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12.1 Summary Description

The plant buildings and structures consist of the reactor building, turbine building, radwaste building, administration building, intake structure, off-gas stack, off-gas building, recombiner building, emergency filtration train building and other auxiliary structures. The design requirements contained in this section apply to the plant structures and equipment which are required to prevent the significant release of radioactivity, which are required for safe shutdown or which are required for power generation. Plant structures and equipment not discussed in this section are designed in accordance with building codes valid at the Monticello Site. The arrangement of the main plant structures on the site is shown in Drawing ND-95209, Section 15.

Drawings NF-36054 through NF-36063, Section 15, show the general arrangement of the reactor building, turbine building, office and control building.

12.2 Plant Principal Structures and Foundations

12.2.1 Design Basis

The environmental conditions of the site have been evaluated in detail in Section 2. Based on this evaluation the design bases described in the following sections have been used.

12.2.1.1 Safety Categories

The plant structures and equipment are divided into two structural safety categories:

Class I - Structures and equipment whose failure could cause significant release of radioactivity or which are vital to safe shutdown of the plant under normal or accident conditions and the removal of decay and sensible heat from the reactor.

Class II - Structures and equipment which are not essential for safe shutdown of the plant or removal of decay heat, but which are required for power generation.

The design of Class I structures and equipment takes into account postulated environmental and accident loading, including the design basis earthquake. Class II structures and equipment are constructed in accordance with industry codes and standards applicable to power plant construction in effect during plant design, including seismic coefficients from the Uniform Building Code (Reference 29).

12.2.1.2 Class I Structures and Equipment

Class I Structures:

Primary Containment (Drywell, Vents, Torus, and Penetrations)

Reactor Building (up to Operating Floor - 1027-foot 8-inch)

HPCI Building

Plant Control and Cable Spreading Structure

Spent Fuel Storage Pool

Off-gas stack

Reactor Primary Vessel Biological Shield and Support Pedestal

Standby Diesel Generator Building[2]

Diesel Fuel Oil Transfer House Containing Diesel Fuel Oil System*[2]

Emergency Filtration Train Building

Intake Structure Pump Room Containing Emergency Service Water and RHR Service Water Pumps and Connecting Pipe Tunnel

Parts of Turbine Building Housing Class I Equipment

Underground Duct Bank - 3rd Floor, EFT to Reactor Building.

Class I Equipment:

Nuclear Steam Supply System

Reactor Primary Vessel Supports

Control Rods and Drive System including Equipment necessary for Scram Operation

Control Rod Drive Housing Supports

Reactor Vessel Internals[1]

* Class II structure analyzed to Class I requirements.

1. The Reactor Steam Dryer Assembly and Reactor Steam Separator Assembly are considered Class II components. (See Class II Equipment List)
2. Class 1 tornado missile barriers (buried and above ground) located in the yard north of the Standby Diesel Generator Building are considered as a substructure that augments the purpose of these structures. See Section 12.2.2.4.3 for further discussion.

Fuel Assemblies

In-Core Housings

Reactor Core Shroud

Reactor Core Supports

Reactor Coolant Recirculation System Piping Including Valves and Pumps

Containment Isolation Valves

All piping from Reactor Vessel to first Isolation Valve External to Drywell

Emergency Core Cooling Systems

High Pressure Coolant Injection System

Core Spray System

Residual Heat Removal System

RHR Service Water System

Emergency Service Water System

Standby Gas Treatment System

Main Control Room & Emergency Filtration Train Building Air Conditioning System

Fuel Storage Facilities, (spent fuel and new fuel storage pools and contained equipment)

Standby Electric Power Systems

Plant Battery Systems

Standby Diesel Generator System

Emergency Buses and other electrical gear to and including power equipment required for safe shutdown

Standby Liquid Control System

Instrumentation and Controls Systems

Reactor Water Level Instrumentation

Reactor Manual Control System

Control Rod Drive System Instrumentation and Control

Reactor Protection System

Nuclear Instrumentation System

Radiation Monitoring System

All Class I equipment is designed to withstand the design basis (maximum) earthquake.

12.2.1.3 <u>Class II Structures and Equipment</u>

Class II Structures:

Turbine Building except portions housing Class I equipment

Radwaste Building

Low Level Radwaste Storage Building and new Shipping Building

Off-gas Storage Building except Fan and Foyer Room which are designed to withstand select Class I loads as they house Class I equipment.

Recombiner Building

Circulating Water System Including Cooling Towers

Administration Building

Turbine Building Addition

Class II Equipment:

Turbine Generator System

Main Condenser System

Reactor and Turbine Building Cranes

Reactor Feedwater and Condensate Systems

Condensate Storage and Transfer System Tank and Pumps

Plant Auxiliary Power Buses

Electrical Controls and Instrumentation (for above systems)

Reactor Cleanup Demineralizer System

Plant Radioactive Waste Control System

Reactor Core Isolation Cooling System

Turbine System Moisture Separators

Reactor Steam Dryer Assembly

Reactor Steam Separator Assembly

All Other Piping and Equipment not listed under Class I, which are required for power generation

12.2.1.4 Criteria for Design

General requirements for the design of all structures and equipment include provisions for resisting the dead loads, live loads, and wind or seismic loads with impact loads considered part of the live load. Selection of materials to resist these loads is based on standard practice in power plant design. Their use is governed by the building codes valid at the Monticello site and the experience and knowledge of the designers and builders. All class I structures and equipment were analyzed to assure that a safe shutdown can be made during ground accelerations of 0.06 g (operating basis earthquake) and 0.12 g (design basis or maximum earthquake). Class I piping and associated components have also been analyzed for appropriate thermal loads.

The loads of concern include the following:

D = Dead load of structure and equipment plus any other permanent loads contributing stress, such as soil or hydrostatic loads or operating pressures and live loads expected to be present when the plant is operating. This includes impact thermal and anchor movement loads as appropriate. The expected live load is the live load due to the use and occupancy of the structure. This includes loads from existing equipment and any other loads that may occur during plant operations.

W = Wind load.

W' = Tornado Loading.

P = Pressure due to loss-of-coolant accident.

R = Jet force or pressure on structure due to rupture of any one pipe as assumed during the original plant design. The High Energy Line Break (HELB) Analysis Report is shown in the USAR, Appendix I.

H = Force on structure due to relative thermal expansion under operation conditions.

T = Force on structure due to relative thermal expansion under accident conditions.

E = Design (operating basis) earthquake load. (0.06g)

E’	=	Maximum (design basis) earthquake load. (0.12g)
F	=	Probable Maximum Flood with flood waters in excess of elevation 930 ft msl and high water at elevation 939.2 ft msl for Class I structures and Class II structures housing Class I equipment.
A	=	Force on the structure due to a halon system actuation.

The criteria followed for all Class I structures and equipment with respect to stress levels and load combinations for the postulated events are noted below[1]:

Load Considerations		Allowable Stress
1. Primary Containment[2] (Drywell, Vents, TORUS, and Penetrations)		
a.	D+P+H+T+E	ASME, Section III, Class B, without the usual 1/3 increase for seismic loadings.
b.	D+P+R+H+T+E	Same as (a), above, except local yielding is permitted in the area of the jet force where the shell is backed up by concrete. In areas not backed up by concrete, primary local membrane stresses at the jet force do not exceed 0.90 times yield point of the material at 300°F.
c.	D+P+R+H+T+E’	Primary membrane stresses, in general, do not exceed the yield point of the material. If the total stress exceeds yield point, an analysis has been made to determine that the energy absorption capacity exceeded the energy input from the earthquake.[3] The same criteria as in (b) above, is applied to the effect of jet forces for this loading condition.

1. Design criteria for the spent fuel pool structure is defined in Section 12.2.2.1.1
2. For the suppression chamber and vent system the design loads, load combinations and stress criteria are described in the Monticello Mark I Containment Long Term Program Plant Unique Analysis Reports (See References 72 and 75 of Section 5)
3. Although this was an original design criterion, no cases were found where the yield point was exceeded by the earthquake energy input and therefore application of this criterion was not required.

Load Considerations			Allowable Stress
2.	Reactor Building and All Other Class I Structures[4]		
	a.	D+R+E	Normal allowable code stresses have been used (AISC for structural steel, ACI for reinforced concrete). The customary increase in design stresses, when earthquake loads are considered, has not been permitted.
	b.	D+R+E’	Stresses do not exceed: - 150% of AISC allowable for structural steel (not to exceed yield stress) - 90% of yield stress for reinforcing steel. - 85% of ultimate stress for concrete.[5]
	c.	D+W	Normal allowable code stresses (AISC for structural steel, ACI for reinforced concrete) with the customary increases in stresses when wind loads are considered.
	d.	D+W’	Stresses do not exceed: (Excluding Steel Superstructure)[6] - 150% of AISC allowables for structural steel (not to exceed yield stress). - 90% of yield stress for reinforcing steel. - 85% of ultimate stress for concrete to assure no loss of function and adequate factor of safety against collapse.[4]
	e.	D + F	4/3 of normal allowable stresses (AISC for structural steel, ACI for reinforced concrete).
	f.	D + A	Stresses do not exceed: 150% of AISC allowables for structural steel (not to exceed yield stress); 90% of yield stress for reinforcing steel; 85% of ultimate stress for concrete.

4. Class I structures were not analyzed to determine that their energy absorption capability was greater than the energy input due to the design basis (maximum) earthquake (not to exceed yield stress for structural steel).

5. Although it was an original design criteria that the concrete stresses not exceed 85% of the ultimate strength of concrete at 28 days for a specified design strength or class, the actual stress level of “in place” concrete is less than 70% of ultimate compressive strength based on actual cylinder breaks at 28 days for the concrete strength or class specified. Therefore, the actual structure is in good agreement with ultimate strength sections of the “(ACI) Building Code Requirement for Reinforced Concrete” (ACI--63) as listed in Table 12.2-1.

6. Although the Reactor Building steel superstructure is not a Class I structure, it is designed for this load combination to ensure that a failure of the superstructure structural members will not occur when exposed to this load combination.

Load Considerations	Allowable Stress
3. Reactor Vessel Supports	
a. D+H+R+E	Stresses remain within Code Allowables without the usual increase for earthquake loadings (AISC for structural steel, ACI for reinforced concrete).
b. D+H+R+E’	Stresses do not exceed: - 150% of AISC allowable for structural steel. - 90% of yield stress for reinforcing steel. - 85% of ultimate stress for concrete.[7]
4. Reactor Vessel Internals[8]	
a. D+E	Stresses which occur as a result of the maximum possible combination of loadings encountered in operational conditions are within the stress criteria of ASME, Section III Class A Vessel (Reference 30).
b. D+E’	The secondary and primary plus secondary stresses are examined on a rational basis taking into account elastic and plastic strains. These strains are limited to preclude failure by deformation which would compromise any of the engineered safeguards or prevent safe shutdown of the reactor.
c. D+P	Primary stresses are within the stress criteria of ASME, Section III, Class A (Reference 30). The secondary and the primary plus secondary stresses are examined on a rational basis taking into account elastic and plastic strains. These strains are limited to preclude failure by deformation which would compromise any of the engineered safeguards or prevent safe shutdown of the reactor.

7. Although it was an original design criteria that the concrete stresses not exceed 85% of the ultimate strength of concrete at 28 days for a specified design strength or class, the actual stress level of “in place” concrete is less than 70% of ultimate compressive strength based on actual cylinder breaks at 28 days for the concrete strength or class specified. Therefore, the actual structure is in good agreement with ultimate strength sections of the “(ACI) Building Code Requirement for Reinforced Concrete” (ACI-318-63) (Reference 31) as listed in Table 12.2-1.

8. For information on the loads that were analyzed for the Steam Dryer assembly, see Reference 95 in USAR section 3.7.

In response to NRC questions, further expansion of item 4 above is included as follows:

There are no specific strain limits in connection with primary type loadings because stress limits are used and are believed to be more appropriate to primary loadings.

For accident conditions, a fatigue analysis is performed to account for large secondary strains (i.e., those produced by thermal stress). The ASME fatigue curves are extrapolated to one cycle such that the strain corresponding to one fatigue cycle is the implied limiting strain. The total fatigue usage is never permitted to exceed a value of 1.0. This implies a factor of safety of 2 on stress and 20 on cycles when using the ASME fatigue curves for the material in question.

Two limiting criteria are considered in the design of reactor internals which negate the need for specific strain limits. These are deflection limits and plastic instability limits described below. In the Monticello case neither criteria was used to the limit permitted by the criteria.

The deflection limit requires that maximum permissible deformation under combination loading be limited to 80% of the loss of function (LOF) deformation (calculated on a conservative basis). The LOF deformation is that deformation which could compromise an engineered safeguards system or could otherwise jeopardize safe shutdown of the reactor. As a practical matter, the stresses in most of the critical reactor internal components are so low that these deformation limits are not invoked.

When combination loading stresses do exceed the yield stress, the General Electric plastic instability design criteria permits a maximum load equal to 80% of the plastic instability load. This criteria is more conservative than Section III of the ASME Code 1965 Edition which permits 90% of the plastic instability load. Using the G.E. criterion the strain corresponding to this load varies from about 10% (non-strain hardening materials) to about 35% (strain hardening materials) of the ultimate strain at temperature as determined by standard ASTM tensile tests. It has not been necessary to use this criterion on Monticello, however the method does represent the upper bound of strain permitted within the criterion. Primary stresses due to fault conditions are limited for design purposes to 2 S_m under combination loading. Since S_m implies a minimum factor of safety of 3 (e.g., S_m < 1/3 of the ultimate) the minimum factor of safety on load obtained from this criterion would be 1.5.

As indicated in Section 3.4.3.2 the most significant strain to occur in any reactor internal component occurs in the shroud to shroud support plate region and is primarily caused by thermal gradients and temperature differences which occur as the core is reflooded following a design basis accident. Since the predominant loads are secondary (thermal) and occur less than 250 times, the stresses are calculated on an elastic basis (even though yield stress is exceeded) and a plastic fatigue analysis performed on the components as outlined in an ASME paper (Reference 1). The energy criterion does not apply to reactor internals.

Load Considerations		Allowable Stress
5. Emergency Core Cooling System (ECCS)		
a.	D + E	Stresses remain within code allowable
b.	D + E’	Stresses remain within minimum yield point of material.

The design codes used to govern the construction documents are listed in Table 12.2-1. Additional loads considered in the containment design, torus attached piping, SRV discharge lines, and ECCS suction header, as a result of the Mark-I Containment Program are described in the Monticello Mark I Containment Long Term Unique Analysis Reports (See References 72 and 75 of Section 5).

12.2.1.5 Live Loads

These live loads are not necessarily the expected live loads to be used for the load “D” in Section 12.2.1.4, but are the design live loads that were used during the original design of the structures. The 50 psf snow load is a ground snow load. The actual snow load on the structure must take into account the roof geometry and the features of the surrounding area.

Live loads used in the design of Class I structures, in general, are as follows:

Roof snow load	50 psf
Concrete floor loading	200 psf (Minimum)
Grating floor loading	150 psf
Lay down areas on Reactor Bldg. Refueling floor	1000 psf*
Stairways and walkways	100 psf
Control room	200 psf
Cable spreading room	100 psf
Emergency Diesel Generator Building Second Floor, Elevation 949 foot	100 psf
Off-gas Stack. Floor Slab, at Elevation 971-foot 6-inch	100 psf

***NOTE:** 1000 psf live load is not concurrent with spent fuel cask laydown in the cask decontamination area.

12.2.1.6 Wind Loads

Wind loads used in the design of Class I and Class II structures are based on a maximum wind velocity of 100 miles per hour, 30 feet above ground, in accordance with ASCE paper 3269 (Reference 32). Wind velocities are permitted to vary with height in accordance with Table 1 (a) of ASCE paper 3269. A gust factor of 1.1 was used.

12.2.1.7 Flooding

12.2.1.7.1 External Flooding

The plant site natural grade level is at elevation 930 ft msl. The maximum recorded high water level at the site was 916 ft msl. The 1000-year projected high water level is elevation 920 ft msl and all Class I and Class II structures have been designed for a high water level of elevation 930 ft msl.

Although significant meteorological antecedent conditions (as described in Appendix G) must be present for a Probable Maximum Flood to occur in the spring, flooding conditions could develop throughout the year. As such, a plan is required to be in place at all times and able to be implemented that will protect the plant from an external flooding event up to and including the PMF which peaks in 12 days at elevation 939.2 feet MSL.

Flood protection is provided to prevent flooding or flood damage to Class I Structures, Class II structures housing Class I equipment and Radwaste Building in the event of the Probable Maximum Flood as defined in Section 2.4.1.

For Flood protection below elevation 930 feet, installation of flood protection features (such as pumps and steel plates, grout, or sandbags to close openings up to elevation 930 feet) provides flood protection for Class I structures and Class II structures housing Class I equipment. Suitable steel plates are stored at the plant for possible future use.

For Flood protection above 930.0 feet, a levee consisting of an earthen levee and bin wall earthen levee is constructed around the Class I structures (excluding the Off-Gas Stack), Class II structures housing Class I equipment (excluding Off-Gas Storage Building), and Radwaste Building to protect them from the effects of a flood. The Off-Gas Stack is outside the boundary of the levee and protected by steel plates and sandbags. The Off-Gas Storage Building is excluded because the only areas that house Class I components are the Fan and Foyer Rooms for Stand-By Gas Treatment and the components are located at an elevation above the PMF. Construction of the levee will stay ahead of river level and adequately protect the plant. Installation of flood protection features (such as pumps and steel plates, grout, or sandbags to close openings above elevation 930.0 feet) provides additional flood protection.

Installation of flood protection features (such as pumps and steel plates, grout, or sandbags to close openings) and construction of the levee would be done at the time of the flood since about 12 days are available from the onset of the conditions leading to the PMF and the time the peak stage would be reached at the site.

The flood protection design and procedures address floods to the Probable Maximum Flood elevation of 939.2 foot msl (Reference 24).

For the design flood of 930 ft msl no increase in allowable stress was permitted in the design of buildings to withstand hydrostatic loadings. For the Probable Maximum Flood stage of elevation 939.2 ft msl, a 1/3 increase in allowable stress was permitted. All structures with the exception of the diesel generator building are sufficiently heavy to resist buoyancy, and the stresses do not exceed the allowable defined above. In order to protect equipment housed within the diesel generator building, the structure is protected from the effects of flood waters by construction of the levee. A factor of safety of approximately 1.5 is maintained against buoyancy.

The emergency diesel oil storage tank has been evaluated (in accordance with the ASME Boiler and Pressure Vessel Code Section III, Subsection NB, 1986 Edition) as acceptable for a flood level up to 932 feet msl with a minimum of 2 feet of fuel oil maintained in the tank. Above elevation 932 feet and up to the Probable Maximum Flood elevation, flood protection is provided by construction of the levee. A factor of safety of 1.5 is maintained against buoyancy.

12.2.1.7.2 Internal Flooding

An evaluation of the possibility that flooding could prevent safe shutdown of the reactor or prevent engineered safety systems from performing their emergency function following postulated accidents was completed for lines not classified as high energy lines (References 21, 22, 23 and 26). Flooding from rupture of high energy lines is discussed in Appendix I.

Emergency power to engineered safety systems is available from two independent sources, one located at the 911 foot elevation and the other at the 931 foot elevation in the turbine building. There is adequate separation of redundant components of safeguards equipment so that no single failure of non-Class I piping could prevent safe reactor shutdown or emergency core cooling.

Modifications were completed to reduce the potential for adverse interactions. The modifications included:

a. Flood probes in the condenser hotwell area and in the circulating water pump bay with alarms in the control room and automatic circulating water pump trip. This provides protection against a major breach of the circulating water system below grade. Rubber expansion joints at four locations on each of the two lines are used.

b. The condensate transfer lines in the RHR-Core Spray pump rooms and the HPCI pump room were analyzed and modified to meet design requirements for Class I piping.

Redundant trains of engineered safeguards equipment are not coincidentally affected by flooding or spraying damage from postulated non-Class I line failures. Failures of non-Class I piping may not be postulated if the piping meets the seismic design requirements for Class I piping. Multiple component failures can be postulated to result from flooding or spraying damage to emergency switchgear and motor control centers. These interactions were reviewed and it was determined that no corrective actions were required to ensure availability of core cooling.

12.2.1.8 Tornado Loading

Tornado loading conditions are as follows:

a. A rotational wind having a tangential velocity of 300 mph.

b. Differential pressure between inside and outside enclosed areas - 2 psi.

c. A torsional moment resulting from applying the wind specified in (a) above, only on one-half of the structure.

In addition, those areas housing critical equipment required to assure safe shutdown of the reactor were designed to prevent penetration of exterior walls from the following two types of missiles that could be generated by a tornado:

a. A utility pole 35-feet long by 14-inches in diameter and a unit weight of 35 lbs per cubic foot having a velocity of 200 mph.

b. A 1 ton missile, such as a compact type automobile traveling at 100 mph at a maximum height of 25-feet above grade and with a contact area of 25 sq-ft.

Discussion of the expected effects of tornadoes on the fuel storage pool is included in a topical report (Reference 2). Subsequently, it was determined that the drywell head could become a missile hazard for the spent fuel pool, however, since the probability is less than 10E-7, it is not a credible event.

Although the Reactor Building steel superstructure is not a Class I structure, it is designed to ensure that a failure of the superstructure members will not occur when exposed to tornado wind loading.

12.2.1.9 Seismic Loads

Seismic loads were based upon the seismic investigation and data developed by John A. Blume & Associates, Engineers. The design earthquake established for this site is the North 69° West Component of the 1952 Taft earthquake, normalized to a maximum ground acceleration of 0.06 gravity. For the emergency filtration train building, synthetic time-history associated with Regulatory Guide 1.60 (Reference 33) normalized to a maximum ground acceleration of 0.06 gravity was used as input to the seismic analysis.

Figure 12.2-9 shows a comparison between the Taft earthquake spectra and the response acceleration spectra for 2% damping shown in Appendix A. Recommended Earthquake Design Procedures, Plate 3. Housner's average 2% spectrum (normalized to 0.06 g) is also shown for comparison purposes. A ten second length of the time record was used in preparing Figure 12.2-9. A comparison has been made with a spectrum obtained from a 12 second record and no significant difference was observed, therefore, it is concluded that the curve of Figure 12.2-9 represents a 12-second length of record.

For the design of Class I structures and equipment the maximum horizontal acceleration and the maximum vertical acceleration were considered simultaneously. Where applicable, the resulting seismic stresses for the two motions were combined linearly. The vertical acceleration was taken as 2/3 of the horizontal ground acceleration. Class II structures have been designed with seismic coefficients in accordance with the Uniform Building Code for Zone I (Reference 29). Class II equipment has been designed for a seismic coefficient of 0.05 g.

Although Class II structures were not subjected to a dynamic analysis, those portions of Class II structures which provide support and protection for Class I equipment were designed in accordance with design criteria for Class I structures as specified in Sections 12.2.1.4 through 12.2.1.7.

Those portions of Class II structures which enclose and/or support Class I equipment are located at elevations which are either at or below grade. Although experience indicates that the maximum ground acceleration of the operating basis earthquake may safely be applied to those integral portions of structures located either at or below grade, seismic forces were determined in the following manner. The Class II structure was compared to the Reactor Building or other Class I structure which had been subjected to a dynamic analysis. Horizontal accelerations were selected from the maximum absolute acceleration curve for the Class I structure at elevations equivalent to the elevation of those portions of the Class II structure enclosing or supporting Class I equipment. The selected horizontal accelerations were then applied to the portions of the Class II structures of concern and the seismic forces determined. Where applicable, forces due to the selected horizontal acceleration and vertical acceleration (2/3 maximum ground acceleration) were considered to act simultaneously and the resulting stresses were combined linearly. Those portions of Class II structures housing Class I equipment were reviewed to assure that a safe shutdown can be made with the structure subjected to ground accelerations (0.12 g) of the design basis (maximum) earthquake. For the design condition of the design basis earthquake the selected accelerations were doubled, forces determined and stresses combined as described for the operating basis earthquake. The entire structure was reviewed to assure the structural integrity of those portions of the structure necessary for the protection and support of Class I equipment and that stresses were within the allowable limits as specified for Class I structures in Section 12.2.1.4 for both the operating basis and design basis earthquake.

A time-history analysis has been made on the following structures and equipment:

a. Reactor Building (Including the HPCI Building)

b. Drywell

c. Reactor Pressure Vessel

d. Main Control Room

e. Off-gas Stack

f. Emergency Filtration Train Building

When the time-history analysis was made, periods were examined and the corresponding spectral accelerations from both the time-history and response acceleration spectra were compared. If substantial differences were noted, the model was modified and a reanalysis was made. In this manner it was possible to avoid using spectral accelerations that were in a valley of the Time-History Spectra. If periods of one or more of the system's modes fell at a valley on the response spectrum produced by the time-history, it was determined to what extent the spectral acceleration, read at this valley, underestimates the spectral acceleration read from the design spectrum for this period. If this underestimation was judged to be excessive, (there is no simple rule for making this judgment) the modulus of elasticity, spring constants, or the systems period itself would be changed so as to move the period of the mode(s) involved away from the valley of the time-history produced spectrum. The period(s) would be changed enough to produce approximately the same spectral acceleration whether read from the design spectrum or time-history produced spectrum.

An examination (October 1969) of modal periods for which a time-history analysis was used on this facility indicates that there was no case where the periods fell at the valley of the time-history spectrum. Therefore, no modifications to the model were required and none were made. The evaluation of modal periods indicated that the time-history analyses are conservative when compared to the design response spectrum accelerations.

All rigid Class I equipment was analyzed using accelerations derived from the results of the analysis for the supporting structure at the appropriate elevation.

Amplification factors were applied for the seismic analysis of non rigidly mounted equipment. Typical amplification factors were:

1.	Reactor Pressure Vessel	2.7
2.	Recirculating Pump	1.5

The amplification factors listed were determined by using the results of the dynamic analysis; i.e., referring to Sheet No. 4, Earthquake Analysis, Reactor Pressure Vessel, Appendix A, the maximum acceleration of the top of the reactor vessel is 0.16 g, since ground acceleration is 0.06 g the amplification factor is 2.7. The other amplification factors were calculated in a similar manner.

A response spectrum method of analysis has been made on the following equipment and piping systems:

	System	Number of Modes	Damping
1.	Recirculation Lines	*	0.5%
2.	Pressure Suppression Chamber	1	1.0%
3.	20-Inch Suction Header	*	0.5%

* All modes evaluated up to 33 Hz.

The damping values used above are the values recommended in Table 1, "Recommended Earthquake Criteria," as included in Appendix A. The use of 1.0% for the pressure suppression chamber is more conservative than the 2.0% recommended as a criteria for steel structures.

Only the fundamental mode was calculated and used to determine the response of the suppression chamber to seismic forces.

The recirculation piping was modeled as a lumped mass system with enough details to accurately predict results for piping dynamic response to 33 Hz. The weight of piping content plus insulation was added to the weight of the pipe in the form of a uniformly distributed load (lbs/ft). For the pump motors and valve operators the extended mass was modeled as an additional weight at their respective center of gravity. The stiffness of each support, either from hand calculation or computer analysis, was included in the modeling.

Bechtel Power Corporation stress report no. SR-10040-SS 1, Rev 2 (Reference 10), presents the stress analysis for the Recirculation System Piping.

The design of the Recirculation System was in accordance with the ANSI B31.1 Code, 1977 Edition through Summer 1978 Addenda (Reference 34). To validate the design, stress analysis was performed in accordance with the ANSI B31.1 Code. However, the Nuclear Class 1 requirements have been verified in accordance with the rules of NB-3600 of the ASME Section III Boiler and Pressure Vessel Code 1980 Edition including Addenda through Summer, 1982 (Reference 35). Design conditions for level A, B, C, and D service limits were considered in the analysis.

The primary loadings considered are weight, pressure, earthquake, and other design mechanical loads. All the loads classified as Level A and Level B service limits, including thermal expansion range, thermal gradients (T_1, T_2), thermal gross discontinuity (a^Ta-b^Tb), earthquake, and other mechanical loads are included in calculating both primary and secondary stress intensity range and peak stress intensity range. Stress intensity ranges in Table NB-3681(a)-1 shall be used in qualification of all the piping products and joints in this system. The following shall be evaluated in the analysis of the piping system:

a. Pressure design shall be in accordance with rules in Section III, NB-3640 to ensure minimum wall thickness required to sustain the internal design pressure has been achieved.

b. Primary stress intensity limit of Equation (9) in NB-3652 shall be met for design condition level A, B, C, and D service limits.

c. Primary plus secondary stress intensity range limit shall be met by satisfying the requirement of Equation (10) in NB-3653.1. If the stress intensity range calculated by Equation (10) exceeds 3 S_m, the simplified Elastic-Plastic Analysis (Equations 12 and 13) shall be performed to qualify the piping system.

d. The cumulative damage shall be evaluated in accordance with NB-3653.5 based on Equation 11 (Peak Stress Intensity Range) and Equation 14 (Alternate Stress Intensity) in NB-3653.2 and NB-3653.6(c) respectively.

All of the Earthquake Analysis Reports listed in Appendix A were made using the seismic criteria given in the Facility Description and Safety Analysis Report (FDSAR) and amendments. These reports gave shears, moments, and displacements which were used in the final design of the Class I structures, equipment, and piping.

12.2.1.10 Class I Piping Seismic Analyses

Class I piping seismic analyses were performed for both operating basis and design basis (maximum) earthquakes as follows:

a. Mode superposition using a floor response spectra.

b. A static analysis was made using conservative static seismic coefficients. These static coefficients were determined in the following manner:

1. Horizontal static coefficients were determined by using the average of the peak values from the unsmoothed ground spectral curve of the normalized earthquake.

2. This average acceleration was then multiplied by the ratio of the building response acceleration at the installed elevation of the piping to maximum ground acceleration.

$$Cp = A\ peak \quad \frac{A\ building}{A\ ground}$$

where:

Cp	=	Static coefficient for the Piping System
A peak	=	Average acceleration of the peak from the ground spectral curve
A building	=	Acceleration of the building at the installed elevation of the piping system
A ground	=	Acceleration of the ground

c. A vertical coefficient was taken at a constant value equal to two-thirds of the maximum base ground acceleration or 0.04 g.

The suction header has been analyzed as an extension of the primary containment pressure boundary. The suction header pipe and penetration nozzles are designed for the load combinations of the containment with maximum stresses limited in accordance with the ASME Code, Section III, Class MC (Reference 36).

The maximum stresses in the ECCS suction header pipe occur at the suppression chamber nozzle penetration to header intersection in the header. The maximum general membrane stress intensity ($P_{m'}$ OBE seismic included) is 6,020 psi. The Class MC allowable is 19,300 psi. The maximum local membrane stress intensity is 7,960 psi ($P_{b'}$ SSE seismic included). The Class MC allowable is 29,000 psi. The maximum secondary stress (P_b + Q, where Q is a through wall bending stress) in the ECCS suction header is 23,700 psi. The Class MC allowable is 67,500 psi.

The nozzle/insert plate assembly for the ECCS suction header suppression chamber penetration has been reinforced to withstand the effects of the Mark I loads. After the modification, the maximum membrane and membrane plus bending stress intensities in the suppression chamber shell were calculated to be 24,300 psi and 44,600 psi. The Class MC allowables are 29,000 psi and 67,800 psi, respectively. The maximum membrane and membrane plus bending stress intensities in the suppression chamber nozzle were calculated to be 6,354 psi and 55,070 psi. The Class MC allowables are 29,000 psi and 67,800 psi, respectively.

Methods for calculating the deformation are not realistic for material stressed beyond elastic limits, therefore deformation was not calculated for the above loading conditions. The ASME code recognizes that high localized and secondary bending stresses will not cause failure under these loading conditions. The basis for calculating all stresses, including secondary and bending stresses which are beyond yield, are in accordance with the ASME code and are not guided by a deformation limit.

Instrumentation Piping Curves

Lateral deflection and force evaluation curves for piping systems were developed by John A. Blume and Associates for Class I instrument piping. The curves provide approximate guidelines for the evaluation of the lateral supports in a piping system. The pipes were considered filled with water and the wall thickness or schedule number was shown on the graph. The modulus of elasticity is 29 x 10^6 psi. The curves were based on a single span with pinned-pinned end conditions.

The use of piping curves was as follows:

1. Knowing the period of the supporting building or structure, the period of the piping as to when it is rigid, flexible, or resonant was established.

Rigid: $$\frac{\text{Period of Structure } (T_b)}{\text{Period of Piping } (T_p)} > 2.0$$

Flexible: $$\frac{\text{Period of Structure } (T_b)}{\text{Period of Piping } (T_p)} < 0.7$$

Resonant: $$0.7 \leq \frac{\text{Period of Structure } (T_b)}{\text{Period of Piping } (T_p)} \leq 2.0$$

2. The maximum spans for various diameters of piping to carry a load of 0.5g and not be stressed more than 1500 psi was established. See Table 121.1.4 in Power Piping USAS B31.1.0,1967 (Reference 37).

3. The resonant limits were established for various diameters of piping by using Blume's curves giving natural periods as a function of pipe size and span.

4. After the span was selected, the maximum deflection and the reaction on the supports was determined.

5. The displacement and support reactions were increased by a factor of 3 due to magnification of the equipment over ground acceleration.

6. Spans were reduced by a factor of 2 to account for valves or branch lines. For 90-degree bends, either leg is not more than L/2 where L is 3/4 of the allowable span.

12.2.1.10.1 Original Equipment and Piping Dynamic Analysis

Supports in general were located in a manner so as to be out of the resonant range.

When a dynamic analysis was made, Floor Response Spectra were used.

For the response spectrum analysis of piping systems, the floor spectra near the points of pipe lateral restraint were considered. The spectrum usually selected to be used in the analysis was the one located nearest the point of lateral support of the majority of the mass of the pipe. For the recirculation lines, the spectrum used was the one occurring just above the elevation of the header, or about half way between the upper and lower elevation of the pipe. Most of the seismic restraints fall below this elevation, and the selection of the point was considered to be realistic for the seismic analysis.

In the response spectrum method of analysis of pipes, the inertia loads were applied mode by mode in the direction of the mode shape. Directional and reversed effects are taken into account by the mode shape. Thus, for the appropriate mode shape, the inertia forces would be reversed on adjacent spans. The inertia loads are thereby applied in a manner to represent the actual conditions of vibration.

When a static analysis was made, all piping systems above the 935 foot elevation used a horizontal static coefficient, 0.82g, and below this elevation a value of 0.53g was used. These values represent an amplification factor of 13 and 9 respectively.

In response to concerns of IE Bulletin 79-14 (Reference 38), an inspection survey of as-built safety-related piping systems was conducted. An inspection was conducted to verify that the input information for the seismic analysis of safety-related piping systems reflect as-built configuration. All discrepancies identified in the inspection were evaluated for their effects on system operability, and were re-analyzed to demonstrate compliance with the original design requirements. Where non-compliances are found, repairs and modifications were made accordingly to meet original design requirements.

12.2.1.10.2 Current Analysis of Equipment and Piping

Addition of new systems or re-evaluation of existing systems is done using current methods of analysis. In a typical seismic analysis the piping system will be modeled as a lumped mass system. The analysis will be performed using the response spectrum modal superposition technique. Modes of the piping system up to a frequency corresponding to 33 hertz or the minimum zero period acceleration frequency of the response spectra shall be considered in determining the responses of the system. Modal responses of the piping system shall be combined by the square root-sum-of-the-squares (SRSS) technique. A damping value of one-half percent of critical damping shall be used for OBE and SSE earthquakes with SSE loads determined by doubling OBE inertia loads.

The vertical direction earthquake shall be considered using a static coefficient of .04g for OBE. Vertical direction earthquake loads for SSE shall be taken as twice those for OBE. The two horizontal direction earthquakes are considered to act independently and each are absolutely summed with the vertical direction earthquake.

Valves and other in-line equipment will be modeled as lumped masses in the piping model. The models will be complete between anchors, or other appropriate termination points, or in some cases separated at locations where piping cross-sectional moment of inertia ratios between the systems are equal to or greater than 40. If the system models are separated, the effects of anchor displacements on the smaller system will be evaluated by applying the anchor displacement of the larger system to the smaller system.

The supports of a system typically are modeled in the piping model with appropriate support stiffnesses.

Design of piping supports and evaluation of stresses in the piping supports shall be in accordance with design code limits. The piping supports shall be evaluated for the load combinations and stress limits defined below.

D+E’	allowables given in the design code or the standard component support allowable as applicable.

The effects of non-safety related piping sharing a support with safety related piping shall be evaluated.

Flexibility of the base plate shall be considered in determining the anchor bolt loads as required by the NRC IE Bulletin 79-02 (Reference 39). Criteria for the interim acceptability of plant operation with less than the design factors of safety for piping supports due to as-built problems, under design, base plate flexibility or anchor bolt deficiencies was provided in NRC IE Bulletin No. 79-02, Supplement 1 (Reference 40).

NRC Regulatory Guide 1.199 (Reference 96) provides methodology acceptable to the NRC for the design, installation, and quality assurance of anchors (steel embedments) used for components and structural supports on concrete structures. This method shall be used for safety related pipe support base plate anchors whose design basis is provided by the ACI Concrete Capacity Design (CCD) Method, also known as the Strength Method. RG 1.199 shall be used in its entirety including the RG NRC positions applied to ACI 349-01 Appendix B methodology.

All modifications or replacements are governed by the ASME Section XI Code.

On February 19, 1987, the NRC issued Generic Letter 87-02, “Verification of Seismic Adequacy of Mechanical and Electrical Equipment in Operating Reactors, Unresolved Safety Issue (USI) A-46” (Reference 41). This Generic Letter encouraged utilities to participate in a generic program to resolve the seismic verification issues associated with unresolved Safety Issue (USI) A-46. As a result, the Seismic Qualification Utility Group (SQUG) developed the “Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment”.

On May 22, 1992, the NRC Staff issued Generic Letter 87-02 Supplement 1 (Reference 43), which constituted the NRC Staff’s review of the GIP and which included Supplemental Safety Evaluation Report Number 2 (SSER-2) on the GIP, Revision 2 (Reference 44), corrected on February 14, 1992. Supplement 1 to Generic Letter 87-02 requested licensees provide information concerning implementation of the guidance provided in the SQUG GIP and a report to the NRC staff summarizing the results of the plant specific USI-46 review. By letter dated September 21, 1992 (Reference 45), Monticello responded to Generic Letter 87-02, Supplement 1. The NRC staff provided a safety evaluation of Monticello’s response by letter dated December 10, 1992 (Reference 46). By letter dated November 20, 1995 (Reference 47), Monticello provided information summarizing the results of the plant specific USI-46 review as requested by Generic Letter 87-02, Supplement 1.

The methodology for the verification of the seismic adequacy of equipment that has been developed by the Seismic Qualification Utilities Group (SQUG) for the resolution of Generic Letter 87-02 is documented in the Generic Implementation Procedure (GIP). The methodology used in the GIP is based on earthquake experience data supplemented by test data and analysis. This methodology, as presented in the GIP, is an acceptable methodology (Reference 43) and may be used for the verification of the seismic adequacy of new and replacement equipment for which there is no previous license commitment for seismic qualification. Plant specific changes made in accordance with the GIP were found acceptable to the NRC (Reference 86).

12.2.1.11 Foundation Design and Construction

A foundation investigation of the site was performed by Dames and Moore, Consultants in Applied Earth Sciences. Their analysis concluded that the reactor building, the turbine building and other structures could be supported on mat foundations. The resulting foundation designs were based on the soil bearing values recommended by Dames and Moore.

The soil borings indicated that a layer of stiff clay existed near the bottom elevation of the mat for the reactor building. This clay layer was confirmed during construction and was removed to the underlying dense sand and gravel. The clay layer was replaced with compacted granular fill.

The foundation for the turbine building was above the clay layer and was designed for a lower bearing value than the reactor mat. Because of this, the clay layer was left in place.

The entire excavation for the turbine building foundation and reactor building foundation was dewatered using deep wells. Additional localized dewatering was accomplished with french drains. No unusual or unforeseen foundation construction problems were encountered. The difference in elevation between the reactor building mat and turbine building mat was accommodated with a concrete retaining wall which was left in place.

When the foundation excavation was completed, a 4-inch thick concrete base or mud mat was poured to provide a working surface. A plastic waterproofing membrane was placed over the mud mat prior to pouring the foundation. The foundation was formed and poured in sections. Waterstops were placed in all construction joints between pour sections.

A survey traverse of points on the buildings was established to monitor foundation settlement. This survey determined that settlement was uniform and within the predicted values.

The intake structure foundation rests on a lean concrete fill which rests on bed rock. The bed rock is a sandstone which is more than adequate for this foundation. Since the intake rests on bed rock it was used as a bench mark for the foundation monitoring mentioned above. The stack and control and cable spreading building are also on mat foundations. The emergency diesel generator building is constructed on a continuous footing foundation. These foundations are near grade, and are designed for appropriately low bearing values.

The emergency filtration train building rests on a combination of mat and caisson foundation. The subsurface investigation was performed by Twin City Testing of St. Paul, Minnesota.

12.2.1.12 Masonry Wall Design

In response to NRC IE Bulletin No. 80-11 (Reference 48), an investigation was performed to identify all masonry walls which are in proximity to or have attachments from safety related piping or equipment, the failure of which could affect a safety related system. Reevaluation of identified masonry walls was performed to demonstrate that an SSE or postulated accident will not cause failure to the extent that functions of safety related items are impaired. Verification of wall adequacy took into account support condition, global response of wall, and local transfer of load.

The investigation identified two walls with unacceptable stress levels (Walls 212 and 222). These deficiencies were corrected with the addition of a truss (Wall 212), and braces (Wall 222). During the field survey, two other walls were found to be subject to jet impingement (Walls T311 and T322). Shields designed to carry the full load from the jet forces were provided at the postulated pipe breaks to prevent these walls from collapsing. In 1986, additional concerns regarding High Energy Line Break (HELB) in the feedwater suction and discharge lines were raised and the effect on walls T311 and T322. Room pressurization from a break would blow out the walls. The solution involved seismic analysis of piping line segments to eliminate the need to consider postulated break locations from this area. As a result of the 1986 HELB work, the jet impingement shields installed for IE Bulletin 80-11 are no longer required and they have been abandoned in place or have been removed. Refer to Appendix I of the USAR for detailed description.

Additional background information on the subject of masonry wall design is given in Reference 6.

The NRC's Safety Evaluation Report (SER) (Reference 9) concluded that IE Bulletin 80-11 has been fully implemented at Monticello.

12.2.2 Structures and Equipment

12.2.2.1 Reactor Building

The principal function of the reactor building, which houses the reactor and associated equipment, are to support and protect enclosed systems and components and to provide secondary containment limiting the offsite radiological consequences of accidents. The building provides necessary space for the equipment in a planned arrangement and provides for layout space for the equipment to be removed and replaced if necessary. Reactor internals and fuel can be moved and conveniently stored within the building. The buildings' containment function is further discussed in Section 5.3.

12.2.2.1.1 Structure Description

The reactor building completely encloses the primary containment. It is a reinforced concrete structure from the foundation at elevation 888-foot 3-inch to the refueling floor at elevation 1027-foot 8-inch Steel siding extends from the refueling floor level to 1074-foot 2-inch, the top of the building. The structure is supported by a reinforced concrete mat of 8-foot minimum thickness, which is founded on medium sand with some gravel. A subsurface investigation of the site was performed by Dames and Moore, Consultants in Applied Earth Sciences, who established the adequacy of the soil to support the building pressure.

Values for the vertical and lateral foundation spring supports are given in Appendix A, "Earthquake Analysis: Reactor Building", page 3. The use of these foundation spring supports in the seismic analysis is explained on pages 6 and 7 under "Remarks on the Computer Program" of this same report.

The resulting shears, moments and displacements from this seismic analysis were used in the design of the reactor building. The soil springs were assumed to be elastic-linear rather than non-linear.

A one inch gap separates the Reactor Building and Turbine Building foundation retaining wall, the turbine building foundation mat and concrete super-structure. The one inch separation is filled with a premolded filler to the 951-foot elevation.

Under seismic acceleration of the design earthquake the displacement diagram for the Reactor Building indicates that the total maximum displacement of the building in N-S direction would be approximately 200 mils at the 951 foot elevation.

For the maximum earthquake (0.12 g) the displacement of the referenced elevation would be approximately 400 mils. Since the Turbine Building is a Class II structure, a dynamic analysis was not made. However, it is safe to assume that the Turbine Building displacement at the top of the concrete superstructure, 951 foot elevation, would not exceed that of the Reactor Building at the equivalent elevation. Therefore, for both the design and maximum earthquake, the one-inch separation between buildings is adequate.

The primary containment, an integral part of the structure, occupies the central portion of the building. The reactor, a field erected vessel, was supported on a steel cylinder designed to carry the empty weight of the vessel. Subsequent loads were designed to be transmitted from the support cylinder, through shear rings, into a reinforced concrete pedestal erected after vessel installation. The steel cylinder supporting the reactor vessel extends through the drywell into the foundation and thereby fixes the drywell to the foundation. The interior face of the reactor support cylinder was coated with a 3-inch Gunnite lining for protection against differential expansion of the steel liner and concrete pedestal in the event of rapid changes in temperature. See Figure 12.2-10 for a cross section of the drywell and its foundation.

Access to the drywell is through a personnel lock and equipment hatch onto a platform at elevation 933-foot 6-inch. An additional platform at elevation 951-foot 6-inch supports a 16 ton monorail used in removing the recirculation pumps and other equipment.

Surrounding the reactor vessel and supported on the reactor vessel pedestal at elevation 947-foot 3-inch is the biological shield whose primary function is to protect equipment inside the drywell against radiation and thermal effects. The structure is capable of transmitting loads due to seismic and jet forces acting on it. The biological shield is composed of two steel cylinders interconnected with 27 WF (177 lb/ft) columns and is filled with concrete. Because of the radiation and temperature effects on the concrete only the lower 12 feet of concrete, up to the 959 foot elevation, has been designed as structural concrete capable of resisting forces and shears. Above the 959 foot elevation the two steel cylinders and 27 WF columns are structurally adequate and the concrete fill has not been considered as adding to the support.

Investigation made by General Electric has shown that the shield wall strength is more than adequate to withstand the pressure that could be developed by a failure of a nozzle safe end.

The biological shield wall, based on an allowable stress of 150% of the 1969 AISC allowable stress, has the capability of withstanding a uniform internal pressure of 58 psid. The annulus pressure load on the biological shield wall due to a postulated recirculation line break has been evaluated (Reference 104 and 111). The biological shield wall design is adequate as there is substantial margin between the peak predicted annulus pressure load of 41.7 psid and the biological shield wall structural design value of 58 psid.

New steel biological shield doors were provided for the 12 recirculation shield wall penetrations. The new steel doors are held in place by door hinges and additional locking pin devices which are designed to overcome the maximum biological shield wall compartment pressurization. The annulus pressure loads for a postulated design break are below the door design differential pressure of 54 psid (Reference 104 and 111).

Also contained within the reactor building are the systems and support facilities for the reactor. In addition, all refueling functions are provided inside the secondary containment. The new fuel storage vault and the spent fuel storage pool are located inside the reactor building.

Due to the modification of the spent fuel pool as discussed in Section 10, the structural capacity of the spent fuel pool floor was reanalyzed for the new High Density Fuel Storage System (HDFSS).

The spent fuel pool floor slab was analyzed as a two-way slab using the ACI Standard 318-71 Strength Design Method (Reference 50), with load combinations defined in Standard Review Plan (SRP) Section 3.8.4 (Reference 91). Structural capacities were calculated in accordance with ACI Standard 318-71 using a concrete strength of 6.4 ksi. The following load combinations were considered:

1. $U = 1.4D + 1.7L$

2. $U = 1.4D + 1.7L + 1.9E$

3. $U = 0.75\ (1.4D + 1.7L + 1.7T_0)$

4. $U = 0.75\ (1.4D + 1.7L + 1.9E + 1.7T_0)$

5. $U = 1.0D + 1.0L + 1.0TA + 1.0E'$

Where: D = dead load of the structure + hydrostatic + fuel racks + transfer cask

L = live load, as defined in step e. below. (For spent fuel pool floor slab, L = 0

T_0, T_A = effect of temperature differential, as defined in step f. below

E = OBE load, as defined in step g. below

E′ = SSE load, as defined in step g. below

The spent fuel pool perimeter walls were analyzed for normal loads as one-way slabs using the ACI Standard 318-71 Strength Design Method (Reference 50), with load combinations as defined above. In addition, the walls were analyzed for in-plane loads as deep girders, using the Working Stress Design Method per ACI Standard 318-71, Section 8.10 (Reference 50). The following load cases were considered:

1. D + L

2. $D + L + E' + T_A$

The applicable design loads used in evaluating the fuel pool structure include the following:

a. Dead load of the structure - weight of the slabs, walls, and other components permanently affixed thereon.

b. Hydrostatic load of fuel pool - The water level was assumed to extend to the top of the walls.

c. Fuel elements and racks - A design load of 2.7 ksf was assumed over the full floor area.

d. Cask weight - 199.8 kips (not concurrent with the design load of 2.7 ksf for the cask area).

e. Live load - Tributary floor live load of 0.2 ksf (Reactor Building floors at the 985-foot 6-inch, 1001-foot 2-inch and 1027-foot 8-inch elevations was used where applicable.

f. Thermal loads

    1. T_0 is due to temperature gradient of 72°F through slab and walls under operating conditions.

    2. T(sub A) is due to temperature gradient of 125°F through slab and walls under accident conditions.

g. Seismic loads

1. A1977 evaluation (Reference 92) included generation of a new response spectra for the fuel pool including vertical spectra.

	OBE	SSE
Lateral direction	0.12 g	0.26 g
Vertical direction	0.045 g	0.092 g

The design floor loading of the spent fuel pool was originally 2.0 ksf. Since the fully loaded HDFSS floor loading is 2.1 ksf, the structure was reanalyzed for a selected design floor loading of 2.7 ksf. The strength of concrete for the spent fuel pool has increased due to ‘aging’. Based on the Concrete Manual of Bureau of Reclamation, 8th Edition, 1975 (Reference 52), the Type II Portland Cement used at Monticello with a design strength of 4.0 ksi has a five year strength of 6.4 ksi. (The aged strength value used in the analysis is still less than the laboratory tests of 90 day strength which ranged from 6.57 to 7.12 ksi for samples of the spent fuel pool floor.) The pool floor slab was reanalyzed for the new floor loads, including bending moment and shear, and found acceptable for the new design floor capacity of 2.7 ksf using a concrete strength of 6.4 ksi.

The reactor service and refueling area is serviced by an overhead 105 ton bridge crane. The crane is capable of handling the drywell head, reactor vessel head, pool plugs and spent fuel cask. A refueling service platform, with necessary handling and grappling fixtures, services the refueling area and spent fuel storage pool.

The reactor building crane system was modified to incorporate redundant safety features which were not provided in the original design. The modification consists of furnishing a new trolley with redundant design features. The new trolley has an upgraded capacity of 105 tons rated load on the main hook with 100% redundancy and a conventional auxiliary hook with a five ton capacity. The new trolley was installed on the existing bridge and is capable of the same lifting envelope and lateral travel, in both directions, as the original trolley.

The integrity of the secondary containment is maintained through construction utilizing cast-in-place concrete exterior walls up to the refueling floor at elevation 1027-foot 8-inch. All access doors are gasketed and joints caulked. Above the refueling floor a steel framed structure which supports the bridge crane has been erected and is enclosed on all four sides with siding. The seams of the siding have been caulked on both the exterior and interior side of the inner sheet. The roof is sealed with a plastic vapor barrier. The location of joints and gaskets throughout the structure is such that in the event of increased leakage at some future date, replacement of gaskets and recaulking of joints is possible. As the reactor building is Class I up to the operating floor, secondary containment pressure (leakage) integrity is not required to withstand Class I seismic or tornado loads. Operation of secondary containment is not required during these Class I external events.

The reactor building was modified in order to accommodate the increased weight of the dry fuel storage casks. In particular, the slab over the southeast corner room (RHR A) was strengthened to support the wheel loads from the cask transfer trailer. The remaining slab on the 935′ elevation was qualified for the transfer trailer wheel loads. The beams supporting the slab under the cask decontamination/laydown area of the 1027′-8″ elevation were strengthened for the increased cask weight and cask welding equipment. The reactor building superstructure was modified for the increased loads associated with the increased crane capacity.

12.2.2.2 High Pressure Coolant Injection Building (HPCI)

The principal functions of this structure are to enclose the HPCI turbine and pumps and protect the equipment from weather, tornado and seismic effects. The building is a Class I structure and is part of the secondary containment of the reactor building.

12.2.2.2.1 Structure Description

The HPCI building is a reinforced concrete structure, constructed monolithically with the reactor building. The structure is supported by a reinforced concrete mat, 8 feet in thickness, which is an extension of the reactor building mat. The top of the mat is at elevation 896-foot 3-inch and the building extends to grade at the 935-foot elevation. Dimensions of the structure are 57 feet long, by 31-foot 6-inch wide, by 47-foot 9-inch high. A seismic analysis of this building was performed by John A. Blume and Associated, Engineers and incorporated in the dynamic analysis of the reactor building which is included in Appendix A.

12.2.2.3 Plant Control and Cable Spreading Structure and Administration Building

The plant control and cable spreading structure is located at the north end of the old office and control building.

The primary functions of this structure are to provide, under all operating or postulated accident conditions, safe enclosure for those portions of the standby electrical power systems and instrumentation and controls systems vital to overall plant operation and safety which are located therein and an environment satisfactory for continuous occupancy by operating personnel.

The main control room, cable spreading room, and battery room are located herein.

Modifications to the WEC have been made so that the office can be considered as part of the main control room for the purpose of meeting the NRC's requirement for the presence of a senior licensed operator in the control room at all times. The WEC is located immediately adjacent to the main control room but outside the previously defined control room boundary.

The administration building is located adjacent to the east side of the old office and control building.

The administration building was designed to provide a records storage area to meet the requirements of ANSI N45.2.9 (Reference 53), training space, lockers and restroom facilities, an instrument shop, library space, a meeting room, open office space, and private offices. The design meets or exceeds the requirements in the Quality Assurance Topical Report (QATR), NSPM-1, which govern.

Plans for the plant control and cable spreading structure and the administration building are shown on Section 15 Drawings NF-36055, 36055-1, 36056 and NF-36056-1, and the elevation on Section 15 Drawing NF-36063.

12.2.2.3.1 Structural Description

The plant control and cable spreading structure is a three story box-like reinforced structure. The three stories include a basement story partially below grade and two stories above grade. Basement floor is at the 928-foot elevation. Ground floor is approximately 4 feet above grade at the 939-foot elevation and is also the floor of the cable spreading room. The main control room floor is at the 951-foot elevation. Floors at the 939-foot and 951-foot elevations are reinforced concrete slabs supported on structural steel beam and girder framing. Reinforced concrete walls extend from the basement floor to the 963-foot elevation providing closure on four sides for the cable spreading and control rooms. The walls support the floor framing at the 939-foot and 951-foot elevations plus a 2-foot thick two-way reinforced concrete slab at the 965-foot elevation which spans and encloses the control room. The walls are based on a reinforced concrete mat-type foundation supported on select fill compacted to 100 percent of maximum dry density as determined by the American Association of State Highway Officials T180-57 Method of Compaction (Reference 54).

The administration building is a three story building with a full basement. The basement walls are reinforced concrete. The remainder of the building construction is structural steel frame. The basement floor is a concrete slab at the 928 foot elevation, the other three floors are reinforced concrete on steel.

12.2.2.3.2 Seismic Analysis and Design

The main control room and cable spreading structure was analyzed in accordance with recommended procedures for Class I Structures and Equipment referenced in Section 12.2.1.2. The method of analysis and analytical procedure is described; and maximum values of shear and moments are graphically presented in Appendix A, “Earthquake Analysis: Control Room.”

The portions of the old Office and Control Building containing the shift supervisor’s office extending above and south of the control and cable spreading structure are assumed to be rigidly attached to the structure at all floor and roof levels. Therefore, for the dynamic response analysis the equivalent mass system shown on Sheet No. 1 in the above portion of Appendix A includes the masses of the entire Office and Control Building lumped at the floor levels indicated.

The plant control and cable spreading structure is the primary lateral force resisting structure in the building and was designed to be self-sustaining with respect to the building as a whole. Thus, the walls of the structure were designed to resist the total design shear in proportion to their relative rigidity, and the foundation mat was designed to resist total overturning moments resulting therefrom.

The old office building is a Class II structure. The building houses a 10″ and 8″ diameter fire main. The structure was analyzed to withstand design basis earthquake in accordance with Class I design criteria set forth in Section 12.2.1.4 to prevent failure of the fire main and subsequent internal flooding of the battery rooms. The resulting forces and displacements from the dynamic analysis of the entire Office and Control Building were used in the analysis of the old office building.

The administration building addition is a Class II structure.

The administration building addition is designed for the wind and snow load criteria for the existing office building.

The administration building addition is designed for Uniform Building Code Zone 1 seismic acceleration. To assure that the new building does not adversely affect the control building structure, a two inch separating joint was left between the two structures. Therefore, during a SSE event the two buildings will not physically interact.

The administration building addition is designed to withstand a one hour fire. In addition the records storage area meets the guidelines of Regulatory Guide 1.88 (Reference 55).

12.2.2.3.3 Tornado Design

The plant control and cable spreading structure was designed to resist the effects of a tornado in accordance with the criteria set forth in Section 12.2.1.8, “Tornado Loading”.

12.2.2.4 Standby Diesel Generating Building

The principal function of this building is to provide a safe enclosure and protection for the standby diesel generators and portions of the power distribution systems enclosed therein.

12.2.2.4.1 Structure Description

The building is primarily a single story structure of reinforced concrete construction. A partial second story extends over a portion of the structure.

Vent paths, ventilation openings and portions of the standby diesel generators (intake and exhaust piping) that extend to beyond the reinforced concrete structure are enclosed or protected by several secondary steel structures of the building.

Ground floor is at the 931 foot elevation and consists of a concrete slab which is independent of the building structure and placed on compacted select fill. Exterior walls are of reinforced concrete and support the lower roof and second story framing at the 950 foot and 949 foot elevations respectively. The second story roof framing is at the 959 foot 6 inch elevation. The roof over the single story portion of the structure and over the penthouse consists of a thick reinforced concrete slab supported by structural steel framing. A north-south interior wall of reinforced concrete extends the full height of the structure providing physical separation of the diesel generator systems. The exterior and interior walls extend 6 feet below grade to form a continuous wall footing supported on select fill. Compaction of subgrade is 100 percent of maximum dry density as determined by the American Association of State Highway Officials T-180-57 Method of Compaction (Reference 54).

The standby diesel generators are located at grade and are supported on a 3 foot thick reinforced concrete mat which is physically independent of the ground floor slab and building structure.

A subsurface investigation of the soil in the area of the Standby Diesel Generator Building was performed by American Engineering Testing, Inc. (AET). This report established the adequacy of the soil to support the building pressure.

12.2.2.4.2 Seismic Analysis and Design

The structure was not subjected to a dynamic analysis. However, in compliance with the requirements for Class I structures, a seismic analysis was made on this structure based on the following criteria:

a. The Plant Control and Cable Spreading structure was selected as a similar structure upon which a dynamic analysis had been performed.

b. Foundations of both structures are supported on select compacted fill at approximately equal elevations.

c. Plan dimensions of both structures are approximately the same.

d. The Standby Diesel Generating Building is a more rigid structure than the Plant Control and Cable Spreading structure.

e. Thus use of the seismic acceleration diagrams of the Plant Control and Cable Spreading structure was considered to be a conservative basis for the seismic analysis of the Standby Diesel Generating Building.

12.2.2.4.3 Tornado and Tornado Missile Analysis and Design

In addition to the original licensing basis tornado missile protection provided by the concrete walls of the EDG building, additional tornado missile barriers were conservatively added to the Standby Diesel Generator Building and in the yard north of the Standby Diesel Generator Building which further protect the operation of critical components of the standby diesel generators and their critical subsystems. The following missile barriers were installed:

a. Diesel generator air intake and exhaust piping above roof, V-SF-9 and V-SF-10 intake louvers, building ventilation exhaust louvers, T-45A/B day tank vent lines outside the building and 11/12 base tank vent lines outside the building are protected from the tornado missiles described in Section 12.2.1.8.

b. The Diesel Fuel Oil Storage Tank, its supply and return lines, and additional lines and conduit running between the Standby Diesel Generator Building and the Diesel Fuel Oil Transfer House and between the Diesel Fuel Oil Transfer House and the Fuel Oil Storage Tank are protected from tornado missiles as described in Section 12.2.1.8.

c. The vent line of the Diesel Fuel Oil Tank is designed to breakaway due to a direct missile impact or tornado wind loads prior to plastic deformation of the vent line, which could restrict vent flow thereby affecting the design and operability of the Diesel Fuel Oil Tank. The vent line is also rerouted to preclude the possibility of a direct vertical impact of a tornado-generated missile causing the vent to puncture, buckle otherwise damage the tank and cause a loss of Fuel Oil inventory.

12.2.2.5 Turbine Building

The turbine building is basically a Class II structure; however, portions that support and protect electrical controls and instrumentation for Class I equipment were designed in accordance with criteria for design of portions of Class II structures enclosing and/or supporting Class I equipment as specified in Section 12.2.1.9, “Seismic Loads”. The primary function of the turbine building is to provide the necessary environment required for safe operation and maintenance of the turbine-generator and other components of the power conversion system.

12.2.2.5.1 Structural Description

The building is a combination of reinforced concrete and structural steel construction.

The foundation is a reinforced concrete mat of variable thickness supported on undisturbed soil. The top of the mat is approximately 20 feet below grade at the 911 foot elevation. The foundation supports the reinforced concrete turbine-generator pedestal as well as the building superstructure.

The reinforced concrete portion of the superstructure extends from the top of the mat foundation to the 951 foot elevation, approximately 20 feet above grade.

Reinforced concrete floor slabs supported by structural steel beam and girded framing are located at the 931 foot and 951 foot elevations. Interior reinforced concrete walls extending from the top of the mat up to the operating floor are oriented so as to protect personnel against radiation emanating from the turbine and auxiliary systems.

A structural steel framed super-structure is based at the 951 foot elevation on reinforced concrete columns located within the exterior walls. The superstructure encloses the operating floor and also provides support and closure for a 150 ton traveling bridge crane. The Monticello traveling bridge crane has a name plate rating of 125 ton but provisions have been identified for obtaining special lift permission from the crane manufacturer (Whiting Corp.) to lift the 281,847 lb. generator rotor.

A fully adhered EPDM membrane and insulated roof is supported by a metal roof deck which also acts as a diaphragm to transmit lateral forces to vertically braced end walls or shear frames.

12.2.2.6 Off-gas Stack

The function of the off-gas stack is to provide for controlled release and dispersal of gaseous radioactive wastes.

12.2.2.6.1 Structural Description

The stack is a free-standing tapered, reinforced concrete structure which encloses and supports an independent gas flue. The overall height of the stack above adjacent grade is 328-feet. The internal diameter of the concrete shell is 7-feet at the top and 32-feet at the 946-foot 6-inch elevation with thickness varying from 7 inches at the top to 10 inches at the 946-foot 6-inch elevation. Below 946-foot 6-inch elevation to the top of foundation at the 932-foot 6-inch elevation the stack shell is a polygon having a maximum inscribed diameter of 34-feet. The wall thickness varies in accordance with radiation shielding requirements.

The stack shell is supported on a 4-foot thick octagonal spread footing with a 1-foot 6-inch pedestal.

The independent gas flue is 18-inches in diameter reducing to 14-inches in diameter at the top.

The location of the stack is shown on Section 15 Drawing ND-95209, and the configuration described above is shown in Figure 1 of “Earthquake Analysis Off-gas Stack” included Appendix A.

12.2.2.6.2 Design

The design and construction of the stack is in accordance with the applicable requirements of the "Proposed Revision of ACI 505-54: Specification for the Design and Construction of Reinforced Concrete Chimneys" (Reference 56) as reported by ACI committee 307 except as follows:

a. Temperature

The stack shell is designed for a minimum ambient temperature of - 35°F and a maximum ambient temperature of 100°F.

b. Wind Forces

The stack is designed in accordance with ASCE Transactions Paper No. 3269 (Reference 32) for a basic wind velocity of 100 mph with a gust factor of 1.1. Wind velocities vary with height according to Table 1. (a) of the referenced paper. Wind pressures calculated for the various height zones above ground and applied to the projected elevation area are as follows:

Height Zone	Design Wind Pressure
0 - 50 foot	19 psf
50 - 150 foot	28 psf
150 foot to top	36 psf

Since the stack is positioned a distance greater than one stack height away from all Class I systems and structures it has not been designed to resist tornado wind forces.

c. Seismic Forces

As indicated in Appendix A, the stack height was analyzed for its response to the recommended earthquake design criteria which is also included in Appendix A. The results of this analysis are included in Appendix A, "Earthquake Analysis: Off-Gas Stack." As recommended by the seismic consultant the stack structure was designed to resist the calculated shear and moments in accordance with the referenced ACI code.

12.2.2.7 Circulating Water System Structures

The system functions and description are contained in Section 11.5 and Section 11.6. The principal structures in the system are the intake structure, discharge structure, access tunnel and cooling towers.

12.2.2.7.1 Intake Structure

The intake structure is basically a chambered box of reinforced concrete construction. Essentially, it consists of four 13-foot 8-inch bays with an invert elevation of 888 feet at the intake end which converges to a two section suction chamber at the discharge end. A circulating water pump is mounted over each suction chamber.

The roof of the structure is approximately 4-foot 3-inches above grade at the 934-foot 3-inch elevation and consists of reinforced concrete beam and slab framing. An operating floor is located at the 919-foot elevation on which the Emergency Service Water (ESW) and RHR Service Water (RSW) pumps are mounted.

Exterior and interior walls and slabs are constructed of reinforced concrete and provide support for the operating floor and roof framing. The structure is supported on a mat foundation 3-foot 6-inches in thickness that was placed on a lean concrete fill which overlays a layer of cemented sandstone.

The intake structure is a Class II structure and as such was designed for seismic loadings in accordance with Section 12.2.1.9. In addition the structure was analyzed for the criteria specified for Class II structures housing Class I equipment.

Those portions of the structure which house Class I equipment were designed to resist the effects of tornado as specified in Section 12.2.1.8.

12.2.2.7.2 Access Tunnel

The underground tunnel between the Turbine Building and Intake Structure provides sheltered access from the Turbine Building to operating floor or auxiliary systems pump room in the Intake Structure. Portions of the Emergency Service Water and RHR Service Water System piping and controls are located herein.

The tunnel is a box section in shape and of reinforced concrete construction. The top of the concrete tunnel at the intake structure is 4-feet above grade at the 934-foot elevation. The tunnel roof then drops below grade to the 928-foot 9-inch elevation and maintains that elevation to the Turbine Building interface.

The tunnel is constructed on an earth foundation of select backfill compacted to 100 percent of maximum dry density as determined by the American Association of State Highway Officials T-180-57 Method of Compaction (Reference 54).

Structurally the tunnel is physically separated at the Turbine Building and Intake Structure interfaces to minimize the effects of unequal settlements and dissimilar seismic response. A flexible water stop is provided in the expansion joint at each structure interface and in all construction joints. The tunnel is also completely enveloped with a waterproof membrane.

The tunnel is basically a Class II structure and was designed to comply with earthquake criteria as specified in Section 12.2.1.9 for Class II structures. Since portions of Class I category systems are located herein the structure was also investigated for compliance with criteria as specified in Section 12.2.1.9 for Class II structures housing Class I equipment. This later case was the governing condition for design.

The structure was also designed to resist the effects of a tornado in accordance with criteria set forth in Section 12.2.1.8.

12.2.2.7.3 Discharge Structure

The discharge structure is constructed of reinforced concrete of approximate dimensions 50 foot by 54 foot and 38 foot high. The roof of the structure is at the 923 foot elevation which is approximately 5 feet above grade. A lower floor at the 898 foot elevation supports two cooling tower pumps of 145,000 gpm capacity. The structure is supported on a 3 foot thick mat.

The discharge structure is a Class II structure and does not contain any Class I equipment. As such it has been analyzed for wind and seismic loadings in accordance with the requirements for Class II structures only, as specified in Section 12.2.1.6 and 12.2.1.9.

An overflow weir structure was added in the circulating water discharge canal. This structure permits the normal outflow of cooling water and yet inhibits fish from entering the canal where they could be subjected to cold shock during plant shutdown in the winter. The structure consists of an earth fill dike and a vertical sheet-pile overflow section. It is built at the end of the discharge canal and re-establishes the previously existing shoreline of the river.

The top of the dike, which is 22 feet wide, is at the 920 foot elevation. The sides of the dike are on a 3:1 slope.

The crest level of weir structure which is 54 feet wide, is at the 910 foot elevation. The height of the overflow is approximately 2.5 feet, and the water level in the discharge canal is at the 912.5 foot elevation. When the water is at this level, the overflow section discharges 645 cfs to the river.

To prevent scouring below the discharge, a 20 foot long concrete apron is built on the downstream side of the sheetpile wall. A 50 foot long rip-rap apron is built downstream of the concrete apron. The top of the concrete apron, and rip-rap section, is at the 897 foot elevation. The concrete apron has energy dissipating blocks and an end sill.

12.2.2.7.4 Cooling Towers

Each tower consists of a wood-frame and fiberglass superstructure mounted on a reinforced concrete basin at the 918-foot 6-inch elevation. Top of the superstructure is at the 966-foot 6-inch elevation and bottom of basin is at the 913-foot elevation. The basin slab is supported on an earth foundation of undisturbed soil.

The structure is a Class II structure and was designed in accordance with the Uniform Building Code.

12.2.2.8 Off-gas Storage Building

The off-gas storage building except for the Fan and Foyer Room portions, was designed for Class I seismic conditions, 939.2 feet above msl flood conditions, and for tornado wind loads and missiles. The only portion of the off-gas storage system which currently has seismic design requirements are the storage tanks and the attached piping up to the first isolation valve. The building meets all Federal, State and local codes applicable to industrial process buildings. The building is constructed of reinforced concrete on a suitable foundation and is situated near the base of the off-gas stack.

The Fan and Foyer Room portions of the off-gas storage building provide adequate Class I level protection for all external events in which the enclosed equipment is required to perform a safety related function. This includes Class I dead, live (snow and floor) and wind loads. It does not include seismic or tornado loads or tornado generated missiles.

12.2.2.9 Recombiner Building

The reinforced concrete portion of the recombiner building is designed for flooding to a maximum of 939.2 feet above msl, plus tornado and missile conditions. Although the recombiner building was designed and built for Class I seismic conditions, the design criteria for this building has been downgraded to Class II in accordance with Regulatory Guide 1.143. The building meets all Federal, State, and local building codes applicable to industrial process buildings. The building is constructed of reinforced concrete on a suitable foundation and is situated alongside the Turbine Building between the condensate storage tanks and the main power transformers.

12.2.2.10 Main Steam Line Restraints

The main steam line anchor frame and restraints are designed to accommodate the effects of the environmental conditions associated with normal operation, maintenance, testing and postulated accidents including earthquakes and pipe failures. The restraints adequately protect other structures, systems and components important to safety against dynamic loads include pipe whip and jet impingement.

The main steam line anchor and restraints are located outside of primary containment in the main steam pipe chase (see Section 15 Drawing NF-36055, along column line 6 and between column lines L and M). The anchor is positioned at the main steam line containment penetrations and the restraints are located downstream of the outer isolation valve. The main steam line anchor frame and restraints are shown in Figure 12.2-11. The anchor and restraints are attached directly to the reactor building structure.

The main steam pipes are free to move axially through the restraints. The design and positioning of the restraints limit excessive motion (i.e., “pipe whipping”) of the main steam lines to assure that a postulated pipe rupture downstream of the outboard main steam isolation valve will not result in damage to structures, systems or components within the main steam pipe chase. The loads due to postulated pipe rupture are applied in any direction transverse to the pipe axis.

The allowable stresses for the design of the restraints for pipe whip reaction loads are as follows:

a. Steel — 0.9 F_y (yield strength of steel)

b. Concrete — 0.85 F_c (compressive strength of concrete)

c. Reinforcement — 0.9 F_y (yield strength of reinforcement)

The main steam line anchor was designed to withstand the reactions from jet impingement forces caused by a postulated pipe rupture (circumferential or slot type) of the process line. Reactions at the anchor were determined from an analysis of the load transmitting capability of the process line based on the ultimate strength and strain hardening properties of the pipe materials.

Evaluation of high energy line breaks outside containment is discussed in Appendix I of the USAR.

In addition, the main steam line anchor has been designed to withstand the following loadings:

a. Thermal Expansion - A summation of piping reactions from both inside and outside of the drywell.

b. Dead Weight - Weight of piping, insulation, and fluid from both inside and outside of the drywell.

c. Seismic Reactions - Inertial forces resulting from the effects of earthquake on piping located both inside and outside of the drywell.

d. Turbine Stop Valve Closure Loads - Impulsive dynamic loads resulting from a reflected pressure wave traveling at the speed of sound caused by sudden closure of turbine stop valves. Reactions from both inside and outside of the drywell are summed.

For design of the anchors, loadings, a, b, c, and d were assumed to act simultaneously. Loadings c and d were combined by the square-root-sum-of-the-squares technique. The resulting stresses in the anchors were found to be within the allowables defined in the Eighth Edition of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (Reference 58).

12.2.2.11 Standby Gas Treatment System Off-Gas Line

The function of this line is to conduct the standby gas treatment system effluent to the off-gas stack for dispersal to the atmosphere. This line is buried and was considered to be uniformly supported. The analysis assumed the line would experience motion and acceleration similar to those experienced by the ground. The stresses resulting from the design basis earthquake were calculated by standard methods to be approximately 7,300 psi.

The line is buried in compacted fill. The movement of the buried pipe is essentially the same as that of the surrounding soil. (See "Nuclear Reactors and Earthquake," Atomic Energy Commission TID7024 (Reference 59)). Therefore, the maximum strain in the soil caused by seismic disturbances would be the same as the maximum strain in the buried pipe. The stress in the pipe can then be calculated from the strain. The assumed ground motion was that of the Taft, California, earthquake scaled to a maximum ground acceleration of 0.06 g. This was the same as the ground motion which was used for the analysis of the plant buildings. Using conservative values for the properties of the soil, the maximum strain was 2.5×10^{-4} inches per inch and the resulting stress in the pipe was approximately 7,300 psi.

12.2.2.12 Piping Modifications

12.2.2.12.1 Main Steam Line Manifold

A full size main steam equalizer was added in order to allow for stop valve testing without reducing plant power. This modification included the addition of an 18-inch main steam line connecting together the existing four main steam lines. The main steam lines were analyzed for seismic and turbine stop valve closure loads. Level D Service Limits were used for the $D + E^1$ load combination as defined in ASME Section III, Article NC-3600 (Reference 60), 1977 Edition with Summer 1977 Addenda.

12.2.2.12.2 Torus Attached Piping and SRV Discharge Lines

These lines were reanalyzed as part of the Mark I Containment Program. See the Monticello Mark I Containment Long Term Plant Unique Analysis Report, References72and 75 of Section 5, for details.

12.2.2.12.3 Scram Discharge Volume

The Scram Discharge Volume was modified in accordance with Section 12.2.1.10, Class I Piping Seismic Analysis. Hydrodynamic piping loads, water/steam hammer, were considered. Stresses for the $D + E^1$ load combination were maintained less than 2.4 S_h when using the design and evaluation requirements of ASME Section III, Article NC-3600 (Reference 60), 1980 Edition with Winter 1981 Addenda.

12.2.2.12.4 CRD Insert and Withdraw Lines

The CRD insert and withdraw lines have been analyzed in accordance with Section 12.2.1.10, Class I Piping Seismic Analysis. Water hammer loads resulting from the “failed buffer” case during plant scram have been included.

12.2.2.12.5 EFT Emergency Service Water System

The EFT Emergency Service Water System was designed in accordance with Section 12.2.1.10, Class I Piping Seismic Analysis. Stresses for the $D + E^1$ load combination were maintained less than 2.4 S_h when using the design and evaluation requirements of ASME Section III, Article NC-3600 (Reference 60), 1980 Edition with Summer 1980 Addenda.

12.2.2.12.6 Recirc System

The recirculation system and associated piping was designed in accordance with Section 12.2.1.10, Class I Piping Seismic Analysis. While the system is designed in accordance with ANSI B31.1 (Reference 34), design was verified in accordance with the rules of ASME Section III, Article NB-3600 (Reference 35). Design conditions for level A, B, C, and D service limits were considered in the analysis.

12.2.2.12.7 Loading Combinations for Class I Equipment Steel Support Structures Which are Subject to Piping Reaction Loads

The following load combinations were used for the re-evaluation of the Residual Heat Removal Heat Exchanger frames and are acceptable for the design of Class I equipment steel support structures which are subject to piping reaction loads:

a. Service Load Conditions

$$S = D + L$$
$$S = D + L + E$$
$$1.5S = D + L + Ro + Te$$
$$1.5S = D + L + Ro + Tc$$
$$1.5S = D + L + E + Ro + Te$$
$$1.5S = D + L + E + Ro + Tc$$

b. Factored Load Conditions

$$1.6S = D + L + E^1 + Ro + Te$$
$$1.6S = D + L + E^1 + Ro + Tc$$
$$1.5S = D + L + E^1$$

Where:

S	=	Required section strength based on elastic design methods and allowable stresses defined in Part I of AISC Code (Reference 58).
D	=	Dead loads and their related moments and forces, including any permanent equipment loads and hydrostatic pressures. For equipment supports, it includes static and dynamic head and fluid flow effects.
L	=	Live loads and their related moments and forces, including any movable equipment loads and other loads which vary with intensity and occurrence. For equipment supports, it includes loads due to vibration and any support movement effects.
E	=	Loads generated by the operating basis earthquake. This will include both pipe and equipment loads.
E^1	=	Loads generated by the safe shutdown earthquake. This will include both pipe and equipment loads.
Ro	=	Pipe reaction loads during normal operating or shutdown condition
Te	=	Thermal effects due to thermal expansion during normal operating or shutdown condition.
Tc	=	Thermal effects due to thermal contraction during normal operating or shutdown conditions.

12.2.2.12.8 Piping Flow Induced Vibration

The Flow Induced Vibration (FIV) effect on the main steam (MS), feedwater (FW), and reactor recirculation (REC) piping inside containment at Monticello was confirmed to be consistent with the generic description provided in the constant pressure power uprate licensing topical report (CLTR) (Reference 101) because the nominal reactor dome pressure remains the same, the REC maximum drive flow does not increase more than 5 percent, and FIV testing of the MS and FW piping systems was performed during power ascension to 2004 MWt (Reference 103).

FIV has the potential to cause main steam safety relief valve (SRV) leakage. Excessive SRV pilot or 2nd stage leakage on the Target Rock 3-Stage SRV design may result in an inadvertent SRV opening and a stuck open SRV event. However, Monticello has procedures to address a leaking SRV. The potential adverse SRV response to excessive leakage have been considered in the development of plant procedures to help ensure leaks are not allowed to become excessive. Nonetheless, the consequences of a stuck open SRV have been included in the plant-specific safety analyses and have been demonstrated to be non-limiting (Reference 103).

The effects of FIV induced stresses at 2004 MWt operating conditions on thermowells in the MS and FW system and the sample probe in the FW system were evaluated. The MS thermowell was removed to insure resonance due to vortex shedding does not occur. The ratio of vortex shedding frequency to the natural frequency for the FW thermowell and FW sample probe remain acceptable under 2004 MWt operating conditions (References 102, 103, and 106).

12.2.2.13 Personnel Hatch

The drywell personnel lock was analyzed for the stresses in the hatch shell and at the juncture of the hatch to drywell reinforcement insert plate and at the juncture of the nozzle insert plate to vessel shell.

The loading combinations for design of the personnel lock considered live loads, dead loads and seismic loads.

The maximum combined bending stress in the personnel lock shell plate is 543 psi for the Operating Basis Earthquake and 1,300 psi for the Design Basis Earthquake with allowable stress of 17,500 psi.

The maximum stress intensity in the nozzle to drywell shell plate is 2,662 psi for the operating basis earthquake and 6,400 psi for the design basis earthquake with allowable stress of 17,500 psi.

12.2.2.14 Emergency Filtration Train (EFT) Building

The function of this building is to provide a safe enclosure and protection for the main components of the main control room (MCR) air conditioning system (including the emergency filtration train units for the MCR air conditioning system) and for other safety-related equipment as necessary.

12.2.2.14.1 Structure Description

The EFT building is an L-shaped reinforced concrete structure. The east section, 46 foot long and 28-foot 8-inch wide, is supported by a mat foundation. The west section, 30-foot 4-inch long and 20-foot 5-inch wide is supported by two reinforced concrete caissons. The east section is three stories high, and the west section is two stories high.

The EFT building was analyzed for an Operating Basis Earthquake (OBE) having a design ground acceleration of 6% g horizontal and 4% g vertical, and a Safe Shutdown Earthquake (SSE) having a design ground acceleration of 12% g horizontal and 8% g vertical. Due to the complexity of the shape of the building, a finite element model was chosen and developed rather than a lumped mass stick model. Plate, beam and spring elements were used to represent the structure. Soil structure interaction was taken to account by coupling the structural model with the supporting soil. 4% and 7% model damping were used respectively for the OBE and SSE condition.

12.2.2.15 <u>Turbine Building Addition</u>

The Turbine Building Addition is a class II structure and does not contain any class I equipment. The structure was designed in accordance with the Uniform Building Code. The primary function of the Turbine Building Addition is to provide a controlled environment for the condenser retubing effort.

12.2.2.15.1 <u>Structural Description</u>

The Turbine Building Addition is approximately 96 feet long x 63 feet wide x 40 feet high. The enclosure walls are insulated aluminum siding with steel liner panels over exposed interior frame built on a reinforced concrete slab. A minimum joint gap of 2 inches is provided between the Turbine Building Addition and existing Class I buildings to eliminate structural interaction during a seismic event.

12.2.2.16 <u>Underground Duct Bank - 3rd Floor EFT To Reactor Building</u>

The underground duct bank is classified as nuclear safety related and is a Class 1 structure. The primary function of the duct bank is to carry Division II safe shutdown cables outside of areas where fire damage could occur.

12.2.2.16.1 <u>Structural Description</u>

The underground duct bank is approximately 700 feet in length. Manholes are approximately 10 feet wide by 10 feet long by 8 feet deep. Seismic joints have been provided at the man-hole to duct bank interface and riser to duct bank interface.

12.2.2.17 <u>Reactor Steam Dryer Analyses</u>

The steam dryer does not perform a safety function but must retain its structural integrity to avoid the generation of loose parts that might adversely impact the capability of other plant equipment to perform their safety functions. Therefore, a BWR steam dryer is a safety significant component located inside the reactor pressure vessel.

Two analyses were performed on the steam dryer to assess it for Extended Power Uprate operating conditions up to 2004 MWt. The first analysis, an ASME structural verification (Reference 107), evaluated the steam dryer primary and secondary stresses as well as cyclic operation due to thermal and mechanical load cycling in accordance with the requirements of Section III, Subsection NG of the 2004 ASME Code. The primary and secondary stresses include those developed as a result of deadweight, differential pressure, thermal hydraulic loads, flow induced vibration (FIV), seismic, etc., due to applicable normal, transient, and accident conditions. Main steam line (MSL) acoustic pressure loads are a FIV load and were included as an input for this analysis. The analysis was performed using finite element modeling. The conclusions of the analysis stated that the primary and secondary stress combinations determined by the analysis were below code allowables and that cyclic operation stresses were below those levels which require further low cycle fatigue analysis.

The second analysis, a high cycle acoustic load evaluation (Reference 108), assessed the loads and stresses on the steam dryer due to MSL acoustic pressure and recirculation pump vane passing frequency (VPF) loads in accordance with the guidelines of Section III, Subsection NG of the 2004 ASME Code. This analysis was also an FIV input for Reference 107. The analysis was performed using a harmonic load methodology that combines the use of a finite element representation of the dryer with special-purpose computer codes. The analysis was divided into two parts. The evaluation of steam dryer components located above the support ring was performed by applying acoustic loads developed using a model benchmarked against Monticello steam dryer stress values while those steam dryer components located below the support ring were evaluated by applying acoustic loads developed using a model benchmarked against Quad Cities steam dryer stress values (References 108, 109, and 110). The objective of the analysis was to show that the maximum alternating stress intensity anywhere in the dryer due to a combination of the MSL acoustic loads and VPF loads was less than the dryer material endurance strength limit of 13.6 ksi at 1011 cycles. Consequently, any stress ratio (material endurance strength divided by calculated stress) for a steam dryer component that is greater than 1.0 satisfies ASME Code allowables for fatigue usage. The results of the analysis show that the minimum stress intensity ratio calculated for all steam dryer components at EPU conditions (2004 MWt) is greater than 1.0. Therefore, the high cycle fatigue stresses in the steam dryer were determined to be less than code allowables.

The NRC reviewed the analytical methodologies, assumptions, and computer modeling used in these analysis and found them to be acceptable (Reference 105).

See USAR sections 3.6.3.1, 3.6.3.2, and 3.6.3.3 for additional information.

12.2.3 <u>Turbine Missile Analysis</u>

In 1986, General Electric prepared an analysis entitled “Probability of Missile Generation in GE Nuclear Turbines” (Reference 63) that was approved by the NRC in NUREG-1048 (Reference 64). This analysis considers two possible turbine failures; a brittle fracture caused by stress corrosion cracking in the keyways and a ductile tensile fracture caused by an abnormal overspeed event. In the monoblock rotor design the wheels are machined from the rotor forging which eliminates the plausibility of stress corrosion cracking by doing away with keyed wheels typical of a built-up rotor, which in turn eliminates the possibility of a brittle fracture. General Electric’s missile analysis of the LP monoblock rotors states that the minimum tensile strength of the rotor forging corresponds to an overspeed of 219%-225% of rated speed (1800 rpm). Using a more practical tensile strength increases the rotor’s capability to withstand an overspeed in the 230%-235% range of rated speed. Assuming the main breaker fails open causing a runaway condition, the maximum attainable overspeed for this unit would be 215%-218% of rated speed. The L-0 buckets have a calculated failure mechanism below this speed. Using the minimum material strength for the buckets, the calculated failure would occur at a minimum speed of 170% rated speed in the vane root area. Both the rotor and bucket failure speeds are well above the redundant overspeed trip protection features of the turbine at 110% and 112% of rated speed.

The overspeed capability of the HP and LP rotors is over 216%. The limiting components, per design, for the rotors are the LP last stage buckets, which have an overspeed capability of 170%.

According to the regulatory guidelines in Regulatory Guide 1.115, Revision 1 (Reference 65), the probability of unacceptable damage from turbine missiles (P_4) should be less than or equal to 10^{-7} per year for an individual plant. Appendix U of NUREG-1048, Supplement 6 (July 1986) (Reference 66) provides a methodology for calculating the probability of unacceptable damage resulting from turbine missiles. Per NUREG-1048, $P_4 = P_1 X P_2 X P_3$, where,

P_1 = Probability of a turbine failure resulting in missiles.

P_2 = Probability of missile striking Safety Related equipment.

P_3 = Probability of struck equipment failing to perform their Safety Related function.

Per Appendix U of NUREG-1048, Supplement 6, $P_2 x P_3$ is specified as 10^{-2} for a plant with an unfavorably oriented turbine. Reference 63 calculated the annual probability of an overspeed resulting in missile generation, P_1 as $8.26x10^{-6}$ based on a 9 month testing frequency of the stop, control and intercept valves with a maintenance interval of 10 years. This changes to $5.05x10^{-6}$ with a testing frequency of 6 months and a maintenance interval of 12 years. The probability of unacceptable damage from turbine missiles (P_4) for Monticello remains $\leq 10^{-7}$ with these values. Either value offers a safe margin within the acceptance criteria established in Regulatory Guide 1.115, Revision 1. 60400000441

12.2.4 <u>High Energy Line Failures Outside Containment</u>

A detailed analysis has been conducted to substantiate that the design of Monticello Nuclear Generating Plant is adequate to withstand the effects of a postulated rupture of a high energy fluid piping system outside the primary containment. Detailed analysis and results of the study are presented in Appendix I.

12.2.5 <u>Control of Heavy Loads</u>

12.2.5.1 <u>Introduction</u>

In an effort to upgrade measures used to control heavy loads, the NRC Staff, on December 22, 1980, issued a letter (Reference 67) to all operating licensees containing NUREG-0612 (Reference 68). Responses were requested in two phases:

Phase I (Six Month Response): Identify the load handling equipment within the scope of NUREG-0612 and to describe the associated load paths, procedures, operator training, special and general purpose lifting devices, the maintenance, testing and repair of equipment and the handling equipment specifications.

Phase II (Nine Month Response): Show that either single-failure-proof handling equipment is not needed or that single-failure-proof equipment has been provided.

NSP responded to both Phase I and Phase II (References 11 thru 17). The NRC issued a Safety Evaluation Report (SER) on Phase I (Reference 18). The NRC concluded that a detailed review of Phase II was not necessary but encouraged the implementation of actions proposed as a result of Phase II although this implementation was not a requirement (Reference 19).

At Monticello, a heavy load has been set as any load greater than 1100 lb. over the spent fuel pool and the reactor core, and 1500 lb. in other areas of the plant. The heavy load limit of 1100 lb. is less than the combined weight of the spent fuel assembly and grapple used in the refueling accident analysis as described in Section 14.7.6.3.1. The heavy load limit of 1500 lb. was established based on the combined weight of the spent fuel assembly and grapple used at the time NUREG-0612 became effective.

12.2.5.2 Applicable Overhead Handling Systems

The applicable overhead handling systems are those that handle heavy loads above safety-related components (components required for plant shutdown or decay heat removal) spent fuel or the reactor vessel. Many plant load handling devices were excluded for one of the following reasons:

a. Physical separation between load impact points and systems or components required for plant shutdown or decay heat removal.

b. Handling device capacity.

c. A sole-purpose lift function and the fact that a load drop would damage only the equipment lifted.

The Monticello administrative procedures list which overhead handling devices are excluded from needing heavy load movement procedures.

For the applicable overhead load handling systems NUREG-0612 requires that 1) safe load paths be developed, 2) procedures be developed for load handling, 3) operators be trained in their operation, 4) special lifting devices reviewed, 5) slings comply with the requirements of NUREG-0612 and 6) systems be designed, inspected, tested and maintained in accordance with the requirements of NUREG-0612.

12.2.5.3 Safe Load Paths

Safe load paths are discussed in NUREG-0612, Section 5.1.1(1).

> "Safe load paths should be defined for the movement of heavy loads to minimize the potential for heavy loads, if dropped, to impact irradiated fuel in the reactor vessel and in the spent fuel pool, or to impact safe shutdown equipment. The path should follow, to the extent practical, structural floor members, beams, etc., such that if the load is dropped, the structure is more likely to withstand the impact. These load paths should be defined in procedures, shown on equipment layout drawings, and clearly marked on the floor in the area where the load is to be handled. Deviations from defined load paths should require written alternative procedures approved by the plant safety review committee."

Safe load paths are procedurally defined for the reactor building crane and the turbine building crane. All other handling systems have fixed load paths as determined by the path of the associated monorail or have no load paths because the lifting point is fixed.

Safe load paths in the reactor building are not marked because protective coverings used to minimize the spread of contamination would obscure the marked load paths. Further, the physical dimensions and the space available for lay down of major heavy loads do not allow major deviations from the designated load paths. Station procedures ensure that the safe load path is followed. In the turbine building, an exclusion area over safe shutdown equipment on lower levels has been identified and marked with painted lines. Movement of heavy loads in this area is administratively controlled.

12.2.5.4 Load Handling Procedures

Load Handling Procedures for the applicable overhead handling systems should include, per NUREG-0612 Section 5.1.1(2), identification of required equipment, inspections and acceptance criteria required before movement of the load, the steps and proper sequence to be followed in handling the load, defining the safe path and other precautions.

These precautions include lift height and load weight restrictions that are based on load drop analysis that are referenced in the associated load handling procedure. Load drop analyses have resulted in load weight and lift height restrictions for the turbine building crane over the exclusion area on TB elevation 951' and for the auxiliary hoist for the reactor building crane.

Wire rope break analysis and heavy load program commitments have resulted in a minimum cask lift height of 6.5" and a maximum cask lift height of 8.5" for the reactor building crane during horizontal movement over the refuel floor/curbing (Reference 98, 99, 100).

12.2.5.5 Crane Operator Training

Crane operators for the applicable overhead handling systems are trained, qualified, and perform in compliance with ANSI B30.2-1976 (Reference 69).

12.2.5.6 Special Lifting Devices

Three special lifting devices for the applicable overhead handling systems are subject to the requirements of NUREG-0612. These are the dryer/steam separator lifting device, the reactor vessel/drywell head lifting device and cattle chute lifting strongback. These devices comply with applicable requirements for design stated in paragraph 3.2 of ANSI N14.6-1978 (Reference 70) [also see NUREG-0612, Section 5.1.1(5)].

The vendor-supplied spent fuel cask lifting device must also comply with the applicable requirements of ANSI N14.6-1978 or equivalent.

12.2.5.7 Slings

Slings used by the applicable overhead handling devices shall comply with ANSI B30.9-1971, “Slings” (Reference 71). Slings must have a minimum factor of safety of 5. Slings are not derated for dynamic loading since the dynamic loads are a small percentage of the overall static load and can be disregarded (References 12, 18 and Section 5.1.1(5) of NUREG-0612).

12.2.5.8 Inspection, Testing And Maintenance

Procedures used for inspection, testing, and maintenance of the applicable overhead handling devices shall comply with ANSI B30.2-1976 (Reference 12, 18 and Section 5.1.1(6) of NUREG-0612).

12.2.5.9 Crane Design

NUREG-0612, Section 5.1.1(7) suggests that cranes should be designed to meet the applicable criteria and guidelines of Chapter 2-1 of ANSI B30.2-1976, “Overhead and Gantry Cranes”, and of CMAA-70, “Specifications for Electric Overhead Traveling Cranes” (Reference 72).

The reactor building crane and turbine building crane were manufactured prior to issuance of CMAA-70 and ANSI B30.2. These cranes were designed to EOCI 61. The NRC concluded that the guidelines on NUREG-0612, Section 5.1.1 and 5.3 have been satisfied (Reference 97). The reactor building crane has been found to meet the criteria for a single-failure-proof crane (i.e., meeting the applicable provisions of draft Regulatory Guide 1.104) (Reference 73, 97).

The Reactor Building crane system is designed to retain structural integrity during the Safe Shutdown Earthquake (SSE). The crane system is designed to withstand a maximum horizontal acceleration of 0.8g with a coincident maximum vertical acceleration of 0.08g applied at the bridge rails. The crane bridge and trolley equipment are adequately restrained to prevent derailment during a seismic event. The design codes and loading conditions applicable at the time of the original installation did not include the lifted load in the seismic analysis because of the extremely low probability of both events occurring simultaneously (References 93, 94 and 95).

The Turbine Building Crane is not in strict compliance with CMAA-70 requirements with respect to the gear durability rating. Periodic inspections consistent with normal maintenance routines will ensure the serviceability of the gearing.

12.2.5.10 Interfacing Lift Points

Interfacing lift points such as lifting lugs used by applicable single failure proof overhead handling devices shall comply with Section 5.1.6(3) of NUREG-0612. Interfacing lift points must have the applicable minimum factor of safety with respect to the ultimate strength for maximum concurrent static and dynamic loads. Dynamic load factor shall be in accordance with CMAA-70 (Reference 72).

12.2.5.11 Safety Evaluation

Controls implemented by NUREG-0612 assure safe handling of heavy loads at Monticello. Safe load paths are procedurally defined for the reactor building crane and the turbine building crane. All other handling systems have fixed load paths as determined by the path of the associated monorail or have no load paths because the lifting point is fixed. The reactor building crane has been found to meet the criteria for a single-failure-proof crane. Safe handling of heavy loads is evaluated and controlled by station procedures. Precautions including lift height and load weight restrictions are based on load drop analysis that are referenced in the associated load handling procedure. Load drop analyses have resulted in load weight and lift height restrictions for the turbine building crane over the exclusion area on TB elevation 951' and for the auxiliary hoist for the reactor building crane.

Table 12.2-1 Governing Codes and Regulations[1]

A. Uniform Building Code (UBC) 1964 Edition (Reference 29).

B. American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings - Sixth Edition. This specification is to be used with one exception, that an allowable weld stress of 21 KSI may be used for E70 electrode (Reference 74).

C. American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete - (ACI - 318-63) (Reference 31).

D. American Welding Society (AWS) Standard Code for Arc and Gas Welding in Building Construction (AWS D1.0) (Reference 75).

E. American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code Sections III, VIII, IX and XI (Reference 76).

F. American Society of Civil Engineers Paper No. 3269, for Wind Design Requirements (Reference 32).

G. Regulations of the Minnesota Department of Health with respect to Water Supply and Sewage (Reference 77).

H. AWWA Standards for water wells and steel circulating water pipe (Reference 78).

I. API Specification No. 620 for Welded Steel Storage Tanks (Reference 79).

J. Regulations of the Minnesota Department of Conservation with respect to water appropriation (Reference 80).

K. Regulations of the Minnesota Industrial Commission with respect to space requirements and industrial safety (Reference 81).

L. Proposed Revision of American Concrete Institute ACI 505-54: Specification for the Design and Construction of Reinforced Concrete Chimneys as reported by ACI committee 307 (Reference 56).

M. USAS B31.1.0-1967 plus code cases (Reference 37) or ANSI B31.1 plus code cases (Reference 34).

1. These are the codes and regulations in effect at the time of original construction. If later code editions o regulations are to be used, a reconciliation or evaluation as appropriate must be performed that justifies the use of the later code edition or regulation.

12.3 Shielding and Radiation Protection

12.3.1 Design Basis

12.3.1.1 General

Plant shielding is designed to provide operational and maintenance access to the plant with personnel exposures within the content of interpretation of the “Code of Federal Regulations,” Title 10, Chapter 1, Part 20, entitled “Standards for Protection Against Radiation” at the time the original FSAR was prepared. A post accident dose analysis and shielding study was later performed to the guidelines of NUREG-0578 (Reference 83) and NUREG-0737 (Reference 84). In 2006, the NRC approved the Alternative Source Term (AST) methodology for Monticello (Reference 90). The AST methodology forecasts personnel dose consequences for design basis accidents.

The design considers three basic operating conditions.

12.3.1.2 Normal Full Power Operation

Design criteria considers all core, coolant and waste sources in the reactor and turbine buildings.

12.3.1.3 Shutdown Operation

Design criteria considers fission products in the core, sources in the shutdown heat exchangers and radiation conditions affecting refueling operations.

12.3.1.4 Design Basis Accident

A design review of plant shielding of spaces for post-accident operations was performed per NUREG-0578 Item 2.1.6.b and NUREG-0737 Item II.B.2 (Reference 4).

Areas and equipment which were considered vital for post-accident occupancy or operation were evaluated to determine if access and performance of required operator activities or equipment functions might be unduly impaired due to the presence of a postulated conservative radiation source in the selected systems. A review by the NRC Staff concluded (Reference 7) that the Monticello plant shielding meets the guidance of TMI Action Plan Item II.B.2 of NUREG-0737.

Since the TMI dose study above was completed, Monticello has implemented the AST post-accident dose methodology for non-Environmental Qualification purposes. The application of the AST methodology revised the assumptions for source terms, occupancy, and pathways and provided updated projected dose results for vital areas at MNGP. The methodology demonstrated that projected post-accident doses in the Control Room and Technical Support Center (TSC) are below the 10CFR50.67 and 10CFR50 Appendix A GDC 19 limits. See USAR Section 14.7 description of the AST methodology.

At Monticello, the vital areas that require post-accident dose analyses presently include the Control Room and the TSC. Per USAR Section 10.3.10, the location of the Post Accident Sampling System is no longer considered a vital area. Vital areas may include any area outside the Control Room or TSC where an operator is required to perform a specific manual action that is credited in the safety analyses to aid in the mitigation of a design basis accident. Areas identified as vital areas in the future will be evaluated using the AST methodology.

Access to non-vital areas during radiological events is controlled under the Monticello radiation and health physics program and the site Emergency Plan. Post-accident radiological habitability for these site areas is determined by personnel dosimetry and by real-time radiation and contamination surveys.

12.3.1.5 Radiation Exposure of Personnel

Radiation exposure of personnel is a function of both radiation level and occupancy time in radiation areas. The Monticello radiation protection policy and program restrict total permissible radiation dose received by personnel to the limits defined by 10CFR20, “Standards for Protection Against Radiation.” Radiation zones within the plant are to be marked in accordance with the regulations in 10CFR20. Radiation control procedures are maintained and made available to all station personnel. These procedures, which implement the Monticello radiation protection policy and program, are consistent with the requirements of 10CFR20. In some instances, temporary shielding is installed to reduce personnel exposure during short term maintenance, modification, or operations activities. Within certain restrictions, temporary shielding can be safely installed on some portions of reactor piping during plant operation.

12.3.1.6 Specific Design Conditions

The offgas system shielding is based on a stack release rate of 260,000 μCi/sec. Reactor water design fission product concentrations and activated corrosion products were assumed to be the maximum values expected; 8.0 μCi/cc, and 0.07 μCi/cc, respectively. These conditions yield maximum shielding conditions in the demineralizer and clean-up systems.

The design basis accident defining the protection required for the plant main control room is the loss-of-coolant accident. The radiological consequences of this accident are described in USAR Section 14.7.2.

During high radiation conditions HEPA/Charcoal Filtered outside air will be supplied to the Main Control Room and EFT Building first and second floors, excluding the battery room, to pressurize these areas and prevent the infiltration of contaminated air. For the design basis accidents other than the loss-of-coolant accident, analysis using Alternative Source Term methodology has demonstrated that operation of an Emergency Filtration Train is not required to maintain Control Room dose below GDC 19 and 10CFR50.67 limits.

The Technical Support Center - Emergency Ventilation System (TSC - EVS) protects personnel in the TSC from bypass leakage by providing positively pressurized HEPA/charcoal filtered air.

12.3.1.7 Radiation Exposure of Materials and Components

Environmental qualification of safety related equipment is discussed in Section 8.9.

12.3.2 Description

12.3.2.1 Radiation Zoning and Access Control During Normal Operation

Plant personnel are protected from radiation exposure by shielding, monitoring, and procedures. The plant is divided into two major areas: the Radiologically Controlled Area and the Clean Area.

a. Clean Area

The Clean Area can be occupied by plant personnel or visitors on an unlimited time basis with a minimum probability of health hazard from radiation exposure.

b. Radiologically Controlled Area (RCA)

The RCA includes all plant areas in which significant radiation and radioactive material are present. RCA boundaries are identified with signs. Signs and barricades are used within the RCA to identify areas with more significant radiation levels.

RCA access is controlled administratively. Normal entry and exit is through the designated access control point. In case of emergency, personnel are able to use alternate portals as escape routes.

12.3.2.2 Plant Shielding During Normal Operation

12.3.2.2.1 Reactor Building

The reactor building contains four major shielding structures; the reactor sacrificial shield, the drywell biological shield, the main steam pipe chase and the spent fuel pool.

The drywell biological shield concrete together with the reactor sacrificial shield provides the main protection for the areas surrounding the reactor vessel, the primary coolant and recirculation systems. More than 8 feet of concrete thickness is used to help keep the radiation dose rates in the fully accessible areas as low as reasonably achievable. This shielding is designed to prevent sources inside the drywell from contributing more than 1.0 mRem/hr to Reactor Building radiation levels.

The main steam line pipe chase with its 4-foot-thick concrete walls is the connecting shield structure between the reactor and turbine buildings. The highly radioactive spent fuel assemblies are stored inside the spent fuel storage pool. A minimum cover of 8 feet of water above the fuel assemblies is maintained to protect plant personnel during fuel storage and transfer operations.

The reactor cleanup demineralizer and spent fuel pool cooling and demineralizer system and radwaste equipment are housed in numerous concrete shielded rooms surrounding the drywell concrete structure. Enclosing these secondary sources of radiation in shielded rooms permits the adjacent areas to be accessible to personnel on a 40-hour week basis.

The entrances into the drywell space are well shielded with a 5-foot thick shield plug at the equipment lock at the 935-foot elevation, and a 4-foot-thick concrete labyrinth at the personnel lock at the 935-foot elevation.

12.3.2.2.2 Turbine Building

Radioactive sources enter the turbine building with the steam from the reactor. These sources are N-16, fission product gases, and some radioisotopes 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 protection is provided around the following areas where access is limited and controlled during operation of the plant:

a. Main Steam Lines

b. Main Condenser Hotwell Area

c. Moisture Separators

d. Reactor Feedwater System Heaters

e. Primary and Extraction Steam Piping

f. Air Ejectors and Steam Packing Exhauster

g. Condensate Demineralizer

12.3.2.2.3 Radwaste Building

Shielding of the radwaste facility is based on processing the accumulation of radioactivity which occurs with the release of fission products which would cause an off-gas release rate of 0.26 Ci/sec after a 30-minute hold-up time.

All areas for preparing, handling or storing the radwaste for removal off site are shielded to meet these conditions. Access to the radwaste building is controlled.

12.3.2.2.4 Service Building

Most areas in the service building are designed for radiation levels below 0.5 mRem/h, and are uncontrolled access areas.

Dose rates in the plant main control room are calculated to be less than 0.5 mRem/h during normal full power operation.

The plant access control room is normally used by plant personnel who leave the uncontrolled zone areas to enter any of the controlled regions of the plant. Personnel leaving the controlled areas must pass through the plant access control room where they are checked for contamination levels.

12.3.2.2.5 Plant Main Stack

The shielding design for the plant main stack is based on the gaseous fission product release rate of 0.26 Ci/sec and the accompanying particulate radioactive matter in the off-gas filters. Shielding is provided for controlled access at ground level to maintain the filters and instrumentation.

12.3.2.2.6 General Plant Yard Area

Plant yard areas which are frequently occupied by plant personnel receive a radiation field of less than 0.5 mRem/h.

12.3.3 Performance Analysis

The normal construction quality control program provided assurance that there are no major defects in the shielding. After plant startup, both the structural integrity of the shielding and the calculations are checked by radiation surveys which are performed at various reactor power levels.

12.4 Radioactive Materials Safety

12.4.1 Materials Safety Program

All areas designated for the normal storage of special nuclear material (SNM) are within the Protected Area. Nuclear Engineering personnel are responsible for the control and handling of SNM in the form of reactor fuel and fission chambers. Radiation Protection personnel are responsible for the control and handling of SNM in the form of sources. 60400000451

The designated reactor fuel storage area is the fuel storage pool, which is only accessible from the reactor building refueling floor. The fuel storage pool is designed for underwater storage of irradiated or unirradiated fuel.

The reactor building is controlled as a vital area. In addition, plant personnel periodically check the fuel storage areas. Plant operating personnel are required to be present in the area when fuel handling operations are in progress. Special fuel handling procedures are written for the receipt, transfer and offsite shipment of reactor fuel.

Sufficient passageways are available on the refueling floor such that heavy crane loads need not pass over fuel in designated storage areas. Fuel storage facilities are of concrete and steel construction and the storage and use of combustible materials in these areas is strictly controlled. Fire fighting equipment is available in the area.

Additional areas of the facility can be designated as interim reactor fuel storage and transfer areas during the receipt of new fuel. These areas may include available open floor space in the reactor building as well as outside areas. Fuel is not removed from the shipping container until it is on the refueling floor in the vicinity of the new fuel inspection stand. Special procedures for fuel receipt consider the security of the fuel and implement additional precautions as appropriate. 60400000451

The on-site Independent Spent Fuel Storage Installation (ISFSI) is also a designated storage area for spent reactor fuel. The ISFSI is contained in a separate Protected Area. Material safety requirements for the ISFSI are provided in 10CFR72. 60400000451

Small amounts of SNM are required for miniature in-core fission chambers. These devices are generally confined inside the reactor vessel with the exception of newly received replacement chambers and failed or depleted chambers awaiting shipment offsite. Provisions are also included in the license limit of SNM for larger, more sensitive fission chambers which may be necessary in the event that the core must be completely unloaded.

All SNM in the form of sources is under the control of the Radiation Protection Manager. Storage, leak testing and physical inventory of SNM sources are done along with that of byproduct material as discussed below. 60400000451

Accountability procedures define the methods used for SNM accountability and for the shipping and status report preparations in accordance with 10CFR70. The accountability system is designed to meet the following criteria:

a. Location of each fuel assembly, detector and source having non-exempt quantities of SNM is known at all times.

b. It is possible to assemble a chronological history of each fuel assembly's movement onsite.

c. SNM accountability to the NRC, fuel vendors and Northern States Power Company-Minnesota is satisfied.

The Radiation Protection Manager is responsible for the control and handling of all byproduct material. This includes reactor startup sources, sealed test and calibration sources and those sources unrestricted as to chemical or physical form used as test and calibration sources. 60400000451

Sources are periodically tested to ensure that leakage of byproduct and special nuclear material does not exceed limits.

12.4.1.1 <u>Sealed Sources</u>

Leak testing requirements apply to each sealed plutonium source and to sealed byproduct material sources containing more than 100 microcuries of beta and/or gamma emitting material, or more than 10 microcuries of alpha emitting material, other than Hydrogen-3, with a half-life greater than 30 days and in any form other than gas. Testing for leakage and/or contamination is performed at intervals not to exceed 6 months. 60400000451

The periodic leak test does not apply to sealed sources that are stored and not being used. Prior to any use or transfer to another person, sources are leak tested unless they have been tested within 6 months prior to the date of the transfer. In the absence of a certificate from a transferor indicating that a test has been made within 6 months prior to transfer, a sealed source received from another person is not put into use until tested.

Tests for leakage are performed by qualified station personnel or by other persons specifically authorized by the Nuclear Regulatory Commission or agreement state. The leakage test is capable of detecting the presence of 0.005 microcuries of contamination on the test sample. The test sample is taken from the source or from appropriate accessible surfaces of the device in which the sealed source is permanently or semipermanently mounted or stored. Each sealed source with removable contamination in excess of the allowable limit is immediately withdrawn from use and either decontaminated and repaired, or disposed of in accordance with the regulations of the NRC. 60400000451

12.4.1.2 Unsealed Alpha Sources

Unsealed sources are typically electroplated or affixed to a sample holder by other secure processes. Leak testing requirements apply to unsealed alpha sources containing more than 0.1 microcurie of plutonium. When not in use, the sources are stored in a closed container adequately designed and constructed to contain plutonium which might be released during storage. Leak testing is required at intervals not to exceed 3 months.

The inventory of radioactive sources in possession is maintained current at all times. A physical inventory of sources is taken at six month intervals.

12.4.2 Facilities and Equipment

The Chemistry Laboratory and Mask Decon Area are two permanent facilities available at the Monticello Nuclear Generating Plant.

The Chemistry Laboratory fume hoods and mask decon equipment discharge air through absolute filters to the reactor building plenum, which is equipped with continuous monitoring.

Survey and measuring instruments are described in Section 7.5. The types of portable radiation instruments and the inspection and testing programs are also discussed.

The plant radiation monitoring devices are also described in Section 7.5.

12.4.3 Personnel and Procedures

The experience and qualification of the personnel responsible for the handling, monitoring and control of byproduct and special nuclear material meet, at a minimum, the requirements described in ANSI N18.1-1971 (Reference 61). 60400000451

The Radiation Protection Program consists of detailed procedures covering radiation safety. These procedures apply to all site personnel. In general, the instructions describe the following:

a. Qualification and Training

b. Area Control

c. Personnel Exposure Monitoring

d. ALARA Plan

e. Radiological Work Control

f. Radioactive Material Control

g. Respiratory Protection

h. Radioactive Effluent Management

i. Radioactive Material Shipping

j. Review and Audit

k. Records and Reports

Radiation areas are posted in accordance with the requirements of 10CFR20.

In addition to the Radiation Monitoring System described in Section 7.5, routine radiation and contamination surveys are conducted using portable and count room radiation monitoring equipment to continually monitor areas of the plant. In addition, specific surveys are made on request to evaluate and determine safe working conditions for personnel on specific jobs. Written procedures for conducting radiation and contamination surveys are provided.

All personnel, other than escorted visitors, are indoctrinated in radiological safety. Until personnel have been adequately indoctrinated in radiological safety they must be authorized by Radiation Protection personnel and be assigned an escort to enter an area where radioactive materials are stored and handled.

Personnel exposure is determined using thermoluminescent devices and self reading dosimeters. Exposure limits are established to conform with the requirements of 10CFR20. Protective clothing and equipment are available to minimize personnel exposure to contamination. Personnel decontamination procedures are provided.

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The emergency plan, defining actions to be taken in the event of a radiation occurrence, is discussed in Section 13.4.3.

12.5 References

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2. General Electric report, APED-5696, “Tornado Protection for the Spent Fuel Storage Pool”, D R Miller and W A Williams, November 1968.

3. Deleted.

4. Bechtel report, “Monticello Nuclear Generating Plant Reactor Building Design Review for Equipment and Areas Receiving Radiation Doses from Post Accident Airborne Activity (NUREG-0578, Part 2.1.6b)”, September 1980.

5. Deleted.

6. NSP (D Musolf) letter to the NRC, “Response to Request for Additional Information Related to Masonry Wall Design (IE Bulletin 80-11)”, dated June 30, 1982.

7. NRC (D B Vassallo) letter to NSP (D M Musolf), “NUREG-0737, Item II.B.2, Plant Shielding Modifications”, dated May 25, 1983.

8. Deleted.

9. NRC (D B Vassallo) letter to NSP (D M Musolf), “Masonry Wall Design (Generic Issue B-59)”, dated January 6, 1984.

10. Bechtel stress report No. SR-10040-SS 1, “ASME Section III Class 1 Analysis of Recirculation Lines for Monticello”, Revision 2, Approved January 30, 1985.

11. NSP (L O Mayer) letter to the NRC, “Control of Heavy Loads (Six Month Submittal)”, dated September 30, 1981.

12. NSP (D M Musolf) letter to the NRC, “Control of Heavy Loads (Revised Six Month Submittal)”, dated July 7, 1982.

13. NSP (D M Musolf) letter to the NRC, “Control of Heavy Loads (Revised Nine Month Submittal and Unresolved Item, Review of Special Lifting Devices 2.1.3d)”, dated October 11, 1982.

14. NSP (D M Musolf) letter to the NRC, “Control of Heavy Loads (Response to Draft TER Open Items)”, dated September 27, 1983.

15. NSP (D M Musolf) letter to the NRC, “Control of Heavy Loads (Turbine Building Crane Evaluation to Two CMAA Requirements)”, dated October 25, 1983.

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19. NRC (H L Thompson) Generic Letter 85-11, “Completion of Phase II of Control of Heavy Loads at Nuclear Power Plants NUREG-0612”, dated June 28, 1985.

20. Deleted.

21. NSP (L O Mayer) letter to the AEC (D J Skovholt), Response to Inquiry Relating to Flooding Incident at Quad Cities 1, dated September 28, 1972.

22. NSP (L O Mayer) letter to the AEC (D L Ziemann), “Supplementary Information on Interior Flooding”, dated August 24, 1973.

23. AEC Memorandum to file, “Flooding of Critical Equipment at Monticello (Northern States Power Company)”, J J Shea, dated September 27, 1973.

24. NSP (T M Parker) letter to the NRC, “Potential Loss of Fuel Oil Transfer Capability During External Flood Due to Procedure Inadequacy”, dated December 26, 1990 and April 24, 1991.

25. Deleted.

26. NSP (T M Parker) letter to the NRC, “Due to Non-Conservative Assumptions in the Internal Flooding Analysis a Break in the Service Water System May Have Resulted in a Loss of Redundant Trains of Safety Related Equipment”, dated January 22, 1992.

27. Deleted.

28. Deleted.

29. Uniform Building Code.

30. ASME Boiler and Pressure Vessel Code, Section III, Class A Vessel, 1965 Edition.

31. American Concrete Institute Building Code ACI 318-63, “Requirements for Reinforced Concrete”, 1963 Edition.

32. ASCE Transactions, Paper 3269, “Wind Forces on Structures”, 1961 Edition.

33. NRC Regulatory Guide 1.60, “Design Response Spectra for Seismic Design of Nuclear Power Plants”.

34. American National Standard Code for Pressure Piping, ANSI B31.1 - 1977, “Power Piping”, though Summer 1978 Addenda.

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38. NRC (J G Keppler) letter to NSP (L Wachter), “IE Bulletin No. 79-14”, dated July 2, 1979.

39. NRC (J G Keppler) letter to NSP (L Wachter), “IE Bulletin No. 79-02”, dated March 8, 1979.

40. NRC (J G Keppler) letter to NSP (L Wachter), “IE Bulletin No. 79-02, Revision 1, Supplement 1”, dated August 20, 1979.

41. NRC (H R Denton) Generic Letter 87-02, “Verification of Seismic Adequacy of Mechanical and Electrical Equipment in Operating Reactors, Unresolved Safety Issue (USI) A-46”, dated February 19, 1987.

42. Deleted.

43. NRC (J G Partlow) letter to all Unresolved Safety Issue (USI) A-46 Plant Licensees Who are Members of the Seismic Qualification Utility Group (SQUG), “Supplement 1 to Generic Letter 87-02 That Transmits Supplemental Safety Evaluation Report Number 2 (SSER-2) on SQUG Generic Implementation Procedure, Revision 2, as corrected on February 14, 1992 (GIP-2)”, dated May 22, 1992.

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45. NSP (T M Parker) letter to the NRC, “Response to Supplement 1 to Generic Letter 87-02 on SQUG Resolution of USI A-46”, dated September 21, 1992.

46. NRC (A H Hsia) letter to NSP (T M Parker), Evaluation of Licensees 120-day Response to Supplement No. 1 to Generic Letter 87-02 (TAC No. M69460), dated December 10, 1992.

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50. American Concrete Institute Standard ACI 318-71, “Building Code Requirements for Reinforced Concrete”.

51. Portland Cement Association paper ST-66, “Design of Deep Girders”.

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FIGURES

Figure 12.2-9 Comparison of Actual Earth Quake Spectrum to Recommended Spectrum

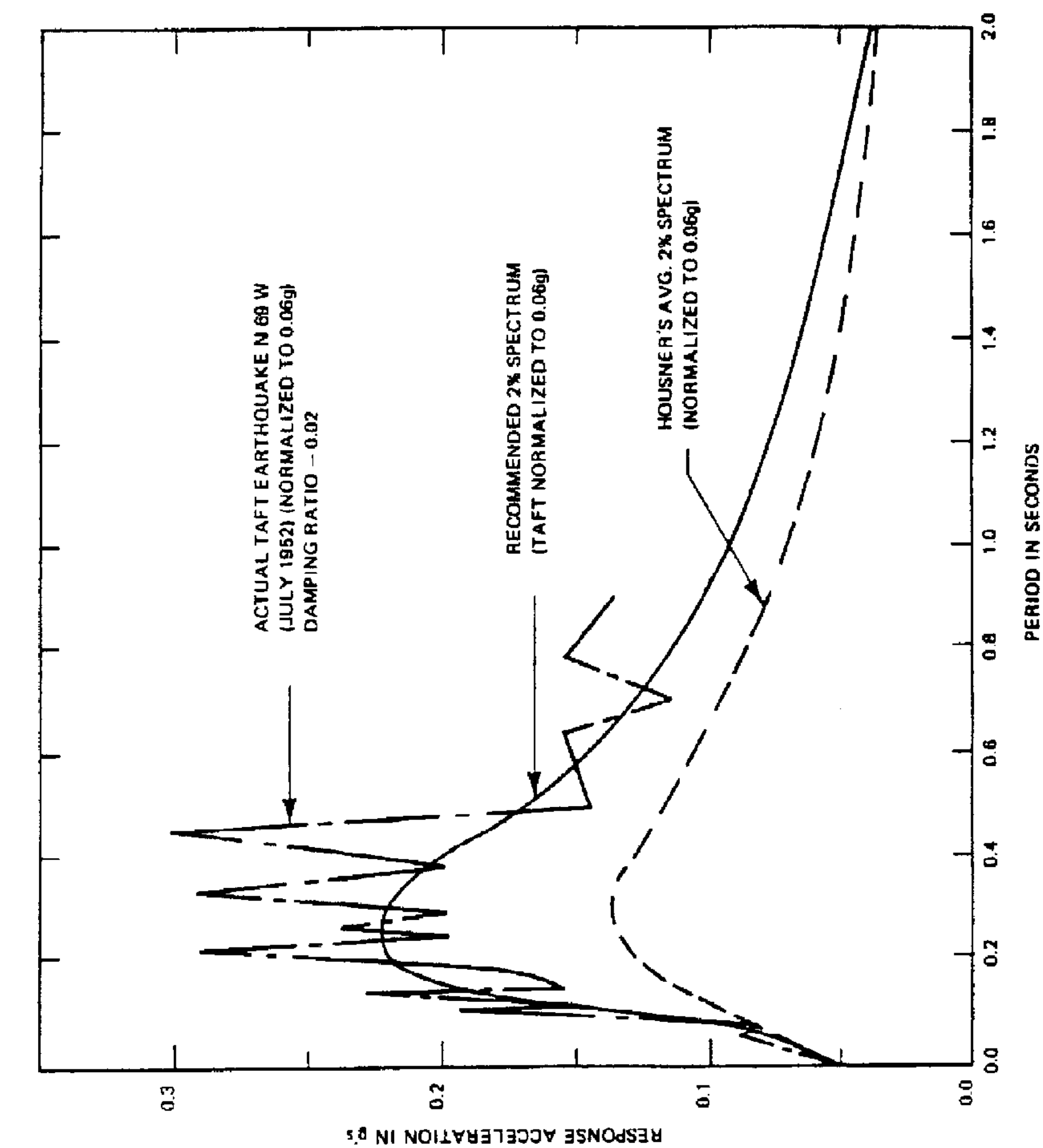

Figure 12.2-10 Drywell and Foundation Cross Section

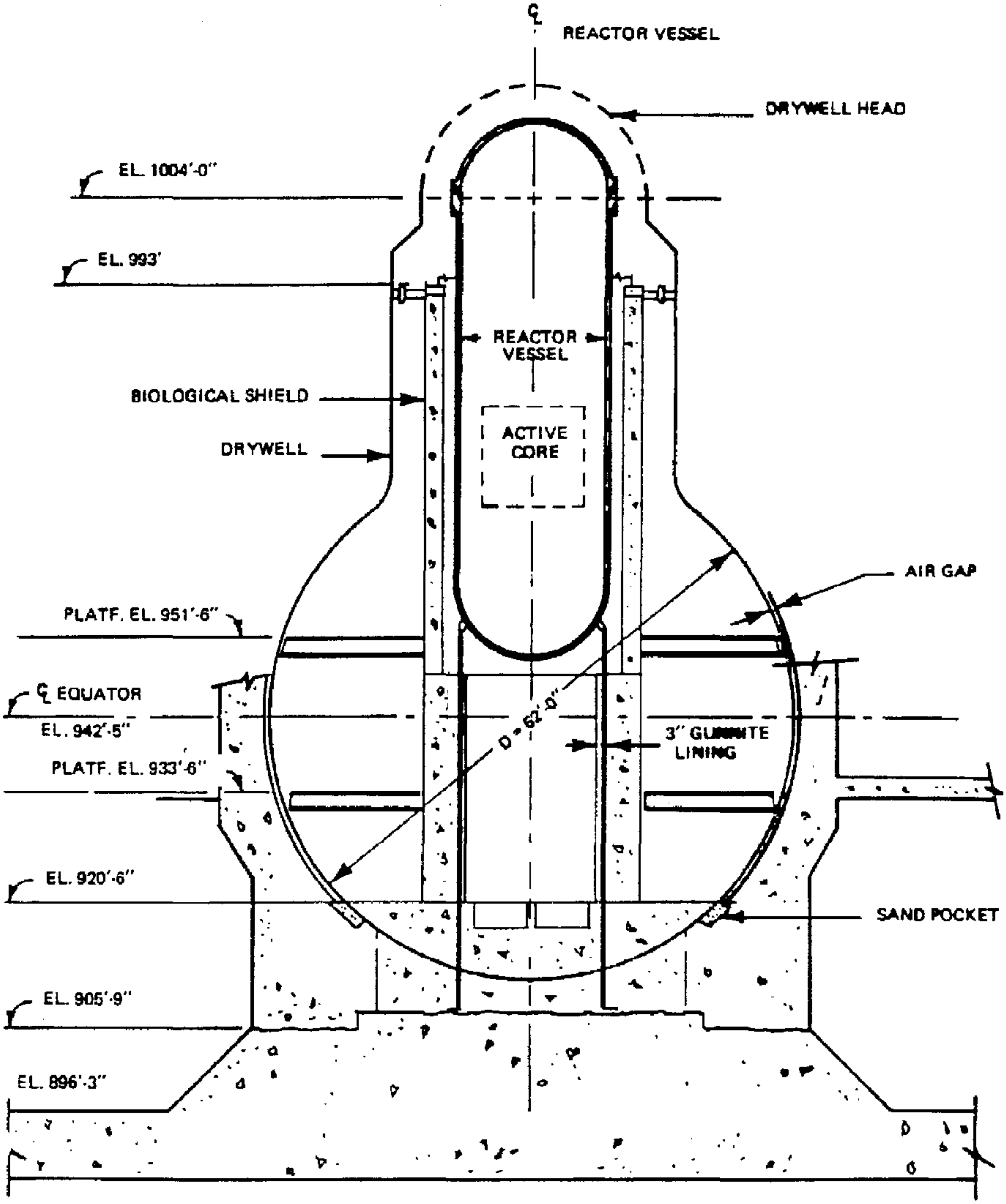

Figure 12.2-11 Main Steam Line Anchor Frame and Restraints