.

2.3.3.2.3 Tornado Protection for Openings

For those openings not shown protected in the general arrangement drawings, the provisions made to protect large openings against tornado and tornado generated missiles are described below:^[9]

1. Door No. R110 between the Railroad AirLock and the Reactor Building (shown on Burns and Roe Drawing 2060) is designed to resist a tornado imposed differential pressure load of 432 pounds per square foot (3 psi) acting outwardly or a wind load of 270 pounds per square foot plus the impact due to the following tornado generated missiles:^[8]

- a. A thirty-five (35) foot long utility pole with a fourteen (14) inch butt with an impact velocity of two hundred (200) miles per hour.

b. A one-ton missile such as a compact type automobile with an impact velocity of one hundred (100) miles per hour and a contact area of twenty-five (25) square feet.

2. Personnel Airlock Door Nos. R101 and R102 located at intersection of Column Lines 12.7 and K and shown on Burns and Roe Drawing 2060 are protected from tornado generated missiles by the adjacent Class I Controlled Corridor and Control Building.

3. Door No. R208 located at Column Line 12.7 leading into the 4,160V and 480V critical switchgear room and shown on Burns and Roe Drawing 2061 is protected as in 2 above.

4. Door No. H105 located at Column Line G just South of Column Line 16 leading into a non-critical corridor of the Control Building as shown on Burns and Roe Drawing 2051 is protected from tornado generated missiles by a thick concrete shield wall on column line 15 on the South; the Class I Diesel Generator Building on the East; the thick concrete bearing wall on column line 17 on the North; and by a thick concrete floor slab at Elevation 932'-6" in the Turbine Building.

5. Door No. H300 located at the intersection of Column Lines H.7 and 14 leading into the Control Room in the Control Room Building as shown on Burns and Roe Drawing 2052 is protected by the adjacent Class I Control Corridor and Reactor Building.

6. Door No. R116 located on Column Line J and 12'-6" North of Column Line 6 leading into the Alternate Shutdown Room and shown on Burns and Roe Drawing 2060 is protected from tornado generated missiles by the adjacent Turbine Building and Main Steam Tunnel concrete walls.

2.3.3.2.4 Tornado Loads - Additional Considerations

Blow-out Panels

When the pressure on the siding of the Reactor Building exceeds 75 psf, the barometric pressure drop that precedes a tornado will cause the siding to blow off, exposing the refueling floor. To ensure that this siding system blows off at 75 lbs per square foot pressure, sag rods are not installed and special bolts called "control release fasteners" are used to connect the girts to the building steel. These bolts will fail in shear when the internal or external pressure against the siding resulting from the tornado reaches that design failure pressure. Two types of load failure rated bolts were fabricated, tested and placed according to their rating to satisfy the failure load requirement of the Reactor Building North/South and East/West side girt locations. These failure load requirements for the North/South and East/West building sides are different since girt length and spacing are different on the North/South and East/West sides.^[10]

The superstructure of the Intake Structure is also designed for the metal siding to blow off during a tornado (when pressure exceeds 75 psf). The release mechanisms is the same as is in place for the Reactor Building where controlled release fasteners are designed to fail under tornadic conditions.

Tornado Effects on Refueling Floor

The tornadic winds are not expected to have any significant effect on the water levels in the operating floor pools. In order for the tornado to suck significant amounts of water from these pools (Spent Fuel Pool, Dryer-Separator Pit, and Reactor Cavity), the eye would have to pass directly over them in a manner such that significant rotary air flow with consequent differential pressures would exist across the pool surface for a significant length of time. Knowledge of lateral movement of tornadoes indicates that it would exist only about one to two seconds over the pool areas. While some water might be expected to be splashed from the pools as a result of such turbulent effects, it is difficult to envision tornado activity over the pools for extended lengths of time which would result in suction type water spouts (which are surface effects) sufficient to remove the water necessary to result in inadequate spent fuel cooling.

The Spent Fuel Pool, the Reactor Cavity, and the Dryer-Separator Pit are all interconnected with open flow paths between them during refueling operations. The Spent Fuel Pool contains about 2.1 million pounds of water of which amount 1.4 million pounds is above the bottom of the refueling slot between the Spent Fuel Pool and the Reactor Cavity. The Reactor Cavity (to the vessel flange) and the Dryer-Separator Pit contain a total of about 2.7 million pounds of water. Therefore, a total of 4.1 million pounds of water is available to flow between the various pools since all of the pools are interconnected and will maintain the same level to the bottom of the interconnection slot.

In order to uncover fuel in the Spent Fuel Pool, the tornado would have to suck 4.1 million pounds of water from the pools. In order to uncover fuel in the Reactor Core, the tornado would have to suck from the pool the 4.1 million pounds of water available for flow between pools plus about an additional one million pounds which must be sucked from the approximate eighteen foot-four inch diameter Reactor Pressure Vessel in order to expose the top of the fuel assemblies. Restoration of any loss of water from any of the refueling pools is readily available.

Potential missiles generated at high elevation are discussed in USAR Section XII-2.3.3.2.2. If the postulated tornado occurs during the refueling operation, the Reactor Pressure Vessel and internals are protected from falling debris by the water cushion. The Reactor Head, Drywell Head, and Shielding Plugs are massive and can easily sustain both wind and anticipated impacts.

Tornado Loads on Cranes

The Reactor Building and Intake Structure cranes and their supporting steel superstructures are designed to withstand tornado loading. Both cranes and columns are designed for tornado loadings with a rack and locking device such that both cranes would be locked to their supporting structure preventing the wind loads from pushing them off their crane runways. Additionally, the protective devices described herein are effective in preventing accidental displacement and mechanical break-up of the crane unit. Reactor Building and Intake Structure cranes are equipped with holddown lugs in order that they maintain stability and not be shaken off their rails in the event of an earthquake or tornado. These building cranes and supporting steel are designed to criteria outlined earlier in this section. Bridge and trolley wheels are double flanged and equipped with electrically activated spring set brakes. In the event of loss of power or when cranes are not under operator

control, the design provides for spring activated brakes which will lock the wheels firmly in place. Positive wheel stops and bumpers are provided in order to prevent the trolley and bridge from leaving the rails in the unlikely event of brake failure. A more complete description of the stations' cranes can be found in Chapter X.

2.3.3.3 Crane Runway Loads

The lateral and longitudinal forces on crane runways due to lifted loads are in accordance with the AISC 6th Edition. See USAR Section X-4.10 and Appendix K.

2.3.4 Pressure and Thermal Loads

The pressure and thermal design conditions for the primary containment system are given in Section V. The Reactor Building is designed to contain an internal pressure loading of 36 psf (7 inches of water or 0.25 psi). The design of primary and secondary containment structure walls was evaluated for High Energy Line Break (HELB) induced pressure loads. Both the primary and secondary containment structure walls are designed such that they will resist a pressure of at least 2 psi (288 psf) from a HELB. The design of concrete walls and floors has included the applicable temperature gradients through the concrete.

With regard to the removable concrete shield plug above the Drywell, the effects of thermal loads, both operating and accident, were evaluated and the resultant stresses were found to be within the allowable stresses when considered in combination with other loadings. Consequently, no detrimental effect will occur.^[11]

2.3.5 Seismic Loads

2.3.5.1 Seismic Design

The following definitions aid in the interpretation of the terms used in this section:

Operating Basis Earthquake (OBE) - Maximum Probable Design Earthquake.

Safe Shutdown Earthquake (SSE) - Hypothetical Maximum Possible Design Earthquake, Maximum Possible Earthquake.

2.3.5.1.1 Class I Structures and Equipment

The seismic design for Class I structures and equipment is based on dynamic analyses using acceleration response spectrum curves which are based on a ground motion of 0.1 g for the Operating Basis Earthquake (OBE). In addition, the seismic design for Class I structures and equipment required for safe shutdown is also based on dynamic analyses using acceleration response spectrum curves which are based on a ground motion of 0.2 g for the Safe Shutdown Earthquake (SSE). Seismic design may also be based on the GIP-3 earthquake experience based method.

For the seismic design of Class I structures, horizontal and vertical earthquake loadings were considered to act concurrently so as to produce the most severe stress combination when combining stresses due to

seismic loadings with stresses due to normal design loads for idle and/or operating conditions. The earthquake loads in the horizontal direction were obtained from dynamic modal analyses of buildings. In the vertical direction, the buildings were considered to be rigid. The original vertical seismic acceleration component was specified as one-half of the horizontal ground acceleration. This design requirement was increased to two-thirds of the horizontal ground acceleration, which is the current Licensing/Design Basis.^[12] Evaluations were performed to confirm that the increase in vertical seismic acceleration component was accommodated by existing margins and conservatism in the original design analyses of the Principal Class I Structures.^[62]

The seismic design for Class I equipment and components is similar to that described above for structures, and is discussed in detail in USAR Section C-3.3.4.

Revision 3 of the Generic Implementation Procedure (GIP-3), as modified and supplemented by the U.S. Nuclear Regulatory Commission Supplemental Safety Evaluation Report (SSER) No. 2 and SSER No. 3, may be used as an alternative to other authorized methods for the seismic design and verification of existing, modified, new and replacement equipment classified as Class I and which are in the scope of equipment in GIP-3. Details regarding the use of the GIP-3 method are in Appendix C.

2.3.5.1.2 Class II Structures and Equipment

Class II structures and equipment are designed to resist effects of seismic loads with the horizontal base shear coefficient as determined from the Uniform Building Code, Zone I, or taken as 0.10, whichever is greater. The base shear is distributed and the structures are designed in accordance with the provisions of the Uniform Building Code, with a 1/3 allowable increase in basic stress. All equipment is bolted or fastened so that it will not be displaced if friction is nonexistent.

For crediting dose consequence mitigation due to iodine plateout in the main turbine condenser for a LOCA, an evaluation was performed of the ability of the MSIV leakage pathway piping, main turbine condenser, and the Turbine Building (TB) to remain structurally intact following a Safe Shutdown Earthquake (SSE). Seismic qualification has been demonstrated by satisfactory completion of a technically detailed seismic evaluation of the ability of these Class II SSCs to maintain sufficient structural integrity during and after an SSE. Details of the seismic evaluation are included in USAR Section XII-2.3.5.3.

2.3.5.1.3 Structure Interfaces^[15]

Class I to Class I and Class I to Class II interfaces were designed so that relative movements between the structures will not endanger the integrity of the Class I elements. A dynamic analysis of the adjacent interfaces was performed and relative deflections computed at the interfaces.

The Class II buildings and the adjacent Class I buildings are separated by a minimum two inch joint to prevent interaction during the Class I seismic occurrences. There are however several component supports and conduits which bridge the seismic gaps between buildings. An evaluation was performed that determined these supports are sufficiently flexible to prevent any dynamic interaction between the buildings.^[55] Furthermore, any possible action from Class II building components which might affect adjoining Class I structures

would be less severe than the effects of tornado generated missiles already considered in the design. When GIP-3 is used for seismic qualification, the evaluation of seismic spatial interaction with other Class I or Class II structures and components is evaluated.

The Turbine Building was specifically evaluated for the potential to impact adjacent Class I structures. It was determined that the only zone of the Turbine Building that can adversely affect the integrity of an adjoining Class I structure is the area in close proximity to the Diesel Generator Building. This particular area has been designed to Class I standards.

The Radwaste Building is classified as partially Class I and partially Class II. A dynamic analysis was performed for the entire Radwaste Building to determine the seismic effects on this structure. The structure below Elevation 903'-6" was designed as a Class I structure. The Class II portion of the Radwaste Building (i.e., above Elevation 903'-6") was checked for the effects of the SSE. The critical components of the structure to ensure the integrity of the Class II portion are the concrete shear walls. The maximum horizontal shear is less than an allowable stress of 225 psi. The allowable stress of 225 psi was derived using a Factor of Safety of two to the stress recommended by C. W. Dunham^[13], ($V_c = 0.15f'_c$). In addition, the maximum compressive combined bending and axial stress in the critical shear wall is considerably below the allowable of $0.85f'_c$. Also, the design stress for concrete in tension was taken as zero. There is sufficient reinforcing steel in the tensile area of the wall to produce a very low tensile stress in the reinforcement. In conclusion, the Class II portion of the structure will not collapse under SSE loadings.

2.3.5.1.4 Shear Walls

The exterior and interior concrete walls in Class I structures are designed to resist stresses from shears, moments, and deflections from the Operating Basis Earthquake (OBE) and the Safe Shutdown Earthquake (SSE). Earthquake shearing stresses and vertical tensile and compressive stresses have been calculated as stresses parallel to the direction of the earthquake motion. Since the height-to-width ratio of the exterior walls is less than one, they can be considered as shear walls when assigning an allowable shear stress. An allowable shear stress of 225 psi for the OBE was derived using a factor of safety of two to the stress recommended by C. W. Dunham^[13], ($v_c = 0.15f'_c$). Shear stresses in the walls were always less than the value of 225 psi for the OBE. Shear stresses for the SSE are considerably below the ultimate value of $0.15f'_c$ as suggested by Dunham.^[14]

2.3.5.1.5 Removable Block Walls^[16,17]

Many areas require removable sections around Class I equipment for operational procedures during the life of the plant. These removable sections consist of readily handled solid masonry block type units. These units may be made from either conventional or high density type concrete. Furthermore, except for the end joints, the block units are set unmortared but in such a manner as to preclude the possibility of direct shine through the crevices. In this state it was recognized that the walls were incapable of resisting any seismic occurrences.

To make the removable block wall units capable of withstanding the Class I seismic occurrences for their particular locations, and to preclude any damage to Class I equipment adjacent to the walls, a system of metal retaining

enclosures was used. These enclosures are made of corrugated metal siding units set on each side of the wall. This siding is fastened at two (2) points at the ends of the wall into the surrounding reinforced concrete walls at either the top and bottom or each side depending upon the direction of the span of the metal siding. The retainers at the ends of the siding are removable. This assembly of siding, end retainers, and concrete inserts are designed to withstand the Class I seismic occurrences.

Lateral seismic g forces for the block walls located in the Reactor Building (see Burns and Roe Drawing 4215) were obtained from the horizontal acceleration curves for the dynamic analysis of the Reactor Building (see Figure C-2-12). The weight of the block walls multiplied by the appropriate g factor resulted in the seismic load to be resisted.

The supports, which resist the seismic thrust of the removable block shield walls, were designed to resist that thrust at allowable working stresses for OBE for both steel and concrete. At SSE they were designed to resist the thrust at stresses of up to 90% of yield for structural steel and up to 85% of design ultimate for concrete.

2.3.5.1.6 Drywell Vessel^[18]

The entire Drywell vessel, including the personnel airlock and equipment hatches, were designed to withstand seismic loading using static seismic coefficients which are derived by seismic analysis as described in USAR Section XII-2.3.5.2. A more detailed description of the analysis methods and acceptance criteria is provided in USAR Section App.C-2.5.7.

Since the equipment hatches and the personnel airlock are completely separated from the Drywell Biological Shield Wall by means of an air gap, there is no interaction between these appurtenances and the concrete wall. This air gap is of sufficient dimension to accommodate all vessel movements.

2.3.5.1.7 Doors and Accessories^[8]

Doors and accessories between the airlocks and the Reactor Building are capable of withstanding the seismic conditions given below without any failure. The doors and accessories are capable of operation during and following the occurrence of an earthquake having seismic forces at the equipment of magnitudes up to and including those specified below.

1. Seismic horizontal and vertical forces are $0.8W$ and $0.14W$, respectively, where W is the weight of the item under consideration.

2. The seismic forces are applied as static loads. The horizontal seismic forces are considered to act in any direction. The vertical seismic forces are considered to act up or down. Horizontal and vertical forces are considered to act simultaneously so as to produce the most severe loading conditions. Seismic loads are applied to the centers of gravity of the component parts and assembly.

3. The stresses due to the seismic forces in combination with other design loads are not greater than those permitted for normal operating loads and not greater than those allowed per the AISC 6th Edition for Steel and ACI 318-63 for 3,000 psi concrete.

2.3.5.1.8 Cranes

The Reactor Building and Intake Structure cranes are equipped with hold-down lugs in order that they will maintain stability and not be shaken off their rails in the event of an earthquake. The Reactor Building crane and supporting steel are designed in accordance with the criteria for Class I earthquake loadings. These cranes were analyzed with maximum operating live loads.

A jib crane is installed at the equipment hatch on the Reactor Building operating floor at Elevation 958'-3". The crane itself does not perform a safety related function and is not critical to plant operation. It was determined that the crane could not break loose and damage any safety related equipment on any of the different levels below it during an earthquake. To minimize the potential for damage the mounting of the assembly is designed to withstand the seismic forces specified in Contract E70-5A under which most of the original hoists and cranes for Cooper were furnished. Also, a special fastener was mounted on the crane to prevent it from rotating when not in use.^[19]

2.3.5.1.9 Sump Z System^[49]

The seismic design for the Sump Z system (sump, sump pumps and motors, level control system, electrical power supply, etc.) is based on the Class I criteria for the Operating Basis Earthquake (OBE) but does not include the Class I criteria for the Safe Shutdown Earthquake (SSE). The power supply for the Sump Z system is routed from the Diesel Generator Building to the Sump. From the Diesel Generator Building, the power supply is routed along the external east side of the Class II Turbine Building Appendages (Exhaust Fan Room and Heating Boiler Room, Water Treatment Area, and Machine Shop), through an underground duct bank, into the Class II Off-Gas Filter Building, along the Class II Elevated Release Point (ERP) walkway and Class I ERP structure, to the sump. To ensure that the power supply is adequately supported, the Class II buildings and ERP walkway to which the power supply is attached are evaluated for OBE loading. The buildings and walkway meet the applicable criteria in those areas where a failure could impact the integrity of the Sump Z power supply.^{[50] [51] [52]}

2.3.5.2 Seismic Analysis

2.3.5.2.1 Class I Structures

Method of Analysis

A dynamic analysis was performed for Class I structures. This was performed in four steps; a mathematical model was developed, the analysis was performed, structural response was obtained, and the spectra were plotted. Note that in 1986 the response spectra were regenerated to get plots with respect to frequency (original analysis plotted the results with respect to period). All basic inputs were the same, however the results were slightly different due to the differences in accuracy of these analyses.^[56]

Idealized mass-spring mathematical models were substituted for the Class I structures. A sketch of the mathematical model used in the seismic analysis of the Reactor Building is shown in USAR Figure C-2-1. It includes the effects of the dynamic coupling between the Reactor Building structure and the Drywell and the soil-structure interaction effects; the rotational spring, K_0 ,

represents the rocking interaction between building and foundation and the linear spring, K_L , models the dynamic soil pressure due to the exterior wall rocking against the backfill around the building. No vertical or horizontal soil springs at the foundation were included in the mathematical model analyzed. Justification for this is based upon the fact that due to the stiffness of the compacted structural fill under the foundation, and its relatively shallow depth to rock (less than 30 feet), the inclusion of a vertical and/or horizontal soil spring would not materially alter the results obtained without such springs.

The ground input motion used in these analyses is described in USAR Section II-5.2. The N69W component of the July 21, 1952, earthquake recorded at Taft, California, was specified as an appropriate accelerogram for the station site. Since the N69W component has a recorded maximum acceleration of 0.157 gravity the accelerogram amplitude was multiplied by 0.100/0.157 to represent the horizontal component of the Operating Basis Earthquake (OBE).

The idealized mass-spring mathematical models which are used to represent actual structural systems consider the mass of the system to be concentrated at discrete points connected by weightless linear elastic springs which simulate the stiffness of the actual structure. The stiffness of the actual structure is determined accounting for flexural and (shear) effects.^[20]

Similar procedures were used to analyze all of the Class I structures. As an example, the mathematical model developed for the Control Building is presented in USAR Figure C-2-2.

The method of seismic analysis was typically the response spectra method of modal dynamic analysis. The time history modal analysis method was used to analyze those structures for which the response spectra method was considered inadequate and/or to develop seismic criteria (floor response spectra) for Class I equipment housed in Class I structures.^[21]

By either method the equations of motion of a multi-degree-of-freedom discrete-mass damped system subjected to ground motion are uncoupled using the property of the orthogonality of natural mode shapes.

Using the response spectra method maximum modal displacements and maximum modal inertia forces were obtained; the other modal quantities such as shears and moments were then computed for each mode by conventional structural analysis procedures. The individual modal maxima were generally combined by the root-mean square method (square root of the sum of squares); if several controlling frequencies in an Eigen value solution were found to be close together the modal maxima were obtained by direct summation (sum of absolute values), or the system was analyzed by the time-history method (which is, computationally, an exact method).^[22]

Responses obtained from both methods of dynamic modal analysis, the time history method and the response spectra method, are contained in Table XII-2-3 for a number of selected points of the Reactor and Control Buildings. These values correspond to an earthquake input of an amplitude equivalent to the Operating Basis Earthquake (the peak of the horizontal component of the ground acceleration equals 0.100G). Comparison of values using both methods provides verification of the seismic system analysis.^[58]

TABLE XII-2-3
 RESPONSES OBTAINED FROM MODAL ANALYSIS

<u>BUILDING</u>	<u>ELEVATION</u>	<u>METHOD OF DYNAMIC MODAL ANALYSIS</u>			
		Time History		Response Spectra	
		Method		Method	
		<u>E-W</u>	<u>N-S</u>	<u>E-W</u>	<u>N-S</u>
Reactor Building	At Foundation Level (El. 854'-9")	0.100	0.100	0.100	0.100
	At El. 976'-0"	0.275	0.265	0.239	0.239
	At Operating Floor Level (El. 1001'-0")	0.305	0.295	0.292	0.292
Control Building	At Foundation Level (El. 873'-6")	0.100	0.100	0.100	0.100
	At El. 918'-0"	0.195	0.210	0.202	0.185
	At Roof Level (El. 948'-9")	0.287	0.334	0.294	0.266

The results obtained from the response spectra modal analysis of Class I structures were checked for reasonableness and showed adequate conservatism. The magnitude of the horizontal response at the foundation level was at least equal to the peak ground acceleration, and the horizontal response at the refueling floor level of the Reactor Building and at the roof level of other Class I structures was approximately three times the peak ground acceleration.^[23]

To account for the effect on the floor response spectra due to the expected variations from the assumptions made for the structural properties, damping, soil-structure interaction, etc., a shift of the peak responses of at least $\pm 10\%$ was considered (note that 15% was used for the 1986 analysis).^[24]

Torsional Effects

The following procedure was followed to account for the effects of the torsional modes of vibration in the design of the Class I structures:

A stiffness analysis was performed to determine torsional effects due to the actual eccentricity between the center of mass and the center of rigidity of the structures on the vertical structural elements resisting lateral loads.

The minimum eccentricity considered in the analysis was as specified in Section 2314 of the Uniform Building Code of the International Conference of Building Officials (1967 Edition). This criteria was used in the design of symmetrical structures as well.^[26]

All structural elements resisting horizontal earthquake loads were designed for the stresses due to torsional effects, determined as described above, in combination with stresses produced by other normal design loads, such as to produce worst stress conditions and remain within code or specified limitations.^[27]

Overturning Moments

The overturning moments calculated for the Safe Shutdown Earthquake results in a factor of safety against overturning of approximately 2.0.

The overturning moments of the Class I structures were obtained from the dynamic analysis performed to determine horizontal effects, and were considered simultaneously with vertical effects determined also by dynamic analysis.^[28]

2.3.5.2.2 Piping

Definitions of classifications used for all piping are presented in Appendix A of the USAR.

Seismic Class IS and IIS piping systems account for seismic loads. For Seismic Class IS piping systems 2 1/2" and greater in diameter, dynamic analyses were performed using response spectra. A detailed description of this procedure is presented in Appendix C, Section C-3.3.3.2. For Seismic Class IS piping systems less than 2 1/2" in diameter, piping and supports were field routed using span and load chart tables. The design of Seismic Class IIS piping systems was based on loading specified in the Uniform Building Code for an earthquake of 0.10g intensity.

In order to assure that seismic effects of Seismic Class IIS piping will not cause failure of Seismic Class IS piping and equipment, sufficient number of seismic controls are provided on the Seismic Class IIS piping in question, so as to assure that it remains in place at time of a stipulated earthquake.^[30]

The portion of the Main Steam piping from the Reactor Pressure Vessel through the second isolation valve and to the anchor outside the Primary Containment has been designed as Seismic Class IS piping. The remainder of the piping to the turbine has been designed as Seismic Class IIS piping. As noted in USAR Section XII-2.1.3.1, the Turbine Building is a Class II structure that has been evaluated as remaining structurally intact following a Safe Shutdown Earthquake. As part of the licensing of the LOCA dose calculations, the NRC has required that the Turbine Building, the Main Condenser, and the piping of the MSIV leakage pathway to the Main Condenser be evaluated as remaining structurally intact following an SSE. This seismic qualification has been demonstrated by completion of a technically detailed seismic evaluation of the ability of these SSCs to maintain sufficient structural integrity during and after an SSE. Details of the seismic evaluation are described in Section XII-2.3.5.3.

2.3.5.2.3 Protective System Instrumentation

Each type of Reactor Protective System instrument and its supporting panel or cabinet is analyzed, tested, or investigated to confirm that it will withstand the interaction effects of the floor acceleration from the Safe Shutdown Earthquake without loss of function. The interaction effects on a Reactor Protective System instrument are determined by the dynamic response of its supporting control panel or cabinet, static analysis, or test.

2.3.5.2.4 Electrical Equipment^[33]

The seismic design criteria to assure the adequacy of Class I cable trays, battery racks, instrument racks, and control consoles was typically accomplished by static analytical procedures, vibration testing, and/or use of the GIP-3 experience-based method.

Static Analysis

The static analysis included the following combination of equivalent seismic coefficient acting as the center of mass applied simultaneously in the most disadvantageous direction.

Operating Basis Earthquake

horizontal	0.75g
vertical	0.07g

Safe Shutdown Earthquake

horizontal	1.50g
vertical	0.14g

The seismic coefficients shown for the static analysis are General Electric's standard values used to define the seismic capability of the equipment with natural periods of vibration of 0.10 seconds or less in low seismic risk areas (Zones 0, 1, and 2) as defined by the Uniform Building Code.

The natural period of vibration for Class I equipment is calculated and the corresponding design acceleration for the Cooper Nuclear Station is derived from the appropriate floor response spectra. The design accelerations are checked against the standard values to insure that the standard coefficients are more conservative.

Vibration Testing

The accelerations used in vibration testing of critical instrumentation to assure no loss of essential function exceeded the maximum accelerations expected from the building motion. The values used for vibration testing at the points of attachment are equivalent to 1.50g horizontal and 0.50g vertical over the frequency range of 5 to 33 Hertz.

The vibration testing criteria was defined to comply with or exceed the static seismic requirements established for low seismic risk areas.

GIP-3 Experience-Based Method

Revision 3 of the Generic Implementation Procedure (GIP-3), as modified and supplemented by the U.S. Nuclear Regulatory Commission Supplemental Safety Evaluation Report (SSER) No. 2 and SSER No. 3, may be used as an alternative to other authorized methods for the seismic design and verification of existing, modified, new and replacement equipment classified as Class I.

Only those portions of GIP-3 listed in Appendix C, which apply to the seismic design and verification of mechanical and electrical equipment, electrical relays, tanks and heat exchangers, and cable and conduit raceway systems shall be used. The other portions of the GIP are not applicable since they contain administrative, licensing, and documentation information which is applicable only to the USA A-46 program.

Seismic Restraints

The methods of seismic restraints including the anchorage system, welded stiffeners, cross-bracing and lateral supports to the building, are designed such that the stresses due to seismic forces in combination with other design stresses do not exceed the allowable design stresses and/or stiffness required.

Alternate Analysis Methods

Alternate methods may be used for the analysis of miscellaneous electrical equipment and their supports. These methods include response spectra analysis, time history analysis, and a static analysis method using seismic coefficients based on the response spectra of the floor where the equipment is located.^[57]

2.3.5.2.5 Damping Values

The damping factors used in the seismic analysis were based upon deformations or stresses of various materials and are shown in Table XII-2-4.

USAR

TABLE XII-2-4

STATION STRUCTURES

DAMPING FACTOR

<u>Item</u>	<u>% of Critical Damping</u>
Reinforced Concrete Structures	5.0 and 7.0
Steel Frame Structures	2.0
Welded Assemblies	1.0
Bolted and Riveted Assemblies	2.0
Essential Piping System	0.5
Or	
Equipment Qualified Using GIP-3	GIP-3 Damping Value

A damping value of 5% of critical was used for the Operating Basis Earthquake (OBE) and a value of 7% of critical was used with the Safe Shutdown Earthquake (SSE) for all reinforced concrete structures founded on soil (structural fill) which are within the range of 3% to 5% and 7% to 10% of critical, respectively, as recommended by Dr. Newmark in his papers^[34,35] and used for similar constructions and conditions. 5% damping was used when the GIP-3 method of seismic qualification was employed.

2.3.5.2.6 Cracks in Concrete Due to Seismic Loads

It is estimated that for the Safe Shutdown Earthquake, cracks will develop in the exterior concrete walls of the Reactor Building below approximate Elevation 958'-0". It should be noted that the part of the Reactor Building below grade is protected against inflow leakage by the existence of a continuous membrane used as water proofing. The estimated total length of cracks which will remain open and unprotected after the earthquake transient is 2,500 linear feet with an estimated 8 mils average width. Considering the preceding data and a negative pressure of 0.25 inches of water within the Reactor Building the inflow leakage would be 160 cfm to 225 cfm for winds of 20 mph to 30 mph. The estimated inflow leakage rate is well within the capacity of the Standby Gas Treatment System or the normal building exhaust ventilation system.^[36]

2.3.5.3 Seismic Analysis - Alternate Leakage Treatment System

As part of the licensing of the LOCA dose calculations, the NRC has required that the Turbine Building, the Main Condenser, and the piping of the MSIV leakage pathway to the Main Condenser be evaluated as remaining structurally intact following an SSE. These SSCs collectively comprise the Alternate Leakage Treatment (ALT) system. The seismic qualification of the ALT System has been demonstrated by completion of a technically detailed seismic evaluation of the ability of these SSCs to maintain sufficient structural integrity during and after an SSE.

The seismic evaluation of the ALT System conforms to the methodology contained in NEDC-31858P, subject to the additional guidance and limitations contained in the NRC Safety Evaluation dated March 3, 1999, "Safety Evaluation of GE Topical Report, NEDC-31858P, Revision 2, 'BWROG Report for Increasing MSIV Leakage Limits and Elimination of Leakage Control Systems,' September 1993". The boundaries of the MSIV leakage pathway to the Main Condenser are shown on drawing CNS-MS-43. Seismic analyses and evaluations were completed for piping, pipe supports, the condenser, condenser anchorage, related equipment, and the Turbine Building concrete structure through application of the above methodology. Several key elements of the pathway configuration and seismic analyses and evaluations are described below:

1. Manual actions are required to configure the MSIV Leakage pathway to the condenser post-DBA LOCA such that radioactive nuclides proceed to the condenser and do not escape through unanalyzed pathways. To ensure the pathway is configured as necessary, operator action is required to manually close the applicable boundary valves and open the pathway valves to the Main Condenser after alarms are received in the control room indicating a LOCA with core damage. In addition, plant personnel action is required to manually use shaft adjustment tools to adjust each of the two turbine stop valve shafts to eliminate a potential leakage pathway to the Turbine Building via the two Turbine Stop Valve shafts. This leakage could occur assuming a loss of gland seal steam. A shaft adjustment tool would be installed on each Turbine Stop

Valve shaft to close the clearance by moving the shaft outwards from the valve body such that a shaft sealing ring (located on the inside of the valve body) is sealed against the face of the valve bushing. Sealing in this manner (shaft sealing ring to valve bushing) is the sealing method that occurs during normal operation, except that steam itself provides motive force instead of a shaft adjustment tool. The time required to accomplish these manual actions has been evaluated, and it has been determined that there is sufficient time to perform these evolutions before local radiological conditions would become a concern due to the LOCA source term.

2. A Turbine Building and Reactor Building Floor Response Spectra (FRS) was developed as input to the MSIV Leakage Pathway seismic evaluation for the detailed, in-depth dynamic analyses of the selected piping systems and for outlier resolution. These specific FRS have been calculated following the guidance in NUREG-0800 (Standard Review Plan) Sections 3.7.1 and 3.7.2, using the Soil-Structure Interaction analysis method and the Regulatory Guide 1.60 ground response spectrum anchored to the CNS SSE peak ground acceleration of 0.2g. These FRS are used expressly for the seismic qualification of the MSIV leakage pathway to the condenser only.

3. Various equipment was evaluated per the requirements of the GIP using the seismic capacity compared to seismic demand methods outlined in Section 4.2 of the GIP (see USAR Section XII-2.3.5.1.1). The majority of the related equipment are valves. The valves were found to meet GIP screening criteria. Valve operability is not a concern because all of the valves are passive in the case of motor-operated valves, or fail safe as in the case of air- and solenoid-operated valves. The manually operated valves do not belong to any of the designated SQUG-GIP valve classes of equipment (i.e., Fluid Operated Valves, Motor Operated Valves, or Solenoid Operated Valves). In the application of the SQUG-GIP, manual valves are classified as "Inherently Rugged Equipment," and as such, do not require their seismic adequacy to be verified in the USI A-46 program. Accordingly, no outlier conditions existed as result of the use of these manual valves. The valves were considered in the overall seismic evaluation of the piping systems.

4. For the evaluation of pipe supports, the anchor bolt capacities of Appendix C of the GIP were used for concrete expansion anchors.

5. The evaluation of piping included the following steps: (a) Walkdowns of the piping systems and associated supports which included identification of items judged to have inadequate seismic capacity, worst case pipe supports, and items requiring limited analytical reviews. (b) A comparison of piping system demand versus experience-based capacity. (c) Limited analytical reviews and pipe support evaluations for piping systems identified during the walkdowns. (d) Generation of Piping System Seismic Screening Work Sheets, a formal method of documenting the walkdown, the limited analytical reviews, the worst case support evaluations, and the final seismic capacity evaluation which included detailed response spectra modal analysis for several piping systems. Additional discussion is included in USAR Appendix A.

6. The Turbine Building structure, piping and equipment supports, and other systems, structures and components within the Turbine Building are subject to periodic structural inspections in support of Maintenance Rule activities as determined to be appropriate by the CNS Maintenance Rule program. The main steam piping system in the Turbine Building is inspected every-other cycle to potentially identify any deficiencies with pipe supports.

2.3.6 Primary Containment Loading Considerations

2.3.6.1 General

The Primary Containment System was designed to withstand the forces associated with a postulated Loss-Of-Coolant Accident (LOCA) including jet forces and forces associated with the post-accident flooded condition. The jet forces given in Table XII-2-5 were assumed to result from the impingement of steam and/or water at 281°F. For the flooded condition, the primary containment was assumed to be filled with water up to the normal refueling level.

A complete reevaluation of the Primary Containment System was performed under the Mark I Containment Program to include the effect of hydrodynamic loads that had not been considered in the original design. These loads were associated with SRV discharges and short-term LOCA events. The redefined loading considerations and results of these evaluations are documented in the Mark I Containment Program CNS Plant Unique Analysis Report. A summary of the Mark I containment reevaluation is also included in USAR Section App.C-2.5.7.1.

2.3.6.2 LOCA Loads

The forces on structures due to thermal expansion of pipes and/or equipment under operating conditions change during a Loss-Of-Coolant Accident (LOCA).

The basis upon which the forces on structures have been established for LOCA conditions are:^[37]

1. Increase of containment vessel temperature to 281°F.
2. Item 1 above in combination with normal operating temperature of Reactor Pressure Vessel to simulate a transition period before cooling down of metal.
3. Item 1 above in combination with Reactor Pressure Vessel temperature corresponding with internal pressure after LOCA.
4. All systems connected to Reactor Pressure Vessel assumed to have the same temperature as the Vessel up to the second isolation valve.

All possible modes of operation were considered such as:

1. Normal plant operation.
2. Normal system operation.
3. Accident condition.
4. Forces on structures only from piping inside the containment vessel to simulate an outside break.

TABLE XII-2-5

JET FORCES LOADING ON PRIMARY CONTAINMENT

<u>Location</u>	<u>Jet Force Maximum</u>	<u>Interior Area Subjected to Jet Force</u>
Spherical part of Drywell	660 kips	3.7 sq ft
Cylindrical part of Drywell and transition to sphere	468 kips	2.6 sq ft
Closure Head	33 kips	0.18 sq ft
Suppression Chamber (reaction force)	*	*

* The Suppression Chamber reaction forces on the downcomers and relief valve discharge headers were reanalyzed as part of the Mark I program. A summary of this program is provided in the CNS Plant Unique Analysis Report.

5. Conversely, forces on structures only from piping outside the containment vessel to simulate an inside break.

The worst combination of all the above thermal modes of operation are utilized, with no increased allowable stresses.

The extent of damage due to a postulated pipe rupture LOCA is dependent on the location of the break, whether there are any missiles generated, and whether the break will allow pressure to build to unacceptable levels in closed areas.

Most piping attached to Reactor Pressure Vessel (RPV) nozzles penetrates the Sacrificial Shield Wall. All penetrations are provided with hinged and latched steel doors, whose thickness varies from two inches to nine inches, depending on the gamma radiation shielding requirement. These steel doors represent potential missiles in an accident. It is noted that the penetration configurations are designed for ready access during in-service inspection (ISI).

The effects of possible effluent discharge into the annulus from a postulated pipe rupture in the vicinity of the RPV nozzle safe-end were investigated. As a result of this investigation, it was found that the maximum differential pressure buildup in the annulus is 2 psi, based on conservative assumptions.

The Sacrificial Shield Wall was conservatively analyzed to determine its capability to resist pressure generated in the annulus. With the latches and hinges on the steel doors being the critical components, detailed analysis of the several door types (eleven in number) was performed to determine the pressure capacity of each door. Results of analysis indicate that the limiting case could sustain a pressure differential of 8 psi in the annulus.

Since the maximum differential pressure buildup anticipated is less than the pressure capacity of the Sacrificial Shield Wall, the penetration steel doors and their components, these will not form missiles. No valves are located inside the Sacrificial Shield Wall; therefore, valves are not potential missiles within this area.

It is therefore concluded that the loadings which could result from a postulated pipe rupture in the Sacrificial Shield Wall annulus could not cause failure of the Shield Wall itself or create missiles. The engineered safety features necessary to mitigate the consequences of this LOCA would not be affected.^[38]

2.3.6.3 Pipe Anchor Loads

All process piping penetrating the containment vessel is anchored at, or near the containment penetration. These anchors comprise a natural separation of analytical models for pipe stress analysis. Therefore, pipe stress analysis was separately performed for piping inside containment and outside containment. The analysis considered the normal plant operation, normal system operation, and the LOCA accident condition. The loading at containment vessel penetrations is the summation of loads from piping inside and outside containment acting on the common anchor.

The actual anchor loads utilized were conservatively taken as the largest loads from piping inside containment, outside containment, or their algebraic summation. It is noted that a separate solution of piping on one side of containment would generate operating loads on that side of containment, which is representative of a simulated break on the other side of containment. For containment penetration design, the loads were utilized in the normal accident pipe rupture and maximum seismic load combinations.^[39]

2.3.7 Barge Impact Analysis^[40]

The Intake Structure and guide wall were reviewed with respect to barge impact and the structures were determined to remain functional. This evaluation included the development of a barge impact response spectra for the Intake Structure and associated equipment in the Service Water pump room.^[56] An oblique angle of 8.5° was determined to be the angle of maximum impact. Without taking credit for the Guide Wall upstream fill material, such a head-on barge impact may cause the Intake Structure and Class I equipment located inside to experience accelerations of about 3.0g at the level of the operating floor degrading to practically zero at the foundation. The Intake Structure and its Class I equipment have been designed to resist a statically applied horizontal acceleration of 3.0g.

With respect to a barge impact on the Guide Wall, the same 8.5° oblique angle is assumed. The upstream portion of the wall, which contains fill, provides added protection to the Intake Structure from the possibility of damage from a runaway barge during navigable river stages. Barge impact on the cantilever portion of the wall should normally result in minor local plastic deflection, by virtue of the flexibility of the cantilever section and the slanting nature of the impact. In the extreme, the sheet piling might be deflected to such a degree as to impact the Intake Structure but the deep embedment of the sheets into the riverbed will not permit deflection of the base of the wall and, therefore, a bridging situation would result. Contact with the Intake Structure at the critical Service Water Bay would occur at the face of the deicing tunnel at elevations varying from 871± to 867±, both elevations being higher than the upper limit elevation of the opening into the Service Water Bay.

A future increase in barge draft consistent with upgrading the Missouri River from a nine to twelve foot channel depth would have only a minor effect upon the form of deformation of the sheet piling under barge impact.

Flow of water to the Circulating and/or Service Water pumps will not be cut off.

2.4 Foundation Analysis

This USAR Section contains historical information, as indicated by italicized text. USAR Section I-3.4 provides a more detailed discussion of historical information. The factual information being presented has been preserved as it was originally submitted to the Atomic Energy Commission in the CNS FSAR in February 1971.

2.4.1 General

The foundation investigation and analysis for the construction of the station was performed in three parts.

1. *Field exploration.*
2. *Laboratory tests and analysis.*
3. *Establishment of foundation design criteria.*

The field explorations and laboratory tests led to the conclusions that the subsurface in-situ conditions in the station were somewhat variable particularly in the upper 15 feet. The borings indicated that the upper 15 feet consisted of combinations of silt and clay forming an impervious layer unsuitable for foundation loadings. Below this impervious layer the soil contained sand down to bedrock which was encountered some 60 feet below the surface.

The foundation design criteria were established on the basis of these conclusions.

2.4.2 Field Explorations

In addition to the overall site geologic and seismic explorations, detailed foundation investigations including boring and field permeability tests were carried out for use in establishing the foundation criteria for the station structures. Prior to construction, a series of test borings and pits were made in the general station area to determine the substructure condition. The test data is shown in Appendix A of the CNS-PSAR. Borings were made at various depths with some going into bedrock. Disturbed and undisturbed samples, suitable for laboratory testing, were taken from the borings and subjected to the laboratory tests outlined in Section XII 2.4.3.

A series of pumping and percolation tests were performed to obtain estimates of the permeability of the in-situ materials for use in establishing dewatering requirements during excavation.

2.4.3 Laboratory Tests

Representative undisturbed samples extracted from the test borings were subjected to a comprehensive laboratory testing program to evaluate the physical and chemical characteristics of the soil encountered at the site. These laboratory tests included the following:

*Direct shear tests
Unconfined compression tests
Confined compression tests
Triaxial compression tests
Moisture and density determinations
Particle size analysis
Liquefaction potential determinations*

2.4.4 Foundation Design Criteria for Structures

2.4.4.1 General

This section describes the foundation criteria for the station structures.

2.4.4.2 Design Considerations

The major station facilities include the Reactor Building, Turbine Building, Control Building, Controlled Corridor, Radwaste Building, Augmented Radwaste Building, Diesel Generator Building, Elevated Release Point, and the

Intake Structure. Auxiliary structures include the Office Building and Railroad Airlock, the Multi-Purpose Facility, and the Off-Gas Filter Building.

The analysis of the in-situ material revealed that the sand stratum underlying the site is susceptible to liquefaction if the site is subjected to the Safe Shutdown Earthquake (see Chapter II). It, therefore, followed that the major consideration in the selection of the foundation scheme is the ability of the soil to safely support the structures during and after an occurrence of the Safe Shutdown Earthquake.

All major structures have mat foundations that are supported by structural fill. The structural fill was constructed as follows:

1. Removal of the top layers of surficial fine-grained stratum consisting of sandy silt, silty clay, and clay. These soils were not considered suitable for structural fill.

2. The remaining soil to bedrock predominantly consisted of a poorly graded medium to fine sand, which was excavated to within nine feet of bedrock. The removed material was graded and stockpiled for use as structural fill material. The size of the excavation was large enough to permit the use of heavy earthmoving equipment.

3. Portions of the bedrock consist of soft shale or hard clay that will absorb water and soften as the overlying soil is excavated. It was expected that the soft shale would slake and weather rapidly on exposure to air and become very soft if it were subsequently allowed to become wet. For this reason the excavation was concluded at a point nine feet above the rock surface.

4. The material used for structural fill consisted of sand or sand and gravel having a maximum passing through a No. 200 sieve of 10%. The maximum size of material was limited to six inches. The sand was placed, spread evenly and thoroughly mixed to obtain uniformity of material and water content. The lift thickness before compactions was limited to twelve inches. Compaction was accomplished with a vibratory compactor (a Vibro-Plus Ch-43, or equal) pulled by a tractor travelling at a speed of not more than one and one-half miles per hour.

5. Ground water level was maintained at a minimum of three feet below the surface of the soil being compacted.

The steps above describe the structural fill replacement of the in-situ material for a suitable material to safely support these structures during construction of the facility. In the case of future excavation of the structural fill material, it must be replaced with a similar soil aggregate type by any method that achieves a relative density of 85%.

In order to provide a waterproof barrier on top of the structural fill, the bottom surfaces of the Reactor, Turbine, Control, Radwaste, and Augmented Radwaste Building mats are separated from their mud mats by plain sheets of high molecular weight vinyl chloride resins, pigments and plasticizers compounded to make permanently flexible sheets. This membrane material is completely resistant to acids, alkaline, or any other harmful soil conditions. The physical properties of the membrane material are such that it has a minimum tensile strength of 2,000 psi, good impact and abrasion resistance, flexibility, and a minimum elongation of 200% when tested in accordance with ASTM D-412. The minimum thickness of material used is not less than 40 mils. Joints or laps of adjacent membrane sheets were sealed by the use of special adhesives and tape strips of similar material to form a completely impervious waterproof barrier.^[41]

The coefficient of static friction established by the manufacture of the membrane material applied below all mat surfaces, has been reported as being 0.41. The friction coefficient was determined by loading a test sheet with a known weight and pulling it across a smooth troweled concrete surface. The coefficient was then calculated from the load and force figures.^[59] This value is less than the coefficient of friction (0.7) between the mud mat and the subgrade material. Using a design coefficient of friction of 0.4 the resultant factors of safety for the OBE is 2.3 and for the SSE is 1.1. The factors of safety were determined by dividing the total design load (DL + .25LL) times the coefficient of friction by the sum of the horizontal seismic shear plus the seismic vertical uplift times the coefficient of friction.^[59] These factors do not take into account lateral or frictional resistances which may be developed along the building foundation walls, which will substantially increase the aforementioned safety values.^[41]

Since it is impractical to check the membrane waterproofing barrier after the mat has been completed, it was necessary to provide rigorous methods of quality assurance to ensure the proper installation of this material. In addition to rigorous inspection, a two inch protective concrete cover was installed for additional integrity assurance against damage during installation of reinforcing and other mat concrete components.^[41]

Membrane waterproofing material used on the exterior foundation walls of the Reactor, Turbine, Control, Radwaste, and Augmented Radwaste Buildings is the same as described above with the exception that integrally molded protrusions were used to obtain a cast-in-place barrier with the concrete foundation wall. This barrier was sealed at the top just below grade into a reglet, and at the bottom was sealed to the mat material by the same adhesives and tape strips outlined above. With respect to in-service inspection of the vertical waterproofing barrier, the same controlled application as described above was used, except that the two inch protective cover was deleted. Therefore, no provisions were made to check the integrity of the membrane during the life of the plant. Drains are provided in the base slab of all buildings to collect any water which may accumulate on the slab for any reason.

All buildings have physical separations in the form of joints varying in width from two to four inches, such that all postulated building movements, have been incorporated to prohibit physical building interactions. These joints are sealed at either the top or their sides so that it is unlikely that any foreign materials will enter the joints. However, periodic inspection of the joint seals may be performed to assure their integrity.

Where penetrations enter buildings through sleeves, details relating to the methods of waterproofing and maintaining building integrity have been developed.^[42]

The waterproof membrane described above prevents leakage of any radioactive liquids in the event that the concrete walls or mats of the Reactor Turbine, Control, Radwaste, or Augmented Radwaste Building crack.^[43]

2.4.5 As-Built Reactor Building Concrete Substructure^[44]

During and after the placement of concrete in the substructure of the Reactor Building several as-built conditions were noted which were not in accordance with the wording and intent of the specifications. These as-built conditions were first noted visually after the removal of the concrete forms.

Subsequently, an extensive sonic test program was instituted to determine the extent and nature of all as-built conditions. The sonic tests were conducted throughout the entire mat, interior and exterior walls, and the Reactor Pedestal. Supplemental air and water tests through test core holes in the HPCI room mat further defined the extent of the as-built conditions.

A grouting procedure was established and instituted to fill the voids encountered by tests in the HPCI room mat. Another repair procedure was developed for repairing the as-built conditions in the walls.

The quality of the in-place concrete was determined from the numerous cylinders taken during the placement of concrete and the ultimate compressive strength of the as-built concrete was compared to that used in the original design.

From the data available from the tests conducted above, it was possible to determine the concrete and reinforcing steel stresses for the as-built conditions prior to repair and to evaluate their effect on the structural integrity of the structure. Included in this as-built evaluation was the effect of the cutting of the reinforcing steel by the numerous cores taken in the mat.

Subsequent to all repairs, tests were made to determine the effectiveness of all mat and wall repairs and to evaluate the as-built (repaired) substructure.

3.0 SHIELDING AND RADIATION PROTECTION

This USAR section contains historical information as indicated by the italicized text. USAR Section I-3.4 provides a more detailed discussion of historical information. The factual information being presented in this section has been preserved as it was originally submitted to the Atomic Energy Commission in the CNS FSAR in February 1971.

3.1 Design Basis

3.1.1 Radiation Exposure of Individuals

The permanent radiation shielding is designed to control radiation doses within the limits of 10CFR20 during normal station operation. In addition, the shielding is designed such that continued occupancy of vital areas is possible under accident conditions. Off site dose limitations are in accordance with 10CFR20. In addition to the requirements of 10CFR20, the Off-site Dose Assessment Manual (ODAM) addresses off-site limitations associated with the environmental radiation standards of 40CFR190 and 10CFR50 Appendix I.

The station shielding design is based on meeting the general radiation dose rates of Table XII-3-1 for normal operations. For the design basis accident, the station shielding design is based on the values of 10CFR100 and 10CFR50.^[60] For continued main control room occupancy, during the design basis accident, the shielding design is based on a Deep-Dose Equivalent (DDE) exposure of less than 0.5 rem in any eight-hour period.

In addition, a shielding design review was completed in response to NUREG-0737 Item II.B.2.^[61] This review was based upon the source terms of 100% of the core equilibrium noble gas inventory, 50% of the core equilibrium halogen inventory, and 1% of the core solids being contained within the Reactor safety systems. As a result of this review it was determined that resultant radiation levels within the Reactor Building will prohibit personnel entry during an accident. However, it was also determined that the station is designed such that entry into the Reactor Building is not required during an accident and is not considered a vital personnel access area.

Those areas considered to be vital areas requiring continuous occupancy to aid in the mitigation of or recovery from an accident are the main control room, the technical support center, and the radiochemistry analysis laboratory. The radiation levels within these access areas during an accident are less than the NUREG-0737 Item II.B.2 15 mrem/hr continuous occupancy criteria which is based on 10CFR50, Appendix A GDC 19. These vital area analyses remain bounding for the AST LOCA methodology. The radwaste control room and secondary alarm system also meet the continuous occupancy criteria. The central alarm station of the security system and the reactor coolant sampling station will have frequent access capabilities.

3.1.2 Radiation Exposure of Materials and Components

Materials and components are selected on the basis that radiation exposure, considering the shielding design, will not cause significant changes in their physical properties which adversely affect operation of equipment during their design life. Materials for equipment required to operate under accident conditions are selected on the basis of the additional exposure received in the event of a design basis accident.

TABLE XII-3-1

STATION SHIELDING
Design Basis Limitations*

<u>Degree of Access Required</u>	<u>Design Radiation Dose Rate, mrem/hr</u>
Continuous Occupancy	
Outside controlled access areas	0.5
Inside controlled access areas	1.0
Occupancy to 10 hr/week	6.0
Occupancy to 5 hr/week	12.0

* These criteria do not preclude entry into areas of higher radiation dose rates, since access is essentially determined by an integrated dose (combination of exposure time and dose rate).

The following general radiation exposure limits were considered in the selection of materials:

<u>Material</u>	<u>Approximate Damage Threshold</u>
Teflon	1.0 x 10 ⁺⁴ rads
Most thermoplastic resins and elastomers	1.0 x 10 ⁺⁶ rads
Some thermoplastics	1.0 x 10 ⁺⁷ rads
Ceramics	1.0 x 10 ⁺¹⁰ rads
Metals	1.0 x 10 ⁺¹¹ rads

3.2 Control of Access to Radiological Areas

Access to radiologically controlled areas of the site is based on the applicable regulatory requirements. Areas of the plant are classified in accordance with 10CFR20. Based on these classifications, access to these areas is controlled in accordance with the requirements of 10CFR20 and the Technical Specifications.

3.3 Shielding Description

The shielding design considers three conditions:

1. Normal full power operation. This also includes shielding requirements for certain conditions such as the release of fission products from leaking fuel elements.

2. Shutdown. This condition deals mainly with the radioactivity from the subcritical Reactor core, with radiation from spent fuel bundles during on-site transfer, and with the residual activity in the reactor coolant and neutron-activated materials.

3. Design Basis Accidents. Several postulated Design Basis Accidents have been investigated which release fission products to the general free space of the Reactor Building.

The materials used for most of the station shielding is ordinary concrete with a minimum bulk density of 144 lb/ft³. Wherever cast-in-place concrete has been replaced by concrete blocks the design assures protection on an equivalent shielding basis. Only in a few instances has steel, water, or high density concrete (minimum density 210 lb/ft³) been utilized as primary shielding materials.

Normal full power operation design conditions assume that the core is operating with a power density of 51.2 kW/liter and the power level of 2,381 MWt. At this power level the N-16 coolant activity leaving the pressure vessel is 31.4 μ Ci/ml. The station shielding is based on an assumed stack release rate of 0.1 Ci/sec after a 30-minute holdup time in the off-gas system. Reactor water fission product and activated corrosion product concentrations were assumed to be the maximum values expected: 2.4 μ Ci/ml, and 0.07 μ Ci/ml, respectively. These conditions yield maximum shielding conditions in the demineralizers, cleanup systems, and other associated radiation handling facilities.

The shutdown condition assumes that the reactor core has been operating at 2,381 MWt for approximately 1,000 hours. At 1,000 hours the fission product inventory approaches the infinite operation case.

The different areas of radiation protection are described as listed by specific location or building for convenience.

3.3.1 Main Control Room

The design basis accidents define the protection required for the Main Control Room. These accident conditions are described in Section XIV-6. The main control room design was based on the airborne fission product inventory in the Reactor Building following the design basis accidents. For continued main control room occupancy, during the design basis accident, shielding design is based on a Deep Dose Equivalent of less than 0.5 rem in any eight-hour period. In addition, a shielding design review was performed in response to NUREG-0737 Item II.B.2 (see USAR Section XII-3.1.1) which determined that the dose in the main control room was less than the NUREG-0737 limit of 15 mrem/hr (averaged over 30 days). The total integrated doses for main control room occupancy for a 30 day duration is summarized in Section XIV-7.

3.3.2 Reactor Building

The Reactor Building contains four major shielding structures; the Sacrificial Shield Wall, the Drywell Biological Shield Wall, the Main Steam Line Tunnel, and the Spent Fuel Pool.

Each Reactor Pressure Vessel nozzle has been provided with hinged steel shielding doors attached to the Sacrificial Shield Wall. These doors have been suitably cut to permit piping and insulation to pass through it. In addition, access ports have been provided in the skirt of the Reactor Pressure Vessel for remote viewing of the Reactor bottom head.

The Sacrificial Shield Wall has several shielding functions. It protects certain major portions of the Drywell space from excessive nuclear radiation exposures during operation. After shutdown it provides shielding from Reactor Pressure Vessel radiation for personnel engaged in inspection, maintenance, and repair of Drywell equipment and components. Also, together with the Drywell Biological Shield Wall it protects the general Reactor Building work areas. The Sacrificial Shield Wall concrete is approximately 2'-3" thick of high density magnetite concrete (minimum density 210 lb/ft³) in the region of the core and ordinary concrete outside the core region.

The Drywell Biological Shield Wall (which is outside the Drywell vessel) together with the Sacrificial Shield Wall provide the main protection for the areas surrounding the Reactor Pressure Vessel and the primary coolant and Reactor Recirculation systems. More than 7'-3" (using minimum Drywell Biological Shield Wall thickness at equator) of concrete thickness is used to keep the radiation dose rates in the fully accessible Reactor Building work areas to less than 1 mrem/hr. Radiation dose rates greater than 1 mrem/hr in the Reactor Building result from radioactively contaminated systems and components outside the Drywell Biological Shield Wall and from system operation.

Additional permanent shielding has been installed inside the drywell at Elevation 901' and 921'. This shielding has been installed around portions of Reactor Recirculation and Reactor Water Clean Up piping to reduce general dose rates inside the drywell. This shielding is in the form of lead wool blankets supported by seismically qualified carbon steel framing constructed from modular components.

The Main Steam Line Tunnel, with up to five feet thick concrete walls, is the connecting shield structure between the Reactor and Turbine

Buildings. The tunnel shielding protects against the very penetrating N-16 gamma radiation which is radiated from the passing steam.

The Spent Fuel Pool contains the highly radioactive spent fuel assemblies, control and instrument rods, and other non-fuel irradiated components originating from nuclear system equipment. A maximum of five feet of concrete thickness is used for radiation protection at the sides and six feet at the bottom of the storage pool. For purposes of providing adequate shielding, a minimum of 7.35' of water is maintained above top of active fuel during fuel storage or transfer activities in the Spent Fuel Pool. During cask loading/unloading operations, radiation dose does not exceed posted radiation area limits as described in 10CFR20 when handling decayed fuel with a minimum water shielding of 6.18'.^[63] The Technical Specifications prescribe a higher minimum level for irradiated fuel stored in the fuel storage racks during fuel transfer operations.

Major portions of the Reactor Water Cleanup System, the in-core flux monitoring equipment, the radwaste equipment, and the Reactor internals storage area are housed in numerous concrete shielded areas. Enclosing these secondary sources of radiation in shielded areas permits the adjacent areas to be accessible to personnel on a continuous basis.

The entrances into the Drywell space are well shielded with up to 5'-0" thick shield plugs at the equipment hatches at elevation 903'-6". The entrance at the equipment and personnel airlock, located at elevation 903'-6" is also shielded.

3.3.3 Turbine Building

Radioactive steam enters the Turbine Building via the Main Steam lines passing through the Main Steam Tunnel. Besides N-16, fission product gases and some radioisotopes are carried over from the Reactor water. Approximately 80% of the activity goes to the Off-Gas System with the other 20% following the condensate and being treated by the condensate demineralizers.

For this reason, radiation shielding is provided around the following areas:

1. Main Steam Lines
2. Primary and Extraction Steam Piping
3. High Pressure and Low Pressure Turbines
4. Moisture Separators
5. Reactor Feedwater System Heaters
6. Main Condensers and Hotwells
7. Air Ejectors and Gland Sealing System
8. Reactor Feedwater System Turbine Driven Pumps

The front end standard of the main turbine above the turbine operating floor is accessible to personnel for a very limited amount of time when the turbine is in operation. Concrete shielding protects personnel from radiation from some of the Main Steam piping and moisture separators. The general area near the generator on the turbine operating floor is accessible for a limited amount of time by shielding the moisture separators and some of the cross-over piping.

3.3.4 Radwaste Building

Areas for preparing, handling, storing, or shipping the radwaste are shielded to permit controlled access as required for operation of the radwaste systems.

The individual radwaste systems have been separated from each other and shielded as much as practical in order to minimize personnel exposure during maintenance and repair of any of the equipment.

3.3.5 Multi-Purpose Facility

The High Specific Activity Waste (HSAW) and Dry Active Waste (DAW) Storage Areas of the Multi-Purpose Facility (MPF) are surrounded by an 8 inch reinforced concrete wall which extends to a height of 16 feet and then opens into the general MPF spaces. The floor beneath these low-level radioactive waste storage areas is on grade.

Additional 10 inch thick concrete walls have been constructed along the North and South walls of the DAW Storage Area. Therefore, a total of 18 inches of concrete are provided for shielding purposes. The 10 inch walls were added for personnel protection in the adjacent work areas in response to planned onsite storage of DAW.

The HSAW storage vault is contained in the Southwest corner of the DAW Storage Area. The vault is surrounded by 2'-4" thick concrete walls and a 1'-6" thick removable cover. These shield walls and cover were specifically designed to adequately shield four 55-gallon drums containing solidified resins, each with a contact dose rate of 50 rem per hour. The floor of this HSAW Storage Area is on grade.

3.3.6 Office Building

All areas of the Office Building are fully accessible at all times.

3.3.7 Elevated Release Point and Off-Gas Piping

The shielding design for the Elevated Release Point provides for access at ground level. The Off-Gas filters are located in shielded cells below grade within a locked building.

3.3.8 General Station Yard Areas

Unposted station yard areas which are frequently occupied by plant personnel are in radiation fields typically less than 1.0 mrem/hr. These areas are also surrounded by a security fence and closed off from areas accessible to the general public.

3.3.9 Site Boundary^[45]

The annual Deep Dose Equivalent at the nearest site area and exclusion area boundary is conservatively estimated as 0.23 mrem/yr. The dose contribution due to the turbines, condensers, and condensate storage tanks is estimated as 0.1 mrem/yr. This is composed of 0.095 mrem/yr due to skyshine from sources in the Turbine Building, and 0.006 mrem/yr due to direct radiation from the condensate storage tanks. (When this calculation was initially performed, the site boundary was approximately 2,500 feet from the Turbine Building. It is now approximately 4,500 feet from it. Since the dosages calculated are conservative, the previously calculated doses are still reported herein.) The maximum estimated dose contribution due to the temporary storage of low-level radwaste at the Low-level Radioactive Waste storage pad is 0.13 mrem/yr.

The estimated contribution of sources in the Turbine Building to the site boundary dose was obtained by using existing dose rate calculations (which included the effects of shielding by equipment shells and pipe walls) to obtain an equivalent "unshielded" point source strength which would yield the same dose rate. This point source was then used with the data in Figure 4.5-10 of Reference [53] to obtain the total gamma ray flux in air at the nearest site boundary. The flux to dose conversion factor, and the fraction of the dose due to scattered radiation were derived from the data in Table 15.1 of Reference [46]. It was assumed that the direct contribution from these sources is negligible because of the presence of the shielding in the Turbine Building. No credit was taken for the shielding provided by the Turbine Building roof.

The estimated contribution of sources in the condensate storage tanks to the site boundary dose was obtained by converting the specification dose limit (1 mrem/hr at 1 meter) to an equivalent point source strength and then calculating the dose at the site boundary including the effects of buildup and attenuation by the air.

The contribution to the site boundary dose due to waste stored at the low-level waste storage pad was calculated assuming the maximum radwaste inventory expected. This assumed inventory was based on five years generation of radwaste, including dewatered resins and condensate sludge, totaling 100 storage containers. This resulted in a 0.13 mrem/yr contribution to the site boundary dose. The contribution to the site boundary dose due to the storage of Dry Active Waste within the Multi-Purpose Facility is negligible.

3.3.10 South Radioactive Material Storage Building

The building can be used to store radioactive material with a general dose rate of less than 5 mrem/hr at 12 inches. The radioactive material is stored and handled to prevent airborne contamination and the uncontrolled spread of contamination within the building.

3.4 Inspection and Performance Analysis

The normal construction quality control program assured that there were no major defects in the shielding. The control program included on-site inspection surveys by a site inspection agent prior to startup.

After startup, the adequacy of the shielding and the efficiency of the access control are periodically checked by radiation and contamination surveys performed at various reactor power levels.

3.5 Radioactive Material Safety

The radiation protection program provides reasonable assurance that byproduct, source, and special nuclear material will be stored, used, and accounted for in a manner which meets the applicable radiation protection provisions of 10 CFR Parts 20, 30, 40, and 70.

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