12.2 PRINCIPAL STRUCTURES AND FOUNDATIONS

See Figure 2.2-4 for location of structures.

12.2.1 Loading Considerations for Structures, Foundations, Equipment and Systems

Structures and Foundations

Dead Loads
Live Loads
Pipe and equipment loads
Reactions from cranes
Impact loads for equipment handling
Wind loads - 100 mph (except for Low Level Radioactive Waste Storage Facility [LLRWSF] - 95 mph)
Tornado wind loads - 300 mph (except for LLRWSF - 290 mph rotational at 150 feet radius and 70 mph translational)
Tornado depressurization
Generated missiles
Operating Basis Earthquake loads
Design Basis Earthquake loads
Thermal loads
Jet load - main steam or recirculating line break
Pressure from rupture of primary piping
Lateral earth pressure loads
Ground water hydrostatic pressure loads
Hydrostatic water pressure
Lateral pressure of freshly placed concrete
Temporary construction loads

Reactor Building Crane

Dead loads
Live loads
Impact loads
Trolley and bridge movements
Bridge collision
Motor stall (Bridge only)
Tornado wind loads - 300 mph
Operating Basis Earthquake loads
Design Basis Earthquake loads

12.2.2 Reactor Building (Class I)

12.2.2.1 Concrete Structure Below EL. 565.0

This portion of the Reactor Building is a reinforced concrete structure founded on rock surrounded by earth backfill or adjacent to another unit Reactor Building structure. It is divided into three similar units by a one-inch-wide expansion joint filled with Fiberglas insulation. Each unit consists of a base slab, a circular mass of concrete at the center supporting the drywell, exterior walls and diagonal corner walls, and a top slab which is haunched over the torus and flat over the triangular-shaped corner areas. Floor plans and sections are shown in Figures 12.2-2 through 12.2-6. See Subsection 2.5 for rock foundation and foundation treatment.

12.2.2.1.1 Base Slab

The portion of the base slab under the torus is a circular plate spanning an area from the face of the concrete mass under the drywell to the exterior walls. The slab is assumed fixed at each end and is analyzed for radial and circumferential stresses using the loading cases as given in Table 12.2-1. In no case do the maximum stresses approach the allowable stresses given in Table 12.2-1. Live loads on the slab are not included in the loading cases, since their inclusion would have an offsetting effect on the hydrostatic uplift on the slab. Because the settlement of the concrete mass under the drywell is greater than that of the exterior walls, differential settlement of the supports for the slab is included in the analysis, as applicable, to produce the maximum stresses.

The triangular areas of base slab in the corners of the structure are analyzed as modified two-way slabs fixed at the supports for the loading cases as given in Table 12.2-1.

12.2.2.1.2 Stage I Walls

These walls are the exterior building walls, which extend from the base slab up to El. 547.0, as shown on Section B-B on Figures 12.2-3 and 12.2-6. Construction above El. 547.0 was temporarily discontinued during erection of the torus; and, during this period, these walls were subjected to the applicable construction condition loads as given in Table 12.2-2.

The center portions of the walls are analyzed as vertical cantilevers from the base slab partially supported by ring action. The corner walls are analyzed as two-way slabs fixed on three sides and free at the top. Horizontal bending moments are distributed at the corners by moment distribution.

Stresses resulting from this analysis are combined by the method of superposition with stresses determined in paragraphs 12.2.2.1.3 and 12.2.2.1.4.
See paragraph 12.2.2.10 for method of determining static and dynamic lateral earth pressure.

12.2.2.1.3 Stage II Slab Over Torus

This slab is the lower portion of a final composite thick slab over the torus as shown on section B-B on Figures 12.2-3 and 12.2-6. It is analyzed as a frame fixed at the bottom of the walls, as shown in Figure 12.2-7, for the loading conditions given in Table 12.2-3. Because the supporting walls are a series of chords forming a circle, and the exterior wall varies in thickness, shape, and external load, the slab is divided into 16 equal segments, each of which is analyzed independently.

The slab is analyzed as a member with a variable moment of inertia. Figure 12.2-7 shows a plan and section of a typical segment. Fixed-end moments, stiffnesses, and carryover factors determined by computer program are used in obtaining the final design moments and shears by the moment distribution method.

Stresses resulting from this analysis are combined by the method of superposition with stresses determined in paragraphs 12.2.2.1.2 and 12.2.2.1.4. See paragraph 12.2.2.10 for method of determining static and dynamic lateral earth pressure.

12.2.2.1.4 Completed Structure

The structure below El. 565.0 is analyzed by parts as discussed in paragraphs 12.2.2.1.1 through 12.2.2.1.3 and as a complete structure for the final design loading cases. For views of this portion of the structure, see Figures 12.2-3, 12.2-4, and 12.2-6. As in the analysis of the Stage II Slab Over Torus, the slab over the torus for the completed structure is also divided into 16 equal segments as shown in Figure 12.2-8, and each frame is analyzed independently for its particular shape characteristics and loading conditions.

The exterior wall of each frame is assumed fixed at the bottom of the wall at El. 565.0. To ensure conservative values for the maximum moments and shears in the frames, each frame is analyzed for three restraint conditions at the interior end of the haunched slab. The restraint conditions are: fixed along a vertical plane at the face of support, 75 percent fixed along a vertical plane at the face of support, and fixed along a sloping plane at approximately the bottom of the vent pipe openings. See Figure 12.2-8 for typical frame section showing conditions of restraint. For the first two conditions of restraint, the fixed-end moments, stiffnesses, and carryover factors determined by computer program are used in obtaining the moments and shears by the moment distribution method. For the third condition of restraint, the column analogy method is used to obtain the moments and shears. The frames are analyzed for the loading cases given in Table 12.2-4, and the resulting maximum

stresses combined by the superposition method with the stresses obtained in paragraphs 12.2.2.1.2 and 12.2.2.1.3 to obtain the maximum final stresses in the radial direction.

Horizontal circumferential reinforcing is provided in the slab over the torus and adjacent to the drywell below El. 565.0 for the stresses obtained from a finite element program as discussed in paragraph 12.2.2.2.

The triangular-shaped areas of flat slab in the corners of the structure at El. 565.0 are analyzed as modified two-way slabs for loading cases I and V, as given in Table 12.2-4.

The exterior walls which form the corners of the structure are analyzed as two-way slabs supported on four sides for the loading cases given in Table 12.2-5. Moment distribution is used to distribute the horizontal fixed-end moments at the corners and vertical fixed-end moments at the El. 565.0 floor slab. Stresses from this analysis are combined with stresses obtained in paragraph 12.2.2.1.2 for calculating the maximum final stresses.

See paragraph 12.2.2.10 for method of determining static and dynamic lateral earth pressure. For exterior wall seismic shear stresses and overturning moment stresses, see paragraph 12.2.2.8 (Dynamic Earthquake Analysis).

12.2.2.1.5 <u>HPCI System Rooms</u>

These structures are appendages to the Reactor Building and are located outside the west wall of Unit 1, the south wall of Unit 2, and the east wall of Unit 3, as shown on Figures 12.2-4 and 12.2-6. They are founded on rock at the same elevation as the Reactor Building, and backfill surrounds three sides of the structures and covers the top of the structures to a depth of approximately 40 feet. Ground water level is approximately two to three feet above the top of the structures.

The structures are designed to support construction condition loads of earth and water to the top of the walls before the top slab is in place. Stresses due to this condition are combined with stresses from the final loading cases by the method of superposition to determine the maximum stresses.

The walls and slabs of the structures are analyzed as two-way slabs by the working stress method using the moment coefficients given in Method 2 of ACI Code 318-63. Transverse and longitudinal frames are analyzed by the moment distribution method using the loading cases as given in Table 12.2-6, with lateral earth pressure determined as described in paragraph 12.2.2.10. Vertical accelerations for Operating Basis Earthquake and Design Basis Earthquake are used respectively, to increase or decrease the vertical load, whichever is conservative.

12.2.2.1.6 Personnel Access Locks

Reactor Building locks are indicated in FSAR Figures 1.6-2, 1.6-3 sheets 1 and 2, 1.6-5, 1.6-6, 1.6-11, 1.6-12, and 1.6-13 (see Section 5.3 of the FSAR).

All of the Reactor Building double door locks are identified on FSAR Figures 1.6-2, 1.6-3 sheets 1 and 2, 1.6-5, 1.6-6, 1.6-11, 1.6-12 and 1.6-13 by means of a "radiation" type symbol encircling the letter "A". The large equipment access lock is described in paragraph 12.2.9 of the FSAR and shown on Figure 12.2-80. The other access locks are described below.

The personnel access locks at El. 664 and the equipment access locks at El. 565 are constructed as shown in Figure 12.2-2b. These walls are made of concrete blocks with reinforcement added. The blocks' cavities are filled with concrete when the top slabs are poured to form a monolithic structure. The walls are anchored by dowels to the floor slabs of the building. The pressed steel door frames are mounted as shown in Figure 12.2-2d. The jambs are anchored with three equally spaced straps on each side of the frames, and a sealant is added all around the frames to reduce infiltration. Hollow metal doors are mounted to the frames with weatherstripping to further reduce infiltration.

The personnel access locks at all the other elevations are constructed of poured concrete as shown in Figure 12.2-2c for a typical lock. These reinforced locks are formed and poured in place following standard construction procedures. The walls are doweled to the floor slabs. The pressed steel door frames are mounted as shown in Figure 12.2-2d. The jambs are anchored with three equally spaced cinch anchors on each side of the frames, and sealant is added all around the frames. Hollow metal doors are mounted to the frames with weatherstripping to reduce infiltration.

The loads imparted to all access lock structures, other than the large equipment access locks, are dead loads combined with either Operating Basis Earthquake or Design Basis Earthquake loads. The acceleration values at the various lock levels, calculated by the dynamic analysis of the building, are the basis for horizontal and vertical earthquake loading. The various walls and roof slabs of the locks are designed as structural members to resist the loads described above. Allowable stresses are those specified for Principal Design Cases II and III, as given in Table 12.2-9 in the FSAR.

In the case of locks constructed of concrete block, the reinforcement actually used is that calculated according to the criteria given in the preceding paragraph.

In the case of locks constructed of poured-in-place concrete, the walls contain minimum reinforcement as specified in the ACI 318-63 Code for load bearing walls.

The sections are investigated according to the criteria above, and the calculated stresses are considerably less than allowables.

12.2.2.2 Drywell Concrete Structure

12.2.2.2.1 Introduction

This paragraph covers the structural analysis of the biological shielding surrounding the drywell vessel. The shielding is made of concrete having a minimum 28-day compressive strength of 3000 psi.

The drywell concrete structure is in a biaxial stress condition. There are vertical compressive stresses and tensile hoop stresses. The plane of shear resistance is a horizontal plane upon which compressive stresses act and is unaffected by the tensile hoop stresses. The radial shear stress caused by the outward thrust at the bottom of the conical part of the structure is considered to be diagonal tension, and the allowable stress is given in the ACI 318-63 Code for the condition which includes compressive stress on a section. For the longitudinal or seismic shear, the allowable stress is as given in the SEAOC code for shear walls.
For the splices in the tensile hoop reinforcement in the drywell concrete structure, the allowable stress for bond is as given in the ACI 318-63 Code. These code allowables are for a uniaxial stress condition. They should be conservative for this structure, since the biaxial compression which exists on the splices should improve the bond capability of the concrete.

Steel reinforcing for the shield is of ASTM A432, having a minimum guaranteed yield strength of 60,000 psi. The shielding conforms generally to the "inverted lightbulb" shape of the drywell vessel and is separated from it by an annular layer of polyurethane foam. The geometry of the shielding is shown on Figures 12.2-2 through 12.2-6.

12.2.2.2.2 Discussion of Analysis

The shield is analyzed using the finite element method. The programs[1] used in the analysis utilize quadrilateral elements formed from four constant-strain, triangular, finite elements. Solution of the system of equations is by the Gauss-Jordan method.

Numerous cases were run with varying boundary conditions for the floors and walls to determine sensitivity, and each loading condition was computed in a separate run as well as in combination with other loading conditions.

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[1] Becker, E. B. and Brisbane, J. J., "Application of the Finite Element Method to Stress Analysis of Solid Propellant Rocket Grains," Vol. I, Vol. II, Parts 1 and 2, Rohm and Haas Co., Redstone Research Division, Reports S-76 (AD-476515, AD-474031, AD-476735).

The axisymmetric model of the shield used in the finite element analysis has the structure fixed at El. 519.0, the top of the building base slab. Restraint boundary conditions at the top of this model (El. 621.25) are generated from a second model, fixed at El. 565.0 which has a thick slab at the upper portion simulating the restraining effect of the fuel and dryer-separator pools. The use of two models is necessary due to computer storage requirements of the finite element program. All loading conditions are computed with and without this restraining effect, and the most conservative results are used. In this analysis, the portion of the shield above about El. 615 was considered as stiffness and restraint, and the portion below El. 615.0 was considered "real" structure.

Major non-axisymmetric loads, such as seismic, and the nonaxial effects of the restraint of the fuel pools on the shield were considered to act on the shield as a unit, that is, the shield was idealized as a beam in resisting these loads. Axial, bending, and shear components of these loadings were converted into circumferential and meridional forces and shears. These in turn were combined with the respective forces and shears obtained from the finite-element analysis. These combined results were then used in proportioning reinforcing.

The effect of walls framing into the shield between El. 565.0 and El. 593.0 is estimated by considering plane-strain, finite-element slices through the shield. Radial deformations generated in the axisymmetric model are restored at the points where the walls frame into the shield. Stresses resulting from analyzing the slices are superimposed upon the axisymmetric stresses to obtain final stresses.

Thermal stresses were computed in two ways: from the finite-element model and by the method outlined by the ACI chimney code. Stresses from the finite-element model were converted to equivalent moments and thrusts by using the relations

$$M = \int \sigma \, dA$$

and

$$N = \int y \, dA$$

etc., where:

M	=	Moment per unit length acting on a section
N	=	Thrust per unit length acting on a section
σ	=	Normal stress acting on a section
dA	=	Differential area
y	=	Distance from neutral axis

These moments and thrusts were then used to compute concrete and reinforcing steel stresses. In all cases, this method gave higher stresses than the ACI chimney code and was used in the design.

Stresses around the openings in the shield are computed using stress concentration factors found using planar finite-element analysis.

The stresses around the openings found from the finite element analysis were checked using Figure 70 of Stress Concentration Design Factors by R. E. Peterson. Both solutions yield peak stress concentration factors of about 4.0. A stress concentration factor of 4.0 was used in proportioning reinforcing for the openings.

12.2.2.2.3 Loading Conditions

Loads considered to act upon the biological shield are dead loads of the shield and the fuel and dryer-separator pools, live loads including those from the floor arbitrary live loads, water and equipment loads from the pools, thermal loads induced by having hot vapors or gases inside the drywell and cool air outside the shield, thermal loads induced by the shield attempting to lift the pools while they are restrained by the exterior walls, internal pressures caused by expansion of the drywell and consequent compression of the polyurethane foam in the annulus, and seismically-induced loads for the Operating Basis and Design Basis Earthquakes.

These loads are considered in the following combinations.

Case 1. Prestartup - DL+LL+P

Case 2. Operating - DL+LL+P+THERM+RESTR

Case 3. Operating + Earthquake
- -A. DL+LL+P+THERM+RESTR+OBE
- -B. DL+LL+P+THERM+RESTR+DBE

where:

DL	=	dead load
LL	=	live load
P	=	Pressure transmitted through polyurethane foam at operating temperature
OBE	=	Operating Basis Earthquake (0.1g)
DBE	=	Design Basis Earthquake (0.2g)
THERM	=	thermal load at operating temperatures
RESTR	=	restraint to thermal growth of shield by pools

The following parameters apply.

Concrete Properties

f'c	3000 psi
E c	3.14×10^6 psi

γc	(Poisson's ratio)	0.20
αc	(coefficient of thermal expansion)	6.5×10^{-6} per °F
wc		150 pcf

Steel Properties

fy	60,000 psi
Es	29×10^{6} psi
γs	0.30
αs	7×10^{-6}/°F

Ambient temperature outside shield	55°F
Operating temperature in drywell	150°F maximum

Pressure transmitted through polyurethane foam at	
Normal operation	0.5 psi
Accident	1.0 psi

Criteria: For normal operating conditions plus the Operating Basis Earthquake, concrete stresses are limited to normal code allowable values as set forth in ACI 318-63, and reinforcing stresses are limited to 0.5 fy. For normal operating conditions plus the Design Basis Earthquake, stresses are limited to 0.85 f'c and 0.9 fy.

12.2.2.2.4 Results

Results for the design cases consist of computer plots of deformation, radial, axial, hoop, and shear stresses, and maximum and minimum principal stresses.

To arrive at reinforcing steel requirements, various sections are analyzed by transforming the stress distributions into normal and tangential stresses and then finding normal loads, shear loads, and bending moments in both the meridional and circumferential planes. Seismic stresses are considered to increase or decrease the meridional normal load on a section. Seismic stresses due to vertical accelerations and overturning moments caused by horizontal motion are considered to act simultaneously. Using these moments and forces, the sections are then designed using the working stress method as outlined in ACI 318-63, using allowable stresses as given above under paragraph 12.2.2.2.3, "Loading Conditions."

12.2.2.3 Concrete Structure Above EL. 565

12.2.2.3.1 Exterior Walls

The exterior walls are designed to resist stresses from shears, moments, and deflections resulting from earthquake forces as described in paragraph 12.2.2.8, earth pressure loads, normal wind loads, tornado wind loads, and a rapid depressurization as a result of a tornado. Loading cases and allowable stresses are as given in Tables 12.2-7 and 12.2-8.
General outline features are shown on Figures 12.2-2 through 12.2-6.

Earthquake horizontal shearing stresses and vertical tensile or compressive stresses are calculated using the properties of the drywell and element walls parallel to the direction of the earthquake motion. Since the height-to-width ratio of the exterior walls is less than one, they can be considered as shear walls when assigning an allowable shear stress. For the 0.2g Design Basis Earthquake, an allowable ultimate shear stress of $5.4\varnothing \sqrt{f'c}$ is used. This is the value specified in the SEAOC Code[2] in Section 2631(c) for walls with a height-to-width ratio less than one. For the 0.1g Operating Basis Earthquake, an allowable value of one-half of the above is used. The calculated shearing stresses in the walls do not exceed these values.

Moments in the walls resulting from loads perpendicular to the plane of a given wall are calculated by Method 2 for two-way slabs as given in ACI Code 318-63 or as one-way slabs, using the method applicable to each individual structural layout. The vertical tensile or compressive stresses due to earthquake are considered as axial loads in combination with these moments. Reinforcing is determined by the working stress design method.

Wind loads are determined as described in paragraph 12.2.2.9. For a tabulation of stresses resulting from 300-mph wind and rapid depressurization, see reply to question 2.1 of Amendment 2 to the Unit 3 Design and Analysis Report. For a discussion of the upper limit tornado that the walls can withstand, see reply to question 2.3 of Amendment 2 to the Unit 3 Design and Analysis Report. For a discussion of the capability for resisting penetration by tornado-generated missiles, see paragraph 12.2.2.9.2.

Lateral earth pressures are determined as described in paragraph 12.2.2.10.

The expansion joint loads as listed in Table 12.2-8 are determined by two factors: (a) the lateral movement, during earthquake, of adjacent Reactor Buildings, and (b) fluid pressure of freshly placed concrete.

The lateral movement of the Reactor Buildings is determined by the dynamic earthquake analysis described in paragraph 12.2.2.8. It is assumed that the two

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[2] Seismology Committee, Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary," San Francisco, California, 1967, p 26.

buildings move in opposing directions, thereby reducing or expanding the width of the expansion joint by their movement. Using the compression characteristics of the 1-1/2-inch-thick Fiberglas insulation in the joint and the amount the material is compressed, the load imposed on the walls by lateral movement during earthquake can then be determined.

The fluid pressure of freshly placed concrete is determined by the methods given in ACI Publication SP-4.[3] The rate of placement is limited to three feet per hour, or two feet per hour for temperature of concrete, in the forms of 70°F and above or below 70°F, respectively.

12.2.2.3.2 Floor Slabs, Beams, and Columns

Floor Slabs

In general, the slabs are two-way slabs with support on four sides by beams and walls. They are designed by using the moment coefficients given in Method 2 for two-way slabs in ACI Code 318-63. Design cases I, II, and III in Table 12.2-9 are used. Where equipment occurs on floor slabs, the actual equipment load per square foot is computed to determine if it exceeds the uniform floor live load. The larger of these two values is used as the live load. The maximum building vertical acceleration at the elevation for the slab is used to increase the vertical loads.

At the 639 floor, vibration isolators are used under the variable speed motor-generator sets to prevent the structure and the machine from becoming resonant.

The slabs at El. 606 and 617 between columns n and p are one-way slabs with the dead load supported by metal decking spanning two steel floor beams. The steel beams are supported on concrete brackets or pockets at the n and p line walls. The slabs are designed by the moment distribution method using design cases I, II, and III in Table 12.2-9. DL is not included, as the metal decking carries this load.

The roof slab at El. 635 is a one-way concrete slab spanning between the n and p line walls. The slab is analyzed as a part of a frame consisting of the n and p line walls, with the connecting slabs at El. 635, 617, 606, and 593. The analysis is by the moment distribution method using design cases I, II, and III of Table 12.2-9. The area in the center of the building where ventilating ductwork occurs is designed for 150 psf equipment load and reactions from steel columns that frame the duct enclosure.

[3] Hurd, M. K., "Formwork for Concrete," Special Publication No. 4, American Concrete Institute, Detroit, Michigan, 1963, pp 74, 75.

Beams and Columns

As can be seen on Figures 12.2-2 through 12.2-6, concrete frames occur on the building column lines. The frames are analyzed for the cases given in Table 12.2-9 using the IBM FRAN computer program. Each frame is analyzed for various load combinations, including moments, shears, and axial loads from the steel frames above the refueling floor, so as to produce maximum positive and negative moments. The effect on the frames resulting from thermal growth of the drywell is included. The moments induced in the columns by differential displacement of column ends as a result of the building deflecting horizontally from earthquake are included. These moments are calculated by assuming that column ends do not rotate but are displaced. The moments are added directly to those calculated for the columns by the computer program. Vertical accelerations from the Operating Basis Earthquake and Design Basis Earthquake are used to increase or decrease the vertical load, whichever is conservative.

The beams and columns are designed by the working stress method using the moments, shears, and axial loads obtained by the frame analyses using the allowable stresses given in Table 12.2-9. In addition, the columns are checked by a TVA-developed computer program that determines the ultimate strength of a given section for combinations of direct load and biaxial bending moments.

12.2.2.3.3 Reactor Support Pedestal

The reactor pressure vessel concrete support pedestal is analyzed and designed using the Finite Element Method and Cantilever Beam Method for the loading cases as given in Table 12.2-10, with earthquake loads determined as described in paragraph 12.2.2.8.2. The jet load used in the design is the worst condition of either a clean break of the 26-inch main steam reactor pressure vessel penetration resulting in a jet reaction force of 595 kips, or a clean break of the 28-inch recirculation loop outlet penetration resulting in a jet reaction force of 658 kips. The Finite Element Method is used to determine the stresses in the support pedestal due to normal operating loads and temperature effects. The Cantilever Beam Method is used to determine the stresses due to earthquake and jet loadings. Resulting stresses are combined by the method of super-position, to determine the maximum stresses in the pedestal, and are as given in Table 12.2-10. Loads due to accidental rupture of various service piping, anchored to the concrete pedestal with embedded plates, are considered in obtaining the final stresses in the pedestal.

Earthquake and jet loads acting on the reactor pressure vessel produce horizontal shears in the RPV ring girder which are transferred to the support pedestal through bearing and shear. See Table 12.2-10 for section through top of pedestal.

The ring girder is designed to transfer the vertical and horizontal loads of the RPV skirt flange to the top of the RPV support pedestal.

The vertical loads on the RPV skirt flange are transferred to the top of the RPV support pedestal by the ring girder acting as a bearing plate. The ring girder is designed according to AISC Code Specifications.

The horizontal shears on the RPV skirt flange are transferred to the top flange of the ring girder by sixty A490 high-strength bolts in the same friction-type connection as is described in the AISC code.

The amount of frictional force available to resist horizontal shear is directly proportional to the normal pressure (proof load) between the RPV skirt flange and top flange of the ring girder. The total frictional force and the coefficient of sliding friction are independent of the areas in contact, so long as the total pressure remains the same. The friction-type connection of the RPV skirt flange to the ring girder, in which some of the bolts lose a part of their clamping force (proof load) due to applied tension during an earthquake, suffer no overall loss of frictional shear resistance. The bolt tension produced by the moment is coupled with a compensating compressive force on the other side of the axis of bending.

The total frictional force due to a coefficient of friction of 0.15 and a proof load of 405 kips per bolt is 3650 kips, or 13.5 times the Operating Basis Earthquake shear load of 269 kips, or 6.8 times the Design Basis Earthquake shear load of 538 kips. However, if the coefficient of friction is assumed zero, the bolts as bearing-type connection could resist a total horizontal shear of 17.8 (at AISC code stresses) times the Operating Basis Earthquake shear load of 269 kips, or 27.9 (at 90 percent of yield stresses) times the Design Basis Earthquake shear load of 538 kips. Therefore, the high-strength bolt connection of the RPV skirt flange to the top flange of the ring girder, with or without friction, is more than adequate for the respective design load.

The anchor bolts connecting the ring girder to the concrete pedestal resist the overturning moments for cases I and II as given in Table 12.2-10. Using the number of anchor bolts actually supplied, the maximum anchor bolt tensile stress is calculated to be less than 12,000 psi for case II. Allowable stress for case II is 0.90 fy or 32,400 psi. Under case I loading, there is no tension in the anchor bolts.

The transmission of horizontal forces (from earthquake and jet load) is described in paragraph 12.2.2.3.3. It is to be noted that no friction between ring girder and concrete or bearing on the anchor bolts in the pedestal is included. Safety factors based on the allowable and actual shear stress values, as given in Table 12.2-10, are 3.6 for case I and 1.97 for case II, neglecting any shear resistance by the anchor bolts.

The allowable stress values are those given in the ACI code for shear stress in slabs and footings. The condition which exists is more of a punching shear condition.

TVA, in shear tests on unreinforced concrete at its Tims Ford Dam, has determined that the ultimate shear stress for a punching condition is about twice the ultimate tensile strength of concrete. Tensile strength of concrete is generally accepted to be about 6 $\sqrt{f'c}$ and these tests confirm this. Using 2 x 6 $\sqrt{f'c}$ gives factors of 22 for case I and 7 for case II.

The annular gap between the ring girder anchor bolts and the holes in the ring girder bottom plate and sole plates is filled with Sika Colma Dur in order that the bolts will act as dowel pins, and thus the two plates will act as a unit in transferring the horizontal load to the grout through bearing on their vertical edges. The projected vertical area of the plates is used in computing the bearing stress on the grout.

The horizontal load in bearing on the grout is transmitted through shear in the grout and shield wall concrete to the pedestal concrete. A 120 degree segment of the annular area between the sole plates and the outside face of the pedestal is used in computing the shear stresses.
An additional capability for resisting horizontal shear, which exists but is not relied on in the above analysis, is the frictional resistance between the sole plates and the grout and bearing of the anchor bolts on the concrete below the sole plates.

12.2.2.3.4 <u>P-Line Wall</u>

This wall is located on column line p as shown on Figures 12.2-2 through 12.2-6. In addition to its treatment as an exterior wall (as discussed in paragraph 12.2.2.3.1), it is also designed as a continuous beam for the loading cases and allowable stresses given in Table 12.2-11.

This beam is supported by the east and west exterior walls of the Reactor Building, the east and west shield walls of the steam line compartment, and two interior columns. The portion of the beam below El. 635.0 is analyzed with the vertical loads applied in stages as the wall is constructed, with each depth of beam, as completed, supporting the wet concrete weight of the next pour plus other applicable superimposed loads. The beam below El. 635.0 is then analyzed for all remaining vertical loads. The vertical tensile or compressive stresses due to earthquake are considered as vertical loads.

The determination of bending moments recognizes the elastic shortening of the columns which are the first interior supports. In computing stresses produced by these moments, the method of transformed cracked section is used, and, in addition, the nonlinear stress variation of deep beams is recognized. Stresses for successive stages of construction are combined by the method of superposition.

12.2.2.3.5 Fuel Storage Pool and Dryer and Separator Storage Pool

The fuel storage pool and the dryer and separator storage pool, together with the drywell concrete biological shield wall and the portions of the Reactor Building exterior walls that support the pools, are considered a structure. Viewed as a whole, with all other parts of the Reactor Building removed, the structure is a massive hollow central support with two horizontal members, beams, or pools extending east and west for a total two-span length of about 150 feet, the ends being supported by three-foot-thick walls. These supporting walls extend down to El. 565.0. For each supporting wall, the north-south extent of this thickness is the same as for the pool it supports. General outline features are shown on Figures 12.2-2 through 12.2-6.

During reactor operation, heat produced within the drywell causes an increase in the total height of the central support of this structure. The structure as a whole is analyzed to determine the moments and shears associated with this "drywell thermal rise." The effects of deflection of supports and shear deflection of the pool walls resulting from the total imposed gravity loads are included. No other analysis is made of the structure as a whole. These moments and shears exert a major influence upon the design of reinforcement for the east-west walls of the pools and for the drywell walls joining the pools. Stresses resulting from this analysis are combined by the method of superposition with stresses computed in the analysis of the structure by parts.

In the analysis of the structure by parts, each pool is designed to span as a beam from its exterior support wall to the drywell shield wall. Each is assumed fixed along a vertical plane approximately at the face of the drywell shield wall and pinned at its exterior support. To assume conservative values for maximum positive moments in the fuel storage pool, it is also designed for 67 percent fixity at the drywell shield wall. The walls and slabs that form the pools are analyzed for their individual framing and loading conditions.

Principal design cases, allowable stresses, and material properties are given in Table 12.2-12. Vertical accelerations from the Operating Basis Earthquake and Design Basis Earthquake were used to increase or decrease the vertical load, whichever is conservative.

The spent fuel storage pool is built up to its full height in stages. The shores below the slab remain in place until the pool walls and the adjacent half of the drywell shield wall are constructed to a point approximately 11 feet above the top of the slab. After these walls have attained sufficient strength to support the structure built to this point, plus the next wall pour, the shores are removed from below the slab.

On the opposite side of the drywell, the dryer and separator pool are constructed by stages in a different way. After the bottom slab is poured and attains suitable strength, the slab shoring is removed, leaving the slab supported on one end by the

exterior end wall, near midspan by rigid shores on the top of a perpendicular wall located below the slab, and on the other end by the drywell shield wall. Concrete is then placed to a point approximately 14 feet above the top of the slab, and the pool walls are designed as two-span, one-way members for this construction condition. After these walls have attained sufficient strength to support the structure built to this point, plus the next wall pour, the slab shores from the midspan supporting wall are removed; and the walls of this pool become single-span members between the drywell shield wall and the end wall.

The end walls that support the pools are analyzed as discussed for exterior walls in paragraph 12.2.2.3.1, with the vertical reactions and moments from the pools included. Also included is the lateral displacement of the tops of the walls due to an assumed 46° thermal lengthening of the pools caused by warm water in the pools.

The slabs of both pools are suspended from the walls above, along much of their perimeter. In both slabs, most reinforcement is proportioned for bending with axial tension, and shear reinforcement is provided to carry diagonal tensions that exceed the allowable shear stress on the concrete. The effect of a 60 degree thermal gradient through the concrete, caused by hot water in the pools, is included in the design.

For the dead load, live load, and water load design case, the bending moments used in the final design are quite conservative. The fuel storage pool slab is assumed fully restrained at its supports for negative moment, and the corresponding midspan positive moments are increased approximately 50 percent to include the effect of possible joint rotation. In the dryer and separator storage pool, moment distribution is used for obtaining the design negative moment, which is 85 percent of the fixed end moment. The positive moment from the moment distribution is increased by an amount equal to 50 percent of the negative moment to arrive at the design moment.

The dryer and separator storage pool slab is designed as a one-way slab. The fuel storage pool slab is designed as a two-way slab; and in the east-west direction, the edge strips of this slab are also stressed in tension by the positive moments associated with the beam action of the pool walls as they span from the drywell shield wall to the end wall. The edge strip reinforcement is adequate for this combined loading.

The east-west pool walls are analyzed for both horizontal and vertical stresses. In the analysis of the walls as beams, the vertical loads are applied in stages as the walls are constructed. In computing stresses, the method of transformed cracked section is used; and for the fuel storage pool walls, the nonlinear stress variation of deep beam action is recognized. The horizontal stresses for the successive stages of construction are combined by the method of superposition.

In the vertical direction, the moments at the bottom of the walls are the same as the slab end moments with the slab reactions applied as axial tensions at the bottom of the walls. These axial tensions are slowly decreased for the design of vertical reinforcement at higher elevations. Stresses are computed for bending with axial tension, and a 'stirrup stress' is added to this stress in recognition of the horizontal beam action of the wall. Reinforcing bars inclined at 45° are provided at the drywell end of the walls of both pools to carry the total shear at the face of the drywell shield wall, with no allowance for shear carried by the concrete.

The north-south water wall at the end of the fuel storage pool opposite the drywell carries a major portion of the load on the pool slab and is, in turn, supported by the main load-carrying horizontal beams (east-west pool walls). This wall is analyzed both horizontally and vertically for bending with axial tension.

The fuel transfer canal between Units 1 and 2 creates a notched condition in this wall. The effect of this notch is accounted for in the design by the use of reinforcement inclined at 45° under the notch to transfer the wall reaction into the main load-carrying beam with no allowance for shear carried by the concrete.

The fuel storage pool wall on the north side of the pool is subjected to horizontal loading at the fuel transfer canal, which creates a condition of a vertical span with support by the intersecting north-south water wall. Vertical elements of this thickened canal wall are reinforced for the substantial bending moments involved. These same vertical elements are securely anchored to the north-south pool wall by means of heavy horizontal reinforcement located immediately below the transfer canal.

In order to minimize the possibility of pool leakage, these pools are lined with stainless steel.

No inlets, outlets, or drains are provided that may permit the fuel storage pool to be drained below approximately 10 feet above the top of active fuel.

Interconnected drainage paths are provided behind the liner seam welds for the pools. These drainage paths are formed by welding channels behind each weld joint which permit free gravity drainage to the equipment drain tank. These paths also prevent pressure buildup behind the liner plate, prevent the controlled loss of contaminated pool water, and provide expedient liner leak detection and measurement.

All seam welds are subjected to dye-penetrant testing.

12.2.2.3.6 Main steamline Anchor Wall in Steamline Vault

The anchor separating the Class I from the Class II part of the main steamline consists of a concrete wall with the main steamline passing through it. Forces from the piping system are resisted by a 4-ft thickness of concrete wall in the steam line vault as shown in FSAR Figures M.0-1; 1.6-8 sheets 1, 2, and 3; and 12.2-3 which is designed to span horizontally and vertically to the supporting walls and floor.

12.2.2.4 Steel Structure Above El. 664

The Reactor Building structure above the refueling floor consists of transverse steel rigid frames and horizontal girt framing supporting insulated metal-siding exterior walls. The rigid frames are spaced on 26-foot centers and provide support for the roof, siding, and crane runway. For longitudinal expansion, the superstructure is provided with joints between units by using a double row of frames spaced 4 feet apart.

Rigid frames are built up of a horizontal member and two haunched column sections, which, in turn, are made up of solid web plates and flange plates. Web plates and flange plates are of varying width and thickness to suit the moments and shear loads at points along the frame. The horizontal member is 43 inches deep at the center, haunched to 63 inches at the corners. The top flange is sloped 3/16-inch per foot from center of frame to corner for roof drainage. Vertical column sections are 67 inches deep maximum, and the haunches are 64-1/4 inches maximum at the corners. The general shape of the column above the crane runway is determined by the clearance requirements of the building crane. Maximum plate thickness is limited to 2-1/2 inches. The frames are designed with fixed bases to resist lateral forces from the overhead crane, wind loads, and seismic loading, in addition to supporting the vertical dead and live loads. A typical cross-section of the superstructure showing the intermediate rigid frames is shown by Figure 12.2-21.

Overhead crane runway girders are supported on thick plate brackets which are an integral part of the columns. This is accomplished by slotting the column flange so the thick bracket plate can be inserted to replace the web for the full depth of the column at this point. The crane runway girders are fabricated by welding 16-inch flange plates to a 42-inch-deep web plate. The top flange of the girder is strengthened for seismic and crane lateral loads by welding a horizontal wide flange section to the flange plate. The girders are continuous over two spans. Bumpers are provided at each end of the crane runway to take the force produced by a collision with the crane traveling with full load at full speed with power off. The crane stops on the east end of the Reactor Building provide a tie-down for the crane holding it against the bumpers preventing crane movement when subjected to the forces of short-term tornado or earthquake loadings. An alternate method of tie-down is provided when the crane is located away from the crane stops wherein slings attached to the superstructure rigid frames prevent movement of the crane. The crane runway layout and girder details are shown by Figure 12.2-22a. Vertical x-bracing is installed in two adjacent end bays of each unit in the longitudinal

direction of the Reactor Building. Diagonal bracing is provided in the plane of the roof to carry seismic and wind loads from the roof framing to the longitudinal vertical x-bracing. Wide flange sections are used for longitudinal bracing to satisfy stress requirements and harmonize with the lines of the rigid frames. Structural tee sections are used for diagonal roof bracing.

Roof framing consists of conventional rolled sections supporting metal decking and built-up roof. Heavy roof framing is required to support the dead load, seismic load, and wind load on large tanks, fans, and ducts located on the roof. Beams located at rigid frame interior corners are haunched and welded to the frame to provide support for the frame flange at this point of high compression. Wind and seismic loads from end columns are taken into roof bracing by haunching roof beams adjacent to the columns, thereby preventing torsion in the frame horizontal member.

Exterior insulated siding is suspended 12 feet off the center of the frame columns for architectural appearance by a girt system of channels and box beams. The siding is stepped in at the top and base of the frame in two-foot steps. Girts are designed to remain in place under tornado wind loading.

Design of the superstructure is based on the 1963 American Institute of Steel Construction (AISC) "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings." For either structural re-design or re-evaluation, the 8th Edition of the AISC Specification may be used. The one-third increase in allowable working stress for the design seismic loading as allowed by the AISC Specification has not been used, and may not be used for either re-design or re-evaluation. An allowable stress of 90 percent yield is used for tensile or bending stresses for the maximum hypothetical seismic loading or tornado wind loading. All material conforms to ASTM A-36 except anchor rods, which are ASTM A-307 steel. Shop connections are ASTM A-502 Gr 1 rivets or welding, and field connections ASTM A-325 high-strength bolts. Load combinations used in designing the superstructure with corresponding allowable stresses, are listed n TVA’s General Design Criteria Document BFN-50-C-7100 Attachment C, “Unique Criteria and Commitments for Civil Structures,” The stresses resulting from various loading combinations are less than the allowable stresses.

Seismic design loads were obtained as described in Section 12.2.2.8.3.

The average design wind pressure is obtained by applying a shape factor of 1.3 to the dynamic wind pressure, which is the product of one-half the air density and the square of the design velocity. The value 1.3 is the shape coefficient for a typical building with vertical walls normal to the wind direction, consisting of windward side coefficient of 0.9 and leeward side coefficient of -0.4. The normal design wind of 100 mph corresponds to an average design pressure of 33 lb/ft^2 (consisting of 23 lb/ft^2 on windward side and 10 lb/ft^2 on leeward side) and the tornado wind to a design pressure of 297 lb/ft^2 (consisting of windward and leeward sides combined).

12.2.2.5 Reactor Building Crane

12.2.2.5.1 Description

The Reactor Building crane is a single-trolley, overhead electric traveling-type with a 125-ton main hoist, a 5-ton auxiliary hoist, and a span of 106 ft 5 in. The general arrangement of the crane is shown in Figures 12.2-22b and 12.2-22c. All motions are driven by DC motors.

The Reactor Building crane is designed single-failure proof. A replacement trolley was installed in 2004 with a single-failure proof main hoist, as discussed further in Section 12.2.2.5.2. Single-failure protection is provided for all crane components except the hoist wire rope drum shell, trolley cross girts and bridge girders. These components are conservatively designed and in accordance with NUREG-0554 are not required to be single-failure proof.

The one crane is used for three reactor units. It handles the spent fuel casks, dry storage casks and equipment, equipment for the service and maintenance of the reactors, and equipment which is received or shipped through the equipment access lock.

The structural portion of the crane bridge consists of welded box-type girders and welded, haunched, box-type end ties. The girders and end ties were designed as simple beams in the vertical plane and as a continuous frame in the horizontal plane. Welded steel construction is used for the structural portion of the trolley, and all members were designed as simple beams. Structural portions of the bridge and trolley were fabricated from A-36 and A516 Gr. 70 steel, in accordance with section 1.23, Part I, of the "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," as adopted by the American Institute of Steel Construction. All welding and qualification of welders was in accordance with the "Standard Code for Welding in Building Construction," of the American Welding Society.

Load combinations used in designing the crane, with corresponding stresses, are listed in TVA's General Design Criteria Document BFN-50-7111, "Cranes and Hoists." The crane was designed to withstand the DBE when loaded and a 300-mph wind when unloaded. Stresses do not exceed 0.9 of the yield point (Fy) for the wind condition.

A seismic dynamic analysis was made on the crane bridge using a coupled model of the crane, including replacement trolley, and reactor building superstructure as described in Section 12.2.2.8.3. For load combinations including tornado winds, it was assumed that the crane with no load would be parked, that the crane bridge would be held against two bumpers and two wheel stops, and the trolley would be

held against two bumpers and two wheel stops. Alternately, the crane bridge may be restrained by slings tied to the superstructure rigid frames. For load combinations including earthquakes, a live load of 100 percent of the rated crane capacity was used, and the pendulum effects of the suspended load were considered. The dynamic model of the crane and the seismic analysis utilized the provisions of ASME NOG-1 as guidance, although some design basis considerations were retained for consistency with previous analyses. Horizontal accelerations were assumed to act in one direction only at any given time.

Refer to Appendix C, section C.8, for a description of the establishment of "The Control of Heavy Loads," in accordance with Generic Letter 81-07, "Control of Heavy Loads," at the Browns Ferry Nuclear Plant.

12.2.2.5.2 Safety Features

A replacement trolley was installed in 2004 with single-failure proof safety features provided for the main hoist in accordance with Ederer's Generic Licensing Topical Report EDR-1. Ederer's Generic Topical Report EDR-1 meets NUREG-0554 and NUREG-0612 requirements for single-proof failure cranes. Ederer's eXtra Safety And Monitoring (X-SAM) system includes the Hoist Integrated Protective System (HIPS) that is comprised of a energy absorbing torque limiter, emergency drum brake system, failure detection system, drum safety structure, wire rope protection, and emergency stop button. The auxiliary hoist includes one gearing system and two brakes. One of the brakes is a high speed holding brake mounted on the gearing system input shaft, and the other brake is an emergency band brake that stops the rotation of the hoist drum when it is actuated by the Failure Detection System. The emergency band brake for each hoist has the thermal capacity to allow lowering of the design rated load continuously from the maximum hook height at rated speed, without exceeding the temperature limits of the brakes. The emergency load lowering capability provided by the Emergency Drum Brake System is in addition to the conventional emergency method, which relies upon the crane's high speed holding brakes.

Each hoist includes two upper travel limit switches, one lower travel limit switch, and overspeed switches set to trip at 125 percent of maximum rated speed for both hoists. In addition, the main hoist is provided with a hydraulic equalizing cylinder in place of the conventional equalizing sheave in order to prevent dropping of the load in case of a single rope failure. The auxiliary hoist has a two part whip-style reeving so that a single rope failure will not result in load drop. Holding brakes for the hoists are the spring-set, electrically released type with provisions for manual release of the brakes. The capacity of the main and auxiliary hoists high speed holding brakes is 150% of rated capacity of their respective motors.

Single-failure protection is provided for all crane, trolley, and main hoist system components except the main hoist wire rope drum shell, trolley cross-girts and bridge girders. This protection is provided by the following features.

a. The Dry Cask Storage spent fuel cask is attached to the main hoist sister hook by a lift yoke with two hook pins. To provide a high degree of protection against failure, the lift yoke is designed to the higher factors of safety specified in ANSI N14.6.

b. The main hoist lower main block, upper sheave nest, and hoisting ropes are separated into two independent units. Either unit will support the loaded cask in the unlikely event of failure of the other unit. The equalizing cylinder prevents application of a shock load on the remaining hoisting rope after sudden complete failure of the first rope. The oil-filled cylinder is bolted to the trolley, with one of the hoisting ropes attached to each end of the connecting rod of the double-acting piston. Oil flow from one end of the cylinder to the other is restricted to limit the piston velocity, if unequal loads were suddenly applied to the two hoisting ropes.

c. The main hoist drum is supported by stub shafts and roller bearings. In addition, safety structures are provided at each end and at the center of the drum. In the unlikely event of failure of the shaft, bearings, or any part of the drum, these plates will limit the drum movement in the horizontal or vertical directions to 1/4 inch.

d. The main hoist is equipped with a single-failure proof hoist system designed in accordance with Ederer's Generic Licensing Topical Report EDR-1 to provide compliance with NUREG-0554. The brake is mounted on the high-speed shaft of the reducer. The emergency band brake engages the drum and has sufficient torque capacity to stop and hold a rated crane capacity load lowering at 125 percent of rated top speed.

e. Each bridge and trolley truck is equipped with a drop bar which limits the drop to 1/2 inch in event of failure of any part of the wheel assembly.

f. Tie-down devices on the bridge are provided for holding it against a set of bumper and wheel stops, or, alternately, the bridge may be restrained by slings tied to the superstructure rigid frames during a tornado. Tie-down devices on the trolley are provided for holding it against a set of bumper and wheel stops during a tornado.

g. The bridge and trolley are both equipped with double flanged wheels. The trolley is equipped with bolted lugs which extend around both sides of the rail head and positively prevent the trolley from leaving the rails. The seismic analysis of the crane and superstructure discussed in Section 12.2.2.8.3

indicates that there is no uplift of either the bridge or trolley wheels in a seismic event.

h. All the crane controls are spring-returned to "off."

i. All motions are provided by DC motors driven from M-G sets with regenerative braking under normal operation. An emergency band brake engages the wire rope drum when actuated by the Failure Detection System. Engagement of the band brake is triggered by detection of a discontinuity by a mechanical encoder mounted off the end of the main and auxiliary drum shafts, or by separate overspeed detection switches. The emergency band brake for each hoist has the capability to lower the suspended load in a safe, controlled manner.

j. The crane is provided with a manual-magnetic main power supply contactor that can be operated manually from the cab, by a pushbutton in the cab. This contactor controls the power supply for all motions and does not interface with the new X-SAM control.

k. A second and separate contactor, or circuit breaker, is provided in the power supply to the main crane feed rails, which can be operated by three emergency stop pushbuttons on the operating floor (El. 664.0). These pushbuttons are located on column line p at the centerline of each reactor. These pushbuttons do not directly interface with the new X-SAM control.

l. The torque of all motors is limited by a current-limiting device to 200 percent rated for the hoists and to 150 percent rated for all travel motions. Ederer provided new motors for the replacement trolley with the same rating as the existing motors. The existing current limiting devices have not been changed.

m. Undervoltage protection is provided on all motions to sense low, or loss of, control voltage and cause the driven equipment to stop.

n. Overload protection is provided by instantaneous overcurrent relays on the DC motors set at about 250 percent rated current to back up the torque limiting device and by inverse time delay overload devices on the AC motors of the MG sets set to trip at 150 percent full load current.

o. Minimum motor shunt field protection monitors the loss of motor field current and stops the respective drive if the motor loses field current.

p. A torque-proving circuit checks that current is actually flowing in the main and auxiliary hoist motor's armatures before the motor brakes are permitted to be released.

q. Two overhoist limit switches and one down-travel limit switch are provided on each hoist.

r. A mechanical overspeed switch is provided on the main and auxiliary hoist drive motors to trip at 125 percent of top rated speed in either direction to stop the hoist motor and set the holding brakes.

s. A monitor is provided to sense phase reversal or loss of one phase of the AC power supply. If loss of one phase or phase reversal were to occur, the drive cannot be started if it is stopped, and the drive will be stopped if running.

t. An operational check circuit is provided to back up the operator's command to the control whereby, in case the drive does not react to the operator's command within a preset time, the drive will be automatically stopped.

u. Runaway limit switches, mechanical trips for the bridge mounted limit switches, and bridge and trolley overspeed switches are used to monitor bridge and trolley speed and to decelerate either when the bridge or trolley is in the runaway condition and to prevent a full speed impact of the crane into the bumper stops located at the east and west ends of the crane wall or of the trolley into the stops at each end of the bridge girders.

v. Relocation of solid state control components which require temperature environments below 104°F into air conditioned control panels. This ensures that an environment is maintained which is suitable for the electrical components.

w. The hoist drum overspooling/overlapping feature is per vendor supplied drawings to trip only the hoist in use. This is the vendor's recommendation and standard industry practice for operation of the hoists.

The features described above apply to the main hoist system on the replacement trolley. Ederer also supplied an auxiliary hoist system with a compact lower block and two part reeving system. The hoist is similar to the main hoist, with the clarification that the two part reeving system does not provide load balance in one horizontal direction, or hoist shutdown upon failure of one wire rope. Also, the auxiliary hoist does not have a load cell.

The auxiliary hoist system includes the following features from the Ederer EDR-1 Topical Licensing Report: 1) energy absorbing torque limiter (EATL), 2) emergency drum brake, 3) hoist/drive train failure detection system, 4) wire rope spooling monitoring, and 5) hoist drum safety supports.

Some features are either not provided or do not conform to Ederer’s Topical Licensing Report EDR-1 for the auxiliary hoist system because of the use of the two

part reeving configuration. Therefore, the replacement auxiliary hoist is not fully compliant with NUREG-0554. However, the safety features on the replacement auxiliary hoist system meet or exceed those of the original auxiliary hoist system, resulting in enhanced load handling capability.

12.2.2.5.3 Safety Evaluation

All drawings and calculations were submitted by the contractor to TVA for review and approval. These drawings and calculations covered the details of all components, except items of standard manufacture. Standard items, such as motors, gear reducers, sheaves, etc., were designed in accordance with recognized standards and proven concepts.

The ambient temperature under which the crane is to operate does not exceed 40°C. Stresses in all structural and mechanical parts are far below the endurance limits for infinite life of the various materials for both the rated crane capacity and the test load of 125 percent capacity. All loads to be handled are below rated crane capability. Therefore, stresses should never reach allowable working stresses. Loads on the structural parts will vary but will not reverse. The only critical parts with stress reversals are the rotating parts, and these are provided with single-failure protection. Since the crane is to operate under normal temperature conditions, and since the stress levels are below the endurance limits for infinite life, testing of the crane to 125 percent of rated capacity provides reasonable assurance that the crane will not fail while handling a Dry Cask Storage spent fuel cask.

The crane was designed to provide factors of safety for the structural portions of the crane when handling rated crane capacity of not less than 2 to 1 based on the yield strength, and not less than 3.88 to 1 based on the ultimate strength. The trolley was designed to provide factors of safety for the structural portions of the trolley when handling rated trolley capacity of not less than 1.67 to 1 based on the yield strength, and not less than 2.78 to 1 based on the ultimate strength. When the crane with no load is subjected to a 300 mph wind, stresses will not exceed 0.9 of the yield strength. For the machinery portions of the crane, the factor of safety when handling rated crane capacity is not less than 5 to 1 based on the ultimate strength. The main hoist cables have a safety factor as specified in EDR-1. The auxiliary hoist cables meet the requirements of NUREG-0554. For the bridge motor stall torque of 275 percent normal torque, stresses in machinery parts do not exceed 0.9 yield. Factors of safety for both the structural and the mechanical portions of the crane when handling the Dry Cask Storage spent fuel cask are somewhat higher than the listed factors of safety, since the estimated weight of the cask is less than the rated crane capacity.

The Reactor Building Crane is classified as Seismic Class II. During an earthquake, the intended function is that the crane bridge and the trolley could be displaced, but they will not leave the runway beams, and any lifted load will be safely retained. The

bridge rails are firmly attached to the supporting steel superstructure, and the trolley rails are firmly attached to the bridge girders. As noted in Section 12.2.2.8.3, the seismic analysis of the crane, replacement trolley, and steel superstructure indicates that no uplift or unacceptable horizontal displacement of any wheel or of the bridge and trolley structure occurs in a seismic event.

During a tornado, tie-down devices will hold the trolley against the bumpers and wheel stops, and tie-down devices will hold the crane bridge against the bumpers and wheel stops or, alternately, the bridge will be restrained by slings tied to the superstructure rigid frames.

After installation of the replacement trolley, the total crane was tested by TVA, in accordance with Ederer’s procedures 250 and 251, to 100 and 125 percent of rated capacity (156.25 tons for the main hoist). The required tests and the test result forms were outlined by TVA design engineers. The ability of the crane to perform all its intended functions was demonstrated during these tests, and each brake was tested separately. The capability of the emergency drum brake to lower the retained load in a safe, controlled manner was also demonstrated.

Operational tests and visual inspections are to be made at periodic intervals during the life of the crane to demonstrate its ability to safely perform its intended functions and to maintain compliance with NUREG-0554. Inspection of critical welds will be performed at 4-year intervals as required by NUREG-0554..

12.2.2.5.4 Inspection and Testing

Sufficient inspections were made at the contractors' plants to assure that the crane was fabricated in accordance with the specification requirements and the contractors' drawings which were reviewed and approved by TVA design engineers.

General material requirements for various groups of items were shown in the TVA specifications. Specific materials used for all non-standard items were shown on the contractor's drawings. Certified mill test reports were received for all material except material used for items of standard manufacture. Test reports show the heat number for material, physical properties, and mechanical properties.

The crane bridge structure (and original trolley), except for ropes, was completely assembled in the shop; and the components were operated to assure the accuracy of fabrication and the quality of workmanship. The replacement trolley was also assembled in the shop; and the components were operated to assure the accuracy of fabrication and the quality of workmanship. The main and auxiliary hooks were tested to 200 percent of their rated capacity. Tests on the main hook were made with a load applied from both the center hole and the ears. After the load tests, the hooks were checked by magnetic particle inspection and for any change in dimensions.

12.2.2.6 Sacrificial Shield Wall

The sacrificial shield wall provides a biological shield for protection of personnel from gamma radiation from activated vessel components, a neutron shield to prevent activation of the drywell components during operation, and a means of supporting the drywell pipe hangers and access platform.

The sacrificial shield wall also provides protection against damage to the nuclear system process barrier due to seismic loading, and against further damage due to vessel pipe penetration rupture jet forces and a limit stop and support for pipe restraints in the event of a drywell pipe rupture.

The sacrificial shield (see Figure 12.2-23) consists of a 24-foot diameter circular cylinder attached to the vessel support pedestal and extending upward approximately 45 feet. This cylinder forms the outer shell of the annulus; the inner shell is formed by the reactor vessel and support skirt. The pedestal forms the base of the annulus with the top open to the drywell. The sacrificial shield wall is 27 inches thick and is constructed from 26-inch vertical WF beam columns, tied together by horizontal WF beams and 1/4-inch plates. These plates are welded to the column flanges, both inside and outside, thereby forming a double walled shell. This shell is filled with concrete to provide shielding capability. The pipes leaving the reactor vessel penetrate the shield at elevations below the top of the sacrificial shield. The penetrations in the vicinity of the reactor core utilize removable shield plugs which fit around the pipe. The plugs are provided in order to allow access to the pipe welds for purposes of inservice inspections. Due to excessive personnel dose received during the removal and reinstallation of these plugs, it is acceptable to leave the plugs (Permali) removed. The dose consequences of the plug removal have been evaluated and are acceptable. These removable plugs are covered by two 9-inch thick steel doors attached to the shield wall by two vertical hinges, each 1-1/4-inch diameter; both halves are locked in place by a 1-1/2 by 1/2-inch locking pin.

This configuration was conservatively analyzed to determine the capability of the shield wall to withstand pressures generated in the annulus. The assumptions and criteria utilized to estimate beginning of shield wall failure are that: (1) only the two 1/4-inch plates, acting as a thin cylindrical shell, resist the pressure forces, taking no credit for beam or concrete strength; and failure commences when shear stress at the fillet weld, joining the plates to the column flanges, reaches 90 percent of shear yield; (2) shear yield is only 1.5 times code allowable which results in a failure commencement at approximately 24 ksi for the weld material, whereas shear ultimate is in the order of 40 ksi; and (3) the pressure differential is a constant load, whereas the pressure differential would actually decrease as the drywell pressurized. For these assumptions and criteria, the beginning of wall failure occurs

when the pressure in the annulus reaches 19 psi. This 19 psi is a differential pressure across the shield wall from the annulus to the drywell space.

Effects of postulated loss-of-coolant accidents occurring within the sacrificial shield area have been investigated. Pipes with nominal diameters of 4 inches or smaller are the only reactor coolant lines investigated, because the reactor vessel safe-end welds for these nozzles are located within the sacrificial shield area. The minimum wall thickness for the various piping systems occurs at the safe-end joint to the piping. All other sections from this joint back to the reactor vessel have thicker wall sections and, therefore, have lower stresses. The largest line which has the safe end located in the annulus is the 4-inch jet pump instrument line nozzle. (For all larger lines, the double-ended line break results in the flow being directed into the drywell volume and not into the annulus.) The rupture of this 4-inch line results in a pressure differential less than 2 psi (pre-uprated) and approximately 2.4 psi (uprated) across the sacrificial shield wall, which is considerably less than the capability of the shield wall. The uprated result is based upon calculations performed with the SAFER/GESTR-LOCA outputs at 105% power uprate operation. The pressure load on the reactor vessel and the support skirt are far less than their capability.

The effects of this 4-inch rupture in terms of missile generation were also investigated. The 2 psi (pre-uprated)/2.4 psi (uprated) pressure differential will not generate a missile, because removable plugs are retained by hinged and locked doors as described above. Forces are not large enough to fail these attachments.

No valves are located inside the sacrificial shield; therefore, valves are not potential missiles within the sacrificial shield area.

An analysis has also been performed of the effects of jet forces resulting from a double-ended break of the 4-inch line, assuming the jet forces from the break were to impinge directly on the removable plug. The resulting load would be 11 kips, which is less than the capability of the locking bars and hinges, the capability of the shield wall, and the capability of the reactor vessel and its support skirt.

With one exception, the piping within the sacrificial shield area is routed from the reactor nozzle directly through the steel shielding doors and outside the sacrificial shield. One 2-inch line has a tee with pipe runs of a few inches. Due to the location of the reactor nozzle safe-end welds and the short runs of small pipe, pipe whip is not a problem.

The sacrificial shield wall consists of 12 steel columns equally spaced around the circumference. The outside surface of the wall is formed by 1/4-inch-thick steel liner plate welded to the outboard flange of the columns. The inside surface of the wall is formed by 1/4-inch-thick steel liner plate welded to the outboard face of the inboard column flange.

A concrete fill is placed between these liner plates to act only as a biological shield. The concrete is assumed to have no structural purpose, except for the lower 10 feet 6 inches of the wall. A heavier density shielding concrete fill, having a density of 215 pcf, is used from one foot below the active core area to 10 inches above the active core area.

There is a removable section of biological shielding at each reactor pressure vessel penetration to facilitate inspection of the penetration and two removable sections near the base of the wall for inspection opening access at the base of the reactor pressure vessel. There are steel beams framing into the column webs to support pipe hangers and access platform floor beams. The sacrificial shield wall is anchored and transfers loading acting on it to the reactor concrete pedestal by the column base anchor bolts and shear bars. The reactor pressure vessel is laterally supported by stabilizers between the vessel and the shield wall and at El. 625 feet 3 inches. There is a pipe truss between the shield wall and Reactor Building to laterally support the shield wall at El. 624 feet 8 inches.

The sacrificial shield wall is designed, without considering the concrete for any structural purpose, to withstand seismic forces, and to support normal pipe loading and pipe restraint loads in the event of a pipe rupture in the drywell. The design seismic loads, moments, and shears are as given in paragraph 12.2.2.8.2.

The jet load used in the design is the worst condition of either a clean break of the 26-inch main steam reactor pressure vessel penetration resulting in a jet reaction force of 595 kips, or a clean break of the 28-inch recirculating loop outlet penetration resulting in a jet reaction force of 658 kips.

The American Institute of Steel Construction (AISC) Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted April 17, 1963, is used in the design of the steel in the sacrificial shield wall for all the forces on the structure, including the design seismic and jet loads, without any increase in the allowable stresses due to short duration loads. For either structural re-design or re-evaluation of the sacrificial shield wall steel, the 8th Edition of the AISC Specification may be used. The method used to design reinforcing is given in paragraph 12.2.2.2.4. The method applied to crack control is to limit reinforcing stresses to that given in paragraph 12.2.2.2.3. Shearing stresses in the wall do not exceed that permitted by equation 12-4 in the ACI code; therefore, no shear reinforcing is required. However, a design is made of the steel under all loading conditions, including jet load plus a hypothetical double seismic loading using the allowable tension and bending stresses as 90 percent of yield stress and the shear as 60 percent of yield stress. These are 1.5 times the corresponding AISC stresses of 60 percent and 40 percent of yield stress used under the design seismic plus jet plus dead loading. This stress criterion is considered desirable for this application and adequate for the 'once-in-a-lifetime' loading condition.

The stresses in the steel due to seismic and jet loading acting in conjunction with dead load of the shield wall, pipe, and platform loads do not exceed the applicable AISC allowable stresses, as described above, for ASTM A-36 steel having a yield stress of 36,000 psi.

The resulting forces used in the design of pipe restraint support steel for recirculating loop are 666 kips for 28-inch pipe, 393 kips for 22-inch pipe, and 127 kips for 12-inch pipe. The stresses in the pipe restraint support steel due to these loadings, in conjunction with other applicable loads, do not exceed the 1.5 times the allowable AISC stresses of 60 percent of yield stress for tension and bending stress nor 40 percent of yield stress for shear stress.

12.2.2.7 Structural Steel Inside Drywell

12.2.2.7.1 Drywell Access Platforms

Description

The drywell access platforms include two main platforms, one at El. 584 feet 11 inches, and one at El. 563 feet 2 inches. The flooring is standard grating, with 1-1/2-inch X 3/16-inch load bars. The grating and support steel extends from the reactor pedestal to the drywell shell at El. 563 feet 2 inches and from the sacrificial shield wall to the drywell shell at El. 584 feet 11 inches.

The platforms are supported by 24-inch-deep, wide-flange beams radiating from the reactor pedestal and sacrificial shield wall to the drywell shell. The radial support beams for El. 584 feet 11 inches are field-welded to header beams framed to the columns of the sacrificial shield wall.

The radial support beams for El. 563 feet 2 inches are field-bolted to embedded plates in the outside face of the reactor pedestal. All beams are supported by beam seats welded to the drywell shell. Lubrite pads under the beams allow drywell shell expansion. Shear bars welded to the bottom flange of the radial beams on both sides of the beam seat prevent lateral movement of the beams. Intermediate grating support beams at 6 feet 6 inches maximum spacing are framed between the radial beams.

All support beams were designed to meet American Institute of Steel Construction (AISC) specification requirements. All structural material used for the Unit 2 and Unit 3 platforms is ASTM A-36 with high-strength bolts, ASTM A-325, and E70XX welding electrodes. The structural material used for the Unit 1 platforms is ASTM A-36, ASTM A-500 Grade B, ASTM A-572, or ASTM A-992 with high-strength bolts, ASTM A-325 or ASTM A-490, and E70XX welding electrodes.

Stresses were held to less than the allowable given in Table 12.2-16.

In Table 12.2-16, the term "dead loads" (D.L.), includes the weight of the grating and beams only. Seismic load factors were applied to both dead and live loads.

Tie-down columns have been provided from El. 549 feet 11 inches to the El. 563 feet 2 inches platform radial beams. These columns prevent the beams from lifting off the beam seats of the shell when there is uplift acting on the beam. Plans, sections, and details of the platform are shown in Figures 12.2-24 sheets 1, 2, and 3 and 12.2-25 sheets 1, 2, and 3.

12.2.2.7.2 <u>Support Steel for Pipe Guides Inside Drywell</u>

The pipe guides included in this paragraph are for the Main Steam and Feedwater Systems. The guides protect the nozzles, attached to the drywell, after a rupture of the pipe inside the drywell.

The support steel for the pipe guides is located approximately 23.4 feet from the centerline of the reactor on the 180° azimuth. It furnishes a "base" on which to set the four main steam and two feedwater guides between El. 563 and elevation 584. The "base" is a wide flange beam supported by two columns anchored to the concrete at El. 549.92.

The load applied to the guide and support is an impact load resulting from rupture of the pipe inside the drywell. Each guide and support is designed individually for the load from the pipe it restrains. This design load is the product of the operating pressure and the cross-sectional area of the inside of the pipe.

All structural material used is ASTM A-36, with ASTM A-307 bolts and E70 series welding electrodes. The brackets attached to the pipes are stainless steel. For design, the columns and beams are assumed pin-connected and the brackets on the columns are assumed fixed.

The stresses in all components of the support are less than 90 percent of yield for tension and bending and 52 percent of yield for shear for all loading except postulated pipe breaks. For loads resulting from the postulated pipe break accident conditions, the support will behave inelastically resulting in a load path change to the concrete floor at Elevation 549'-11" instead of loading the drywell floor steel at Elevation 563'-0 1/2".

The lateral thrust from the pipe is first transferred to the "base" and thence to bracing members welded to embedded plates in the pedestal.

This support is designed for a load of 520 kips acting in any direction perpendicular to the axis of the pipe.

12.2.2.7.3 Drywell Temperature Effect

The containment drywell vessel is capable of withstanding combinations of pressure-temperature transients other than the specified values of 56 psig and 281°F. The vessel's capability for withstanding higher temperatures and lower pressures is largely dependent on vessel expansion, because the allowable design stress intensities are not affected for the vessel material in the 20°F to 650°F temperature range in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class B Vessels, 1968 Edition.

The drywell expands both radially and vertically with increases in vessel temperature and compresses the expansion gap material between the vessel and the Reactor Building concrete. This compression of the resilient material (2-inch minimum thickness) develops an external loading that tends to buckle the shell, and the worst loading condition occurs with an internal pressure of zero psig, which was assumed in the analysis. The magnitudes of the expansions perpendicular to the vessel surface for three points on the vessel were calculated for a 340°F shell temperature. The 1.169-inch vessel expansion (point D, just below the "neck" of the vessel) produces a corresponding maximum deformation of the resilient material. This deformation is resolved into an equivalent 1.1 psi external load on the vessel. The critical buckling load on the vessel at this point is 7.1 psi as calculated by the rules of the ASME Boiler and Pressure Vessel Code, Section III, Class B, 1968 Edition. This 1.1 psi load is substantially less than the critical load, and, in fact, it is less than the 2 psi design value used for the vessel; therefore, buckling of the drywell vessel will not occur.

Although the containment temperatures will be limited, an evaluation was conducted into the behavior of the containment at higher temperatures. An atmospheric temperature of 340°F, the maximum which the drywell could possibly achieve from steam leaks, is associated with partial reactor depressurization and throttling from about 500 psia into the containment at 45 psia. To be conservative, 45 psia was assumed as the containment pressure, and the very short-term transient effects of large breaks were neglected. (See Chapter 14) Throttling from rated reactor pressure will superheat the containment atmosphere to about 320°F, whereas throttling from lower reactor pressures (<300 psia) will superheat the atmosphere to about 330°F. Thus, an upper limit of 340°F was used to evaluate the drywell vessel.

The containment stress analysis report was reviewed, and in no case did the effects of an increase of temperature from 281°F to 340°F result in calculated stresses higher than code allowable stress intensity values.

Jets that might impinge on the structure could produce local thermal effects wherein the local temperature would exceed the wall temperatures. The high conductivity in the drywell wall will relax the temperatures in the small affected area so that the

temperatures are only slightly above the wall temperatures. The thermal stresses associated with these slightly elevated temperatures will be correspondingly small. Excessive stresses will be self-limiting by yielding of the material, but could lead to thermal ratcheting. However, thermal ratcheting from jets will not constitute a serious problem because of the anticipated small number of incidents expected in the life of the vessel.

The electrical penetrations have been designed, tested, and installed to operate when required in their locations during normal and accident conditions as specified by the Browns Ferry 10 CFR 50.49 program.

The piping penetrations were investigated by conservatively analyzing the penetrations with the largest temperature movements in the upper part of the vessel and at the equator. The peak stresses in these penetrations were determined to be less than the stresses at the design conditions of 281°F and 56 psig. From this conservative examination, it was deduced that the stresses in the piping penetrations would be less than the allowable stress intensities for all penetrations on the vessel at a temperature of 340°F and a pressure of 30 psig. The vessel movements are definitely less than the constructional clearance built in between the penetrations and concrete pipe sleeves. Therefore, the piping penetrations in the vessel will not be a limiting factor at a condition of 340°F and 30 psig.

The safety components inside the drywell that must function during normal and accident conditions have been designed, tested, and installed to operate in their locations as specified by the Browns Ferry 10 CFR 50.49 Program.

12.2.2.8 Dynamic Earthquake Analysis

12.2.2.8.1 Reactor Building Structure

Three mathematical models are used in the dynamic earthquake analyses of the reactor building. The analyses of the reactor building, outside the drywell containment (enclosure structure) are described in this section; the reactor building analyses inside the drywell containment are described in 12.2.2.8.2; and the reactor building steel superstructure analyses are given in 12.2.2.8.3.

A time interval of integration of 0.005 seconds is used in these analyses and in the seismic analyses of all other seismic class I structures. Separate analyses are performed for the north-south east-west and vertical directions. Coupling effects between the horizontal and vertical responses are considered only for soil-supported structures. Amplified response spectra (ARS) are generated at the center of mass of the seismic Class I structural models.

For the design of the reactor building enclosure structure and subsystems located in the reactor building outside the drywell containment, analyses are performed using

the one branch, lumped mass model consisting of eight masses lumped at the five floors, the elevation of the pressure suppression chamber, the crane rail and the roof as shown in figure 12.2-26. The model is considered fixed at the base since the reactor building is founded on sound rock. The model element properties are calculated considering shear and bending deformations. Floor slabs are assumed to provide rigid inplane diaphram actions; out of plane flexibilities are not considered. The fixed-base model frequencies and modal masses for modes up to the frequency limit of 20 cps are summarized in Table 12.2-16.4, for each horizontal and the vertical response directions. A minimum of 90 percent of the total mass is included in the cumulative modal mass for each response direction. For the vertical direction, the modal properties for modes with frequencies beyond 20 cps up to mode giving a cumulative modal mass of approximately 90% of the total mass, are also shown in the table. Five (5) percent of critical damping is used for all modes of vibration of this fixed based structure for both the OBE and DBE.

Two separate modal superposition time history analyses were performed using the model shown in figure 12.2-26.

a) Designs using the El Centro earthquake for the input ground motion (Refer to 2.5.4).

   A minimum of the first six seconds of the El Centro earthquake acceleration time history is used in the analyses. Building responses (shear forces, bending moments, axial forces and ARS) resulting from these analyses are used in the design of the concrete structure and subsystems housed in the enclosure structure. Since the lumped mass model is uncoupled in the horizontal directions, there is no calculated torsional response. To account for possible torsional moments, an eccentricity equal to the distance between the shear center and mass center at each elevation is calculated. This eccentricity is compared to 5 percent of the largest plan dimension of the building, and the larger of these two values is used to calculate a torsional moment. This moment is used along with the other calculated forces and moments to design the shear walls.

   Concrete floors which are integrally connected to the reactor building are designed for the maximum vertical acceleration of the structure at the elevation where the floor is located if they have a fundamental natural frequency greater than 20 Hz. If they have a natural frequency less than 20 Hz, they are evaluated using the ARS derived from the calculated EL Centro based acceleration time history at that elevation.

   ARS are generated at periods varying from 0.05 seconds to 20 seconds in increments of 1 radian per second of circular frequency. The ARS are not peak broadened.

b) Alternate design basis using the artificial earthquake time history for the input ground motion (refer to 2.5.4).

The results of these analyses are building accelerations, displacements and ARS and are applicable for use in analyses and design of subsystems located outside the drywell containment.

A minimum of 24 seconds of the artificial earthquake acceleration time history is used in the analyses. The ARS are generated at periods varying from 0.05 seconds to 2.0 seconds in increments of 1 radian per second of circular frequency. The ARS are peak broadened ± 10 percent.

12.2.2.8.2 Reactor Pressure Vessel - Shield Wall - Pedestal and Reactor Building Coupled System

A multi-branch, lumped mass, model with branches representing the drywell containment, sacrificial shield wall and pedestal, reactor pressure vessel (RPV) and RPV intervals coupled with the enclosure structure is used in the dynamic earthquake analyses of the reactor building inside the drywell containment. The model is shown in figure 12.2-27A, 27B, and 27C. The branches representing the various structural elements are discretely modeled and interconnected at the appropriate points such as the star truss, stabilizers and control rod drive (CRD) housing lateral restraints. The drywell containment is embedded in concrete at elevation 547.089 feet and braced in the horizontal direction at elevation 624.667 feet. The damping values used for the various elements of the model are given in table 12.2-17.

Two separate modal superposition time history analyses are performed. The fixed-base model frequencies, modal masses, and the composite modal damping ratios computed using the strain energy method for modes up to the frequency limit of 20 cps are summarized in Table 12.2-17.1A, 17.1B, and 17.1C respectively, for each of the horizontal NS and EW, and the vertical response directions. A minimum of 90 percent of the total mass is included in the cumulative modal mass for each response direction. For the vertical direction the modal properties for modes beyond 20 cps up to the mode giving a minimum of 90% of the total mass, are also shown in the table.

a) Designs using the El Centro Earthquake for the Input Ground Motion (refer to 2.5.4).

A minimum of the first six seconds of the EL Centro earthquake acceleration ground motion is used in the analyses. The results of the analyses are used for qualification of the RPV and RPV internals, drywell containment, shield wall and pedestal, structural elements connecting the branches and subsystems located inside or attached to the drywell containment.

Amplified Response Spectra (ARS) are generated at frequencies varying from 0.2 Hz to 20 Hz at intervals consistent with those given in the Standard Review Plan (SRP). The ARS are not peak broadened.

b) Alternate design basis using the artificial earthquake time history for the input ground motion (refer to 2.5.4).
The results of these analyses are building accelerations, displacements, and ARS and are applicable for use in the analyses and design of subsystems located inside or attached to the drywell containment.

A minimum of 24 seconds of artificial earthquake acceleration time history is used. The ARS are generated at frequencies varying from 0.2 Hz to 20 Hz at intervals consistent with those given in the SRP. The ARS are peak broadened ± l0%.

12.2.2.8.3 Steel Structure above EL 664.0

The steel superstructure is modeled as a three-dimensional frame structure and coupled with the one branch reactor building enclosure model shown in Figure 12.2-26. The superstructure model replaces masses and elements 1 and 2 of the reactor building enclosure structure model. The 125 ton reactor building crane is discretely modeled and coupled with the model of the superstructure. The seismic analysis of the crane and superstructure was updated to reflect the replacement trolley discussed in Section 12.2.2.5, to consider the effects of 100 percent of rated load for the crane, and to extend the model to include pendulum effects due to the suspended load. The analyses are similar to the reactor building analysis described in 12.2.2.8.1. The scope of the reanalysis described above was to provide structural loads to reconfirm the adequacy of the crane and steel superstructure.

Two dynamic analyses using the time history model superposition method are performed.

a) Designs using the El Centro earthquake for the input ground motion (refer to 2.5.4).

A minimum of the first six seconds of the EL Centro earthquake acceleration time history is used in the analyses. The results of the analyses are used for qualification of the reactor building steel superstructure above elevation 664.0 and subsystems supported by this structure.

Amplified response spectra (ARS) are generated at periods varying from 0.05 seconds to 2 seconds in increments of 1 radian per second of circular frequency. The ARS are not peak broadened.

b) Alternate design bases using the artificial earthquake time history input ground motion (refer to 2.5.4).

The results from these analyses are structural accelerations, displacements, and ARS and are applicable for use in analyses and design of subsystems supported by the steel superstructure.

A minimum of 24 seconds of the artificial earthquake acceleration time history is used in the analyses. The ARS are generated at periods varying from 0.05 seconds to 2.0 seconds in increments of 1 radian per second of circular frequency. The ARS are peak broadened ± 10 percent.

12.2.2.9 Wind Load

12.2.2.9.1 Pressure Magnitude and Distribution

The magnitude and distribution of wind pressures, both for the 100 mph wind and the 300 mph tornado wind, are determined by following the recommendations of ASCE Paper No. 3269.[4]

The dynamic pressure is determined by

$q = 0.002558V^2$
q = dynamic pressure, psf
V = wind velocity, mph

The pressure for various parts of the building surface is determined by use of shape coefficients, the selection of which is guided by those given in Table 4(a) of Paper No. 3269. The product of the shape coefficient and the dynamic pressure gives the design pressure for the various parts of the building. The total force in any direction is obtained by summing the pressures on appropriate surfaces.

In determining shape coefficients, various structures given in Table 4(a) are compared, since the Reactor Building dimensional ratios are not exactly as those tabulated. To compare the effect of the ratio of width to length, the following is taken from Table 4(a), using h = height, b = width and L = length with wind perpendicular to structure.

[4] Task Committee on Wind Forces, "Wind Forces on Structures," Paper No. 3269, Transactions, American Society of Civil Engineers, Vol. 126, Part II, 1961, pp. 1149-1167.

Structure	External Pressure Coefficient Windward Side	Leeward Side	Parallel to Wind
House - Roofs 0 to 10 degrees h:b:L = 1:1:1	0.9	-0.5	-0.6
House - Roofs 0 to 10 degrees h:b:L = 2.5:2:5	0.9	-0.5	-0.7

Comparison of the above shows that, with the height and width approximately equal, an increase in length has no effect on the windward and leeward coefficients but does affect the coefficient on the side parallel to the wind (short side).

To compare the effect of roof slope, the following is taken from Table 4(a) using h, b and L as above.

Roof Slope	External Pressure Coefficient Windward Side	Leeward Side	Parallel to Wind
Closed hall - Roofs 0 to 3 degrees h:b:L = 1:4:4	0.9	-0.3	-0.4
Closed hall - Roofs 30 degrees h:b:L = 1:8:16	0.8	-0.5	-0.5

Comparison of this tabulation shows that the coefficient on the leeward side for buildings with flat roofs is less than for pitched roofs.

The Browns Ferry Reactor Building has a ratio for h:b:L = 1:1:4. From the preceding tabulation, it is obvious that the pressure coefficient on the windward side should be 0.9. The pressure coefficient on the leeward side should lie somewhere between -0.3 and -0.5. The Reactor Building h:b:L ratio approximates that tabulated above for a house with h:b:L of 2.5:2:5 and a roof 0 to 10. It has been shown above that for a flat roof, the leeward coefficient is less than for a pitched roof. Since the reactor building does have a flat roof, a selection of -0.4 for the pressure coefficient on the leeward side seems justified. For the sides parallel to the wind, it is logical to select the most conservative of the above tabulated coefficients.

In summary, the pressure coefficients for design are as follows:

Windward Side $\quad$ Cpe = 0.9

Leeward Side	Cpe = -0.4
Parallel to Wind	Cpe = -0.7

The pressure on the walls is computed by using the equation:

$$P = (Cpe)(q)$$

P = pressure, lb per sq ft
Cpe = shape coefficient
q = dynamic pressure, lb per sq ft

The resulting design pressures in pounds per sq ft are as follows:

Wind Condition	Windward Side	Leeward Side	Parallel to Wind
100 mph Wind	23	-10	-18
300 mph Tornado Wind	206	-91	-160

In addition to the pressures described above, consideration is given to the pressures resulting from the depressurization effect when the Reactor Building is enveloped by the eye of a tornado. The method of analysis and determination of loadings are as given in Appendix 2.1A, pp. 2.1-5 through 2.1-22 of Amendment 2 to the Unit 3 Design and Analysis Report.

12.2.2.9.2 Tornado Generated Missiles

The potential missiles that could be generated at any point, with respect to the Reactor Building, and which serve as the design basis missiles are:

a. A 2-inch x 4-inch x 12-foot board weighing 40 pounds/cu ft, end on;

b. A cross-tie, 7 inches x 9 inches x 8-1/2 feet weighing 50 pounds/cu ft, end on;

c. A compact car weighing 1800 pounds with an impact area of 20 square feet;

d. Pieces of concrete 6-1/2 inches x 12 inches x 2 inches thick, end on, as a result of the spalling effect from the concrete chimney during postulated failure of chimney from tornado winds; and

e. Aircraft warning beacon from chimney.

As an upper limit, each missile is assumed to be traveling 300 mph at impact. No credit is taken for the crushing effect of missiles. The depth to which these missiles will penetrate the exterior concrete walls of the Reactor Building is calculated by the

modified Petry formula as given in report by Amirikian.[5] The results are given in Figures 12.2-40 and 12.2-41. Maximum penetration is 9 inches (one half of wall thickness). According to Moore[6] spalling on the inside face will not occur for penetration less than two-thirds the wall thickness. The reactor building walls can therefore adequately resist the spectrum of postulated missiles.

12.2.2.10 Lateral Earth Pressures

Static earth pressures are determined by means of Coulomb's "wedge of pressure" theory. The angle of internal friction (ϕ) is taken as 32°, the weight of moist earth is 120 pounds per cubic foot, and the weight of submerged earth is 65 pounds per cubic foot. Where a fill is completely or partially saturated, the static water pressure below the saturation line is computed as if no fill were present. To this pressure is added static earth pressure computed by the Coulomb theory using the buoyant weight of the fill material.

As a result of seismic motions, the lateral soil pressure against structures founded on rock will be greater than the static soil pressure. The magnitude of this increase has been determined by shaking table experiments performed for the design of TVA's Kentucky hydro project. For a ground acceleration of 0.1g, the static soil pressure is increased by 23 percent for a moist fill and 11 percent for a saturated fill. This incremental increase is combined with the static pressure as a triangle of pressure whose apex is at the rock surface and maximum ordinate is at the ground surface. In addition to the soil pressure increase, as described above for a saturated fill, the static pressure of water within the fill is increased 11 percent. This incremental increase is combined with the static water pressure as a triangle of pressure whose apex is at the water surface and maximum ordinate is at the rock surface.

For a ground acceleration of 0.20g, the incremental pressure increase will be twice that given above.

12.2.2.11 Watertight Personnel and Equipment Access Lock Doors

12.2.2.11.1 Description

The doors are watertight closures for the north openings in the Reactor Building personnel and equipment access locks and are identified as follows:

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5 Amirikian, A., "Design of Protective Structures," NP-3726, Bureau of Yards and Docks, Department of the Navy, Washington, D.C., August 1950.

6 Moore, C. V., "The Design of Barricades for Hazardous Pressure Systems," Nuclear Engineering and Design, Vol. 5, 1967, pp. 85-86.

One door, 3 feet 8-1/4 inches x 7 feet 1-3/8 inches, for the north opening to Unit 2 personnel access lock located immediately east of column line R8 along column line n; see Figure 1.6-6.

One door, 3 feet 8-1/4 inches x 7 feet 1-3/8 inches, for the north opening to Unit 3 personnel access lock located immediately east of column line R15 along column line n; see Figure 1.6-13.

One double-leaf door for 8 feet 4-3/4 inches wide x 8 feet 8-3/8 inches high opening to Unit 2 equipment access lock located immediately east of column line R9 along column line n; see Figure 1.6-6.

One double-leaf door for 8 feet 4-3/4 inches wide x 8 feet 8-3/8 inches high opening to Unit 3 equipment access lock located immediately east of column line R16 along column line n; see Figure 1.6-13.

The single-leaf personnel access doors are the sealed, hinged, manually-operated, welded steel type. The doors are provided with electrical interlocks and are normally closed and latched by dogs to provide watertight units. The dogs are operated by a single mechanism which is actuated by handwheels on both sides of the doors. The double-leaf equipment access doors are the sealed, hinged, manually-operated, welded steel type. Each leaf is provided with electrical interlocks. The active and inactive leaf of each door is normally closed and latched by dogs to provide a watertight unit. The dogs on each leaf are operated by a single mechanism. The dogging mechanism is actuated by handwheels on both sides of the active leaf and from lock side only of the inactive leaf.

Load combinations used in designing the doors with corresponding allowable stresses are listed in Table 12.2-38. For all combinations, the resulting stresses do not exceed the allowable. Structural portions of the double-leaf door and frame assemblies are fabricated from ASTM A-36 steel, while structural portions of the single-leaf door and frame assemblies are fabricated from ASTM A276-304 stainless steel.

12.2.2.11.2 Safety Evaluation

The doors for the two personnel access locks and the two equipment access locks seal the openings and provide adequate flood protection for the probable maximum flood. The doors are designed to withstand static water pressure to El. 572.5 feet and the Design Basis Earthquake (0.20g). The doors will remain intact and retain their seal after the occurrence of either of these conditions, or any combination of conditions as listed in Table 12.2-38.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel

fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction. Certified mill tests, covering chemical analyses and physical properties, are on file for all materials.

12.2.2.11.3 Inspection and Testing

After initial installation, each door was tested for operation of hinges and dogging mechanism. Thereafter, all parts of the doors are to be inspected periodically. Doors and dogging mechanisms are to be inspected for free operation. Seals are to be visually inspected to see if any cracks or blemishes have developed, and the surface of the skin plate is to be inspected for paint deterioration.

12.2.2.12 Masonry Wall Design

Concrete masonry walls in the Reactor Building, as well as all other category I structures, have been evaluated in accordance with NRC OIE Bulletin 80-11, "Masonry Wall Design." Each masonry wall in the category I structure was identified and located by a drawing survey and then verified by a plant survey. All postulated loads required by NRC OIE Bulletin 80-11 were identified and the walls evaluated for the effects upon safety-related equipment. The design reevaluation criterion was developed using the allowable stresses of ACI Code 531-79 and the alternate design method of ACI 318-77 as its basis. The results of this investigation were reported by letter from L. M. Mills, Manager, Nuclear Regulation and Safety, TVA, to J. P. O'Reilly, Office of Inspection and Enforcement, NRC Region II, entitled "Office of Inspection and Enforcement," Bulletin 80-11-RII:JPO 50-259, -260, -296 Browns Ferry Nuclear Plant, dated October 1, 1981.

The results of the evaluation showed that masonry walls may be subjected to loads due to tornado depressurization, seismic events, and pipe breaks. The analysis resulted in the requirement of some additional structural steel restraints to ensure the adequacy of the walls whose failure could jeopardize safety-related equipment.

These restraints consist of structural steel shapes spanning the block walls either horizontally or vertically. The restraints are to be anchored into the surrounding concrete walls (if restraint spans horizontally), or the floor and ceiling (if restraint spans vertically). The analysis and addition of restraints assure that masonry walls do not adversely affect Class I features.

12.2.3 Turbine Building (Class II)

12.2.3.1 Concrete Structure (Figures 12.2-42 through 12.2-49)

12.2.3.1.1 Columns, Beams, and Slabs

The building below the operating floor, El. 617, is a reinforced concrete framed structure supported on steel H-piles to bedrock. Piles are spaced far enough apart within each cluster to ensure that the maximum average unit bearing stress on the rock area is limited to 500 psi. Stresses in the piles are limited to one third of the yield stress. See Subsection 2.5 for rock foundation and foundation treatment.

During installation of piling, drilling logs are maintained for recording the number of blows per inch of penetration. This record provides assurance that the piles are driven to the specified refusal criteria shown on design drawings.

The reinforced concrete building columns are assumed pinned at the bottom where they join the pile caps. Full continuity is assumed in the remainder of each building frame. A unique aspect of this building is its variation in concrete structural sections from bay-to-bay. Due to different shielding requirements for each area, floor thicknesses vary radically. In some bays, shield walls stiffen the frame, while in other bays only beams are required with the resulting decrease in frame stiffness.

The design moments and shears for the concrete frames are determined by use of a computer program. The design moments and shears for the slabs are determined either by the coefficients in Appendix A of ACI Code 318-63, or by moment distribution methods.

Using the moments and shears, as calculated above, the beams and slabs are designed by the working stress method using allowable stresses as given in Table 12.2-23. The columns are designed by the working stress method and checked by the ultimate strength design method using a load factor of 1.8. Applicable factors are as set forth in ACI Code 318-63.

12.2.3.1.2 Earth Retaining Walls

Where walls must retain lateral earth pressure, horizontal reactions are provided by the building frames. Horizontal reactions at the bottom of the walls are provided by either the basement floor slab, or by batter piles.

Lateral earth pressures are calculated using the Coulomb theory and the following values.

Angle of internal friction	ϕ = 32 degrees
Angle of friction between fill and structure	δ = 16 degrees
Moist Unit weight of fill	w = 120 pcf
Surcharge	200 psf

12.2.3.1.3 Main Steamline Enclosure

Within the Turbine Building, between the high-pressure turbine and the Reactor Building, the main steamlines are enclosed in special compartments with shield walls approximately 4 feet 6 inches thick.
These compartments and the adjoining area between columns m and j are investigated for compliance with the earthquake regulations of Section 2314 of the Uniform Building Code. Pile caps in this area are interconnected by ties, each of which can carry, by tension or compression, a horizontal force equal to 10 percent of the larger pile cap loading.

The 4-foot-6-inch-thick walls of each unit steamline compartment are designed as shear walls to protect the main steamlines in the event of earthquake forces of intensity assumed in zone 1 of the Seismic Probability Map of the above code. This structural system is assumed a box system as defined in Section 2314(b) of the Uniform Building Code. The value of "K" used is 1.33.

12.2.3.1.4 Turbine Foundation

A one-inch expansion joint separates the turbine foundation from the building frame, above the basement floor.

The foundation design is based on the General Electric Co. booklet, Steam Turbine-Generator Foundations, GET-1749B, except as noted below. The principal design cases and allowable stresses are as given in Table 12.2-24.

For live and dead loads combined with temperature change, allowable stresses specified in Chapter 10 of the 1963 ACI Building Code are used. In addition, the allowable value of shear is 120 psi with shear reinforcing, and 25 psi without shear reinforcing. The effect of temperature is based on a change of 30°F in the top longitudinal beams. Minimum reinforcement is 0.75 percent of the cross-sectional area for beams and columns and 0.50 percent for bearing walls and slabs.

Although the turbine-generator foundation is not specifically designed for earthquake loading, a horizontal force equal to 25 percent of the machine weight is applied at the shaft centerline in both the longitudinal direction and the transverse direction. This design case includes foundation dead load, machine load, and floor live load. Stresses for this case are the normal allowable stresses given in Chapter 10 of the 1963 ACI Code.

12.2.3.2 Steel Superstructure

The Turbine Building, above the operating floor El. 617, is framed by transverse welded steel rigid frames spanning approximately 107 feet. An expansion joint is provided between a 2-bay frame for the first two units and a single-bay frame for Unit 3. These buildings house common services to all three units. For longitudinal

expansion, the superstructure is provided with joints by using double rows of frames spaced four feet apart. Assurance that the Turbine Building will not damage the Reactor Building is provided by the inherent strength of that part of the Turbine Building that is adjacent to the Reactor Building. As can be seen in FSAR Figures 12.2-42 through 12.2-44, the Turbine Building is constructed in essentially two parts, separated by an expansion joint. The part adjacent to the Reactor Building consists of floor slabs 18 to 48 inches thick which act as diaphragms. Walls, 48 to 54 inches thick, support the floor slabs and act as shear walls. It can, therefore, be seen that the structure is inherently strong in resistance to horizontal loads. The steel frames which form the Turbine Building structure above the concrete structure are braced to provide rigidity in the direction of the Reactor Building. The frames provide support for the turbine cranes as well as the elaborate girt system which cantilevers eight feet from center of columns to support the metal siding. Frames are designed with fixed bases to resist lateral forces from the overhead cranes and wind-loads, in addition to supporting the vertical dead and live loads. A typical cross section of the superstructure showing the intermediate rigid frames is shown by Figures 12.2-50 and 12.2-51.

Design of the superstructure is based on 1963 American Institute of Steel Construction 'Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings.' For either structural re-design or re-evaluation, the 8th Edition of the AISC Specification may be used. All material conforms to ASTM A-36, except anchor rods which are ASTM A-307 steel. Shop connections are ASTM A-502 Gr. 1 rivets or welded, and field connections ASTM A-325 high-strength bolts.

12.2.3.3 Cranes

Each unit is provided with an overhead, electric, single-trolley, traveling crane. The main hoist of the Unit 3 crane is rated at 180 tons and the auxiliary hoists at 25 tons. The Units 1 and 2 main hoist is rated at 210 tons and the auxiliary hoist at 25 tons. The cranes are designed as Class II equipment.

12.2.4 Reinforced Concrete Chimney (Class I) (Figure 12.2-52)

12.2.4.1 Shell

The design cases and allowable stresses are as given in Table 12.2-25. A discussion of each case follows.

Case 1 - 100-mph Wind

This case is treated in accordance with the ACI Chimney Code (ACI 307-69), and allowable stresses are those specified in that revision and also shown in Table 12.2-25. This case does not control the design of any section in the chimney.

Case 2 - Tornado

This is the controlling design case for most of the height of the chimney. The tornado moments are those caused by a 300-mph wind. Since the wind pressure is proportional to the square of the velocity, these moments are nine times the moments for a 100-mph wind. It is not practical to design the chimney for these forces by the working stress method and to limit stresses in the extreme reinforcing bars to yield stress. The chimney is 600 feet high and is located south of the Class I Off-Gas Treatment Building and approximately 365 feet from the Class I Diesel Generator and Standby Gas Treatment Buildings. In order to ensure that the chimney does not fall on these buildings in the event of a tornado, the top 320 feet of the chimney are designed to fall well before the lower 280 feet reach their ultimate load capacity.

The following design approach is used.

a. Calculate moments, Mw, for a 300-mph wind on entire chimney.

b. Design the bottom 280 feet of the chimney for ultimate moments, Mu, equal to 1.1 times Mw. Calculations for Mu are based on the assumptions given in Section 1503 of the ACI Building Code (ACI 318-63). Design the upper 320 feet of the chimney for values of Mu equal to 0.55 Mw.

Calculations are made using a computer program in which the computer selects the reinforcement for any required moment capacity when the concrete and steel strengths, the dimensions of the section, and the direct load acting are given. The computer also determines if compression or tension controls the design. For this chimney, tension is the controlling factor at all sections.

The ultimate strength design method is not yet included in the ACI Chimney Code. However, the approach is well established for normal concrete structural components; and, when applied to a chimney, if proper load factors are used, the results are in line with results of a working stress analysis. Figure 12.2-53 shows the relation between tornado moments and the calculated ultimate moment capacity.

This design case resulted in very heavy reinforcing, generally two or three times that which would be expected in a chimney designed for conventional forces. Figure 12.2-54 shows the calculated steel areas and ratios used.

With the final reinforcing selected for the chimney, stresses are calculated for various wind velocities. These calculations support the ultimate strength approach. They show that the top portion of the chimney will fail at a wind velocity of less than 250 mph, while at that velocity the reinforcing in the lower portion will not have reached its yield stress, nor will the concrete in that region be overstressed. Figures

12.2-55 and 12.2-56 show steel and concrete stresses, respectively, as functions of wind velocities.

Case 3 - Earthquake

The idealized lumped mass model used to calculate structural earthquake responses of the concrete chimney, interior structure and off-gas stack is shown in figure 12.2-63. The interior structure includes concrete floor slabs at elevations 568.0, 580.5, 599.5 and a steel roof at elevation 665. The slab at elevation 568 is built monolithically with the chimney shell. The slabs at elevations 580.5 and 599.5 are separated from the chimney shell by 1-inch expansion joints filled with fiberglass. The steel roof is supported by the chimney. The off-gas stack is supported horizontally by a pinned connection at elevation 665.0 by the steel roof and vertically by the interior structure at elevation 599.5. The seismic model is fixed at the base since the foundation is founded on sound rock and anchored into the underlying rock as described in section 12.2.4.2.

Dynamic earthquake analyses were performed using the normal mode time history method. The analyses were performed using structural damping values of 3 percent for the concrete structures and 1 percent for the steel roof and off-gas stack system and composite modal damping. Analyses were performed using the El Centro earthquake and artificial earthquake time histories described in Section 2.5.4.

Results from the analyses using the EL Centro earthquake input ground motions are used in the design of the chimney shell, interior structure and off-gas stack. The earthquake loads are combined with other loads using the load combinations and allowable stresses given in Table 12.2-25. The load combinations with earthquake govern the design of the top portion of the chimney shell.

Results from the analysis using the artificial earthquake input ground motion are used in the analyses of subsystems located in the chimney. Amplified response spectra (ARS) were generated at frequencies from 0.2 Hz to 20 Hz at intervals consistent with the Standard Review Plan. The ARS were peak broadened $\pm$ 10 percent.

Case 4 - Resonant Wind

This analysis is based on a method described by Professors L. C. Maugh and W. S. Rumman in the September 1967 issue of the ACI Journal. The assumption is made that vortex shedding occurs and that the wind velocity causing vortex shedding is directly proportional to the diameter of the chimney. This particular chimney has unusual proportions. Its diameter varies from 6 feet at the top to 62 feet at the base. It does not appear logical to assume that vortex shedding could occur simultaneously over its full height. A conservative estimate appears to be to assume vortex shedding possible for its upper portion where a change in diameters is not as

pronounced as in the lower portion and then assume an average diameter for that portion to establish the critical wind speed and the resonant wind force. Two cases are considered.

a. Vortex shedding is assumed to occur from the top of the chimney down 300 feet. Assumed average diameter is 11.15 feet. Based on these assumptions, the calculated resonant wind moments are less than moments from a 100-mph static wind. The critical wind speeds are 17, 44, and 90 mph for resonance in the first, second, and third mode, respectively.

b. Vortex shedding is assumed to occur from the top of the chimney and down 420 feet. Assumed average diameter is 15.40 feet. With these assumptions, the calculated resonant wind moments are less than the earthquake moments caused by the El Centro 1940 records, when those records are normalized to 0.1g maximum ground acceleration. The critical wind speeds are 23, 59, and 122 mph for resonance in the first, second, and third mode, respectively.

Case 5 - Thermal Loading

The stack was originally designed to emit gases at ambient temperatures. Therefore, no thermal analysis was included in the original design. However, due to the Unit 3 HWWV temperatures in the chimney could reach 241^{o}F. The concrete chimney was analyzed for a conservative temperature differential of 236^{o}F across the chimney shell. Thermal stresses are calculated using the ACI Chimney code (ACI 307-69) and combined with stresses due to dead load + 100 mph wind load. These combined stresses are less than the allowable shown in Table 12.2-25.

Shear

Generally, shearing stresses are not high in ordinary chimneys. Due to the extreme tornado loading assumed on this chimney, the maximum shearing stress calculated is 317 psi. The shear due to the tornado load is calculated assuming the whole chimney standing during a 300-mph wind.

Shear reinforcing is designed using the method described by Taylor and Turner,[7] using 1.1 V for the bottom 280 feet and 0.55 V for the upper 320 feet of the chimney. Reinforcing is designed to resist all the shear, i.e., no allowance is made for the concrete in resisting shear.

12.2.4.2 <u>Foundation</u>

The foundation for the chimney shell is shown in Figure 12.2-60. It is a concrete slab 21 feet thick, circular in shape with a diameter of 82 feet. It is founded on sound rock

7 Taylor, C. P., and Turner, L., Reinforced Concrete Chimneys, 2nd Edition, Concrete Publications Limited, London, 1960, pp. 52-53.

and keyed into it 2 to 3 feet deep. The slab projects outside the chimney shell 10 feet and is reinforced in the top and bottom faces as well as the side.

Comparison of the moments in the chimney shell at El. 568.0 for the 100-mph wind, 0.1g earthquake, 0.2g earthquake, and 300-mph tornado wind result in the obvious conclusion that the controlling design case is the 300-mph tornado wind. The foundation is therefore designed to the same criteria as the lower part of the chimney shell; i.e., 1.1 times the tornado moments assuming the entire chimney standing.

To provide stability against overturning requires that the foundation be anchored into the underlying rock. This is done by a ring of 122 No. 11 reinforcing bars (ASTM A-432, FY = 60,000 psi) spaced equally around the circumference of a circle whose radius is two feet less than the radius of the foundation slab. These bars are grouted in three-inch-diameter holes drilled to a depth of 23 feet into sound rock.

In the design of the anchor bars it is assumed that the ground is saturated to El. 561.0. The compressive stress under the foundation and the tensile stress in the anchor bars are calculated according to the straight line stress distribution in a reinforced cracked section. The maximum compression is 1030 psi, well below the allowable concrete stress of 0.85 f'c or 2550 psi, and the minimum value of 11,419 psi for the rock as given in subsection 2.5. The maximum stress in the reinforcing bar at the maximum distance from the center of gravity of the transformed section is limited to 60,000 psi. The depth of the anchor bars is sufficient to engage a wedge of rock whose buoyant weight is equal to the tensile force in the bars.

The toe of the foundation, extending 10 feet past the shell, is designed for the moment and shear at the face of the shell produced by the compressive stress on the rock or the tensile stress in the anchor bars. Reinforcing is placed in the bottom and top face of the toe to resist this moment. The calculated shear stress is 163 psi, which is less than the allowable of $4 \phi (f'c)^{1/2}$ (186 psi) given in Section 1707 of ACI Code 318-63.

12.2.4.3 Internal Structures

The internal structures, as shown in Figure 12.2-61, include of a central shaft of reinforced concrete below El. 599.5 with concrete shielding slabs at El. 599.5 and 580.5. Above this concrete structure is a steel stack to El. 670.0, below which point there is a steel framed roof at El. 665.5.

12.2.4.3.1 Concrete Structure (Figure 12.2-61)

The central shaft acts as a vertical support for the slabs and steel standpipe. The slab at El. 599.5 is also supported around the periphery by concrete columns. The slab at El. 580.5 cantilevers from the central shaft. To prevent interaction between the internal concrete structure and the chimney shell for earthquake conditions, the

slab at El. 599.5 is separated from the shell by a one-inch expansion joint filled with fiberglas insulation.

Concrete slab thicknesses are determined entirely by shielding requirements. In the slabs at El. 580.5 and 599.5 the minimum steel percentage is 200/fy in accordance with Section 911 of the ACI Code 318-63. This percentage is more than adequate for the dead, live, and equipment loads.

In addition, the slab at El. 599.5 is investigated for a 3 psi tornado depressurization creating a low pressure beneath the slab. Using allowable stresses of 0.85 fc and 0.90 fy, steel required is less than the minimum described above.

The central shaft is designed to resist the forces resulting from the dynamic earthquake analysis.

12.2.4.3.2 Steel Structure

The off-gas stack exhaust vent shown in Figure 12.2-62 is a cylindrical steel shell approximately 70 feet 6 inches high and is considered to be fixed at the base to the concrete floor at El. 599.5.

This vent is designed for an internal pressure of 50 psi and external loads of the roof support beams. Design stresses do not exceed the allowable stresses specified in Section VIII of the ASME Boiler and Pressure Vessel Code for low alloy steel with a minimum tensile strength of 70,000 psi.

The vent material conforms to ASTM A-242. Full penetration butt welds are dye penetrant tested.

12.2.4.3.3 Dynamic Earthquake Analysis

In order to evaluate the off-gas duct work, equipment and piping of the off-gas system, an additional dynamic analysis is performed. Using the model described in Section 12.2.4.1, a modal time history analysis was performed, using the artificial acceleration time history input ground motions described in Section 2.5.4. Amplified Response Spectra (ARS), displacement and accelerations are calculated at attachment points on the concrete shell and internal structures for use in subsystem seismic analyses. These spectra were calculated at frequencies ranging from .2 Hz to 20 Hz at intervals consistent with those given in the Standard Review Plan. The ARS were peak broadened ± 10 percent.

12.2.5 Radwaste Building (Class II) (Figures 12.2-64, 12.2-65)

12.2.5.1 Concrete Structure

The Radwaste Building is a cellular box-type concrete structure extending approximately 20 feet below grade and 30 feet above grade and is supported by steel H-piles driven to bedrock. This building houses common services to all three units. It is located adjacent to the Turbine Building, Service Building, Reactor Building, and Diesel-Generator Building. It is isolated from the Reactor and Diesel-Generator Buildings by a two-inch expansion joint filled with fiberglass insulation.

This building is comprised predominantly of thick walls and slabs, the dimensions of which are determined by shielding requirements. In a few instances, walls and slabs are determined by structural requirements. The roof system is a steel framed structure with either bracket supports on concrete walls or steel columns supported by the concrete floor at El. 580.0. One exception is the waste packaging area, the roof of which is concrete.

Two methods of design approach are used. Where frames through the total building can be assumed, moments and shears for the walls and slabs are determined by use of the IBM FRAN computer program. For these same frames, moments and shears are also determined considering the walls and slabs as two-way spans by use of method 2. Moments and shears due to the partial triangular loading on the exterior walls of the waste surge and waste collector tank room are determined by use of Bureau of Reclamation Monograph No. 27.[8]

Output from these analyses is compared and the largest moments are used for design of the sections. Where the building arrangement does not lend itself to a total frame assumption, Method 2 of Appendix A in ACI Code 318-63 is used to determine moments and shears. The principal design cases and allowable stresses are shown in Table 12.2-26.

The walls and slabs of the building which house radioactive tanks and equipment are also investigated for the effect of tornado depressurization forces. The assumption is made that the building is not vented, with the result that a 3 psi internal bursting pressure occurs. The allowable stress criteria is 0.85 fc for concrete in compression and 0.90 fy for reinforcing in tension. The result of this investigation is that the walls and slabs are more than adequate for the internal bursting pressure. Lateral earth pressures are calculated using the Coulomb theory and the following values:

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8 Moody, W. T., "Moments and Reactions for Rectangular Plates," Bureau of Reclamation Engineering Monograph No. 27, U.S. Government Printing Office, Washington, D.C., April 1966.

Angle of internal friction	ϕ= 32 degrees
Angle of friction between fill and structure	δ = 16 degrees
Unit weight of fill:	
Moist	w = 120 pcf
Submerged	w = 65 pcf
Surcharge	200 psf

The Radwaste Building will withstand the design basis tornado except for the steel supported portion of the roof. The remainder of the Radwaste Building roof and all walls and floor slabs will withstand the tornado. The only radwaste equipment that could be affected is the waste sample tanks and the floor drain sample tanks, and liquid radwaste processing equipment which are located under the steel supported roof. The tornado could result in failure of these tanks, the associated piping systems, or liquid radwaste processing equipment. A small fraction of gaseous radioactivity would escape from the building; and, therefore, would result in negligible doses to persons in unrestricted areas (see paragraph 9.2.6 of the FSAR).

The Radwaste Building will not flood because all entrances are either above flood level or are protected by appropriately designed sealed doors. All piping penetrations below flood level are sealed to exclude the water and withstand the water pressure. Thus, the Radwaste Building is adequately protected from the Probable Maximum Flood.

The exterior walls of the room enclosing the waste surge and the waste collector tanks are designed to withstand the Design Basis Earthquake, using maximum allowable concrete stresses in bending of 0.85 fc and reinforcing steel stresses of 0.90 fy. The remainder of the building was not designed for seismic conditions; however, the concrete walls and slabs have been examined for seismic loading. It has been determined that the remainder of the walls and slabs housing radioactive equipment will experience stresses, under Design Basis Earthquake loading, approximately 20 percent greater than the normal stresses specified in the ACI 318-63 code for working stress design. Therefore, reinforcing steel stresses will be approximately 29,000 psi. After the earthquake, maximum reinforcing steel stresses will be 24,000 psi, the normal permissible stress for working stress design.

It can, therefore, be concluded that the Radwaste Building walls and slabs housing radioactive equipment can withstand Design Basis Earthquake loading.

12.2.5.2 Flood Protection Doors

The doors are closures for all personnel and equipment access openings into the Radwaste Building and are identified as follows:

Door for 10-foot by 10-foot exterior opening located in the south wall between column lines W2 and W3; see Figure 1.6-24.

Door for 15-foot 10-1/8-inch-high by 14-foot-wide exterior opening located in the south wall between column line W1 and the outside wall line; see Figure 1.6-24.

Two doors, each 3 feet 0-3/4 inch by 6 feet 7-1/4 inches, for the openings between the Turbine Building and Radwaste Building on column line T1; see Figure 1.6-24.

Two doors, one 3 feet 6-1/4 inches by 7 feet 1-1/4 inches adjacent to column line W4 and one 4 feet 5-1/2 inches by 7 feet 10-1/4 inches at column line W7, for the openings between the Service Building and Radwaste Building on column line Wa; see Figure 1.6-24.

One door 2-feet wide by 4-feet-1-inch high for the opening to the pipe and cable tunnel on column line T1 between column lines k and m at EL. 554.2; see Figure 1.6-24.

12.2.5.2.1 Description

The doors for the two exterior openings are double doors of the sealed, hinged, welded-steel type. They are normally open for personnel and equipment access, but may be closed and latched by dogs to provide watertight units. The door swing and latching dogs are operated by hydraulic cylinders. Hydraulic power units and control stations are provided for each double-door unit. In addition, each door unit is provided with a manually operated hydraulic pump and quick disconnect fittings to provide a means for manual operation of the door. The closure mechanics are part of the Plant Preventive Maintenance Program and are tested periodically.

Load combinations used in designing the exterior doors with corresponding allowable stresses are listed in Table 12.2-39. For all combinations, the resulting stresses do not exceed the allowable. Structural portions of the doors and frame assemblies are fabricated of ASTM A-36 steel.

The four doors for the openings from the Turbine and Service Buildings and the door from the pipe and cable tunnel are the sealed, hinged, manually operated, welded-steel type. The doors are latched by dogs to provide watertight units. The dogs are operated by a single mechanism which is actuated by handwheels on both sides of the doors. The two doors from the Turbine Building are to serve as exit doors only from the Radwaste Building and are normally closed. The handwheels on the Turbine Building sides of these doors latch the dogs only. The door from the Service Building to the waste baler room is normally closed. The door to the corridor leading to the radiochemical lab and the radwaste control room is normally open for

personnel and equipment to access but may be closed and latched by dogs to provide watertight units.

Load combinations used in designing the doors with corresponding allowable stresses are listed in Table 12.2-40. For all combinations the resulting stresses do not exceed the allowable. Structural portions of the door and frame assemblies are fabricated from ASTM A-36 steel.

12.2.5.2.2 Safety Evaluation

The doors for the two exterior openings in the Radwaste Building seal the openings and provide adequate flood protection for the Probable Maximum Flood. These doors are designed to withstand wind to 100 miles per hour; water to EL. 578.0 resulting from floods to El. 572.5 and wave runup or from floods to a lower elevation and greater wave runup; broken waves; and an Operating Basis Earthquake (0.10g) when in the closed position.

In addition, the door at the radwaste packaging room, door 183, is designed to withstand a Design Basis Earthquake (0.20g) when open to prevent the possibility of the door falling on a radwaste package loaded on a transport truck.

The four doors for the openings to the Turbine and Service Buildings and the door for the pipe and cable tunnel provide adequate flood protection for the personnel access openings against the Probable Maximum Flood and are designed to withstand a static water pressure to El. 572.5.

All the doors will remain intact and retain their seal after the occurrence of any one of the above listed conditions, or any combination of conditions listed in the applicable load combination table, Table 12.2-39 for doors in exterior walls and Table 12.2-40 for doors in interior walls.

Welding for all the doors was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction.

12.2.5.2.3 Inspection and Testing

After initial installation, each door was tested for operation of hinges and dogging mechanism. Thereafter, all parts of the doors are to be inspected periodically. Doors and dogging mechanisms are to be inspected for free operation. Seals are to be visually inspected to see if any cracks or blemishes have developed and the surface of the skin plate is to be inspected for paint deterioration.

12.2.6 Office and Service Building (Class II)

12.2.6.1 Office Building

This is a conventional type structure consisting of concrete footings and floors with a structural steel frame enclosed by architectural panels and walls. The structure is designed for applicable dead loads, wind loads, and floor live loads. Floor plans and elevations are shown in Figures 12.2-66 and 12.2-67.

12.2.6.2 Service Building

This structure consists of exterior concrete walls and footings with an interior structural steel frame supported by concrete footings. Concrete floor slabs are used. The structure is designed for applicable dead loads, wind loads, and floor live loads.

12.2.7 Condenser Cooling Water System

12.2.7.1 Pumping Station Structures (Class I) (Figures 12.2-69, 12.2-70)

12.2.7.1.1 Concrete Structure

This structure is 232 feet long, 81 feet wide, and 50 feet high. It is founded on bedrock and backfilled on three sides to approximately the elevation of the top deck. Retaining walls hold back the fill at the ends of the intake side of the structure.

In the longitudinal direction, it is stiffened by two full height walls and three partial height walls extending the full length of the structure. These are shown in section B-B on Figure 12.2-69. In the transverse direction, the structure is stiffened by the many walls and piers making up nine pump bays shown in Plan El. 532 ± on Figure 12.2-70.

Transverse walls are thicker than necessary for structural design because of pump spacing and pump well requirements. All structural concrete elements are thicker than necessary for balanced flexural design of the reinforced concrete. Concrete stresses are therefore far below the allowable compressive stress.

The working stress method of design is used in the interest of crack control. Crack width tolerances vary from 0.005 inch for structural elements of the electrical board and equipment room to 0.015 inch for underwater elements of the structure.

Normal allowable stresses used in design are as specified in the ACI Code 318-63 for concrete having a 28-day compressive strength of 3000 psi and for deformed bars having a yield strength of 60,000 psi or more.

Load conditions for which normal allowable stresses are used include the normal maximum reservoir water level, increases in structure and fill loads for 0.1g ground acceleration due to the Operating Basis Earthquake, and normal operating loads.

The flood protection walls from the deck and grade El. 565 ± to El. 578 are designed to protect the RHRSW pumps from water and wave forces resulting from the Probable Maximum Flood. The contraction joints are sealed with PVC material from the deck to one foot below the top of the wall. The one foot of unsealed contraction joint has been evaluated in design calculations and determined to be insignificant to the wall's protective function. The design is in accordance with the design method given in paragraph 8.1 of ACI Code 318-71. The members are proportioned such that they have a strength capability 1.24 times that required for the water and wave forces from the Probable Maximum Flood. The walls are capable of withstanding a Design Basis Earthquake, since a strength capability of only one-third of that for the Probable Maximum Flood is required.

Full hydrostatic heads measured from the reservoir surface are applied to the entire area of the structure. Stop logs are furnished for the closure of two pump bays, and design is based on the premise that any two pump bays could be unwatered at any time.

Principal design cases, allowable stresses, and calculated safety factors against overturning, floating, and sliding are summarized in Table 12.2-27. The top slab or deck of the structure is designed for a 400-psf uniform live load and in the roadway area is investigated for a 35-ton capacity mobile crane.

The deck was investigated for a flood to El. 578 creating maximum water pressure on the underside and no pressure on the top. The design method given in paragraph 8.1 of ACI Code 318-71 was used for this investigation, with the determination that the deck has a strength capability of 1.4 times that required to resist this flood condition.

The seismic analysis was performed for the DBE case with water elevation at 529 feet and for the OBE case with water elevation at 556.0 feet. This was done so that the analyses would be consistent with the load cases given in Table 12.2-27. The mass of the water enclosed by the structure was included as a lumped mass in the model. The effects of the water in the channel were not included in the analysis.

The structure is surrounded by soil on three sides and by the water in the intake channel on the fourth. Two analyses were performed. In both seismic analyses the soil on the east and west sides were modeled using soil springs. One of the analyses modeled the soil on the north side with soil springs, and one did not. Results from the two analyses were enveloped for use in the design.

Since the structure is founded on sound rock, it is considered fixed in the base. The model was analyzed using the El Centro record, which is described in Section 12.2.2.8.1 to evaluate the structure. The model was analyzed using the artificial acceleration time history, which is also described in Section 12.2.2.8.1 to generate amplified response spectra. Amplified response spectra are generated at frequencies from 0.5 Hz to 20 Hz at intervals consistent with those given in the Standard Review Plan.

Static and dynamic lateral external soil pressures were calculated as described in Section 12.2.2.10 and included in the design.

The structure is investigated for a tornado consisting of an atmospheric pressure decrease of 3 psi in 5 seconds. The calculated pressure differential for the enclosed parts of the intake building, including its ventilation exhaust openings, is 126 psf. This is less than the floor design live loads and is less than the roof deck dead load. The minimum reinforcing in the two-foot-thick walls is also more than adequate for this small pressure, using normal allowable design stresses.

Water conduit transitions are designed to span between the wall of the pumping station and a pile support. Earth support for the transitions is assumed only for the dead load of the first concrete pour. The steel liner is neglected in the structural design of the reinforced concrete conduit transitions. The transitions are designed for two conditions: no external load and full internal pressure, and maximum external load when empty.

12.2.7.1.2 Personnel Access Doors

Description

The doors are closures for the four 3 feet 11 inches wide x 7 feet 4 inches high openings at El. 565 in the intake structure north wall (see Figure 12.2-69), which provide personnel access to the Residual Heat Removal Service Water (RHRSW) pump compartments.

The doors are identical and are the sealed, hinged, manually operated, welded steel type. The doors are normally closed and latched by dogs to provide watertight units. The dogs are operated by a single mechanism which is actuated by handwheels on both sides of the doors.

Load combinations used in designing the doors with corresponding allowable stresses are listed in Table 12.2-41. For all combinations, the resulting stresses do not exceed the allowable. Structural members of the doors and frame assemblies are fabricated of ASTM A-36 steel and stainless steel.

Safety Evaluation

These personnel access doors provide adequate flood protection against the Probable Maximum Flood for the Residual Heat Removal Service water pumps and are designed to withstand a wind velocity of 300 mph, static water pressure to El. 578.0 (13 feet) and the Design Basis Earthquake (0.20g). The doors will remain intact and retain their seal after the occurrence of any one of these conditions or any combination of conditions listed in Table 12.2-41.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction. Certified mill tests, covering chemical analyses and physical properties, are on file for all materials.

Inspection and Testing

After initial installation, each door was tested for operation of hinges and dogging mechanism. Thereafter, all parts of the doors are to be inspected periodically. Doors and dogging mechanisms are to be inspected for free operation. Seals are to be visually inspected to see if any cracks or blemishes have developed and the painted portion of the door and frame assembly are to be inspected for paint deterioration.

12.2.7.2 <u>Intake Channel</u>

The intake channel that connects the pumping station to Wheeler Reservoir is designed to provide water to the pumps during both of the following conditions:

a. The probable maximum flood condition of the reservoir; and
b. The minimum water level which could conceivably be caused by the breach of Wheeler Dam.

The pumping station intake building houses equipment which provides all cooling water for the plant.

The depth of the intake channel shown in Figure 12.2-71a is controlled by item b. above. The side slopes are controlled by the requirement for a 1.5 minimum safety factor for a sudden drawdown condition--the channel drained and the embankment saturated.
The design was based on the following investigations.

1. Soils values for static and dynamic design analysis were obtained in a soils investigation program in 1966, as described in paragraph 2.5.2.4.3. The soil structure is such that liquification should not be a problem.

2. Slip-circle analyses of the channel slopes were performed, and the minimum safety factor calculated for the critical design condition was 1.67. These calculations were based on the assumption that all soils in the cross section had the same values as the least desirable material sampled.

3. An assessment was made of the seismic-resistant capabilities of 1 on 3 earth slopes when subjected to the Operating Basis and Design Basis Earthquake ground motions. A two-dimensional finite-element method of analysis was used to obtain the seismic response of the channel slopes.

In order to simulate the condition at drawdown, the effect of the water in the channel was neglected. The mathematical model was subjected to both static (dead load) and dynamic earthquake forces, and the resulting shear stresses were computed. Under both the severe drawdown condition and the Operating Basis Earthquake, the maximum combined shear stress was calculated to be about 1400 psf. With an allowable shear stress of 3000 psf, the safety factor was more than 2.1. In addition, the shear stress determined on the basis of the Design Basis Earthquake criteria (twice the design stress of OBE or about 2800 psf) did not exceed the allowable value.

The original intake channel was excavated with a 1 on 3 slope over its full length. Subsequently, a slide occurred in one slope of a portion of the channel between a construction dike and the pumping station. This slide occurred in April 1970, about 1-1/2 years after the channel was excavated and before the dike was removed. The slide took place between two of the locations where samples were taken during the 1966 soils investigation program. To evaluate the cause of the slide, additional tube and block samples were then taken approximately midway between the old sampling locations and on both sides of the channel. Two conditions were noted which had not been observed in the 1966 investigation: (1) the in-situ clay was fissured, and (2) there was a thin low-shear strength fat clay layer between the fissured cherty clay and bed rock. Analytical investigations resulted in the conclusion that the slide action was that of a motive wedge sliding on the near-horizontal lens of fat clay, being driven by full hydrostatic pressure in a near-vertical crack.

Using the motive wedge approach and the shear strength of the fat clay lens just above bed rock, static analyses of the slope were made using various lengths of rock fill in place of the in-situ material. For a safety factor equal to 1.5, it was necessary to replace all in-situ material above bed rock for a minimum width of 52 ft

measured on each side of the Underline of the intake channel, as illustrated in Figure 12.2-71b. The channel was rebuilt in this manner between the construction dike and the pumping station.

Beyond the construction dike the slopes were flattened to 1 on 6 as shown in Figure 12.2-71b. The static factor of safety for this slope is greater than 1.5. After the safe slopes for the static conditions had been determined, two analyses were performed to evaluate the seismic stability of the channel slopes. The first analysis determined the increase in the driving forces on the motive wedge required to reduce the static factor of safety from 1.5 to 1.0. This incremental force was then divided by the saturated weight of the motive wedge to give a pseudo acceleration necessary to cause movement of the motive wedge. Since the peak acceleration at bedrock exceeded the pseudo acceleration, it was concluded that during a Design Basis Earthquake a crack would form and the wedge would begin to move. The second analysis assumed that the movement of the wedge as outlined above would occur and an evaluation of the horizontal displacement was made using the method outlined by Newmark in Geotechnique, Volume 15, 1965. A maximum displacement of 8.5 was computed for the Design Basis Earthquake.

The sensitivity of the soil was investigated to determine the effect of a possible loss in shear strength on the stability of the slopes after an earthquake. It was concluded that the shear strength of the soil used in the post earthquake static analysis was representative of the residual shear strength which the soil would have following disturbance by an earthquake.

From the earthquake analysis it was concluded that movement of the intake channel slopes due to the Design Basis Earthquake following a rapid drawdown of the reservoir would not pose a threat to safe shutdown of the reactor.

The Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, provided consulting services for this investigation. These services included a field examination of the slide, recommendations for additional field and office investigations, a review of all field, laboratory and design office investigations and analyses, and several consultations. The consultant's final report states that the investigations and analyses of the channel slopes are reasonable and sufficient and provide a basis on which to initiate repair and reconstruction of the slopes.

12.2.7.3 Circulating Water Conduits

The conduits are constructed of reinforced concrete, 14 feet 6 inches square, or 16 feet 6 inches round, as shown in Figure 12.2-71a. The conduits are designed by column analogy and checked by the IBM FRAN computer program. The principal design cases and allowable stresses are as given in Table 12.2-28.

12.2.7.4 Discharge Stoplog Structure

This structure was located over the three barrel discharge conduit at the edge of the reservoir, as shown in Figure 12.2-71a. This structure has been abandoned and cut off below grade as shown in Figure 12.2-72c. Modification of this structure is discussed in paragraph 12.2.7.7.

12.2.7.5 Diffuser System

When conditions permit operating the condenser circulating water system in the open mode, the warmed condenser water is returned to the Wheeler Reservoir by means of diffuser pipes. Three partially perforated pipes are connected to the discharge conduits of the three units as shown in Figure 12.2-73. These perforated, corrugated, galvanized steel pipes are laid side by side across the bottom of the 1800-foot-wide channel, which is approximately 30 feet deep. The pipes (17 feet, 19 feet, and 20 feet 6 inches in diameter) are of different lengths. Each has the last 600 feet perforated on the downstream side with more than 7000 two-inch-diameter holes. Thus, approximately 22,000 holes spaced 6 inches on centers in both directions distribute the 4400 cfs (approximate) of warm water into the river for thermal mixing.

The diffuser pipes within the channel are secured firmly by a system of plastic-coated steel cables and anchor rods grouted into bedrock. The anchorage system is designed to withstand design exposures ranging from high speed barge crossings at minimum water elevation to maximum design floods.

12.2.7.6 Gate Structure in Intake Channel for Cooling Tower System (Class I)

The gate structure across the intake channel is shown in Figures 12.2-72a and 12.2-74. Parts of the gate structure were built before the first unit was placed in operation. The gate, the skimmer wall, and the machinery deck were installed after operation began. These latter items are identified in Figure 12.2-74. The gate guide cells were built before startup of the first unit; and construction of the closure cells was started, but not completed, before startup of the unit.

The gates are not seismically designed. Therefore, the gate was investigated for a free fall. In a free fall from the open position, neither gate nor the gate supports will fail in such a way that the gate can prevent the required emergency flow through the opening below the gate. It is also practically inconceivable that all three gates can be buckled laterally to the extent that they are pushed out of the guide slots and in this shape completely block the flow through all three gate openings.

The gate has no safety-related function. The gate will not shut off flow to the RHR service water pumps located in the pumping station. The design will ensure sufficient

emergency cooling water at all times by providing a permanent fixed opening below the gate.

The gate guide cells consist of steel sheet piling driven to bedrock and filled with concrete. Concrete was placed under water by tremie. Steel and concrete keys into bedrock were provided by drilling large holes through the cell concrete into rock and placing steel and concrete in these keyways. The keys are designed to resist sliding without taking advantage of any friction between the cell concrete and the rock surface.

The gate cells are designed for loads representing both final closed cycle conditions and initial construction conditions. For both of these conditions, design loads include dead loads, earth pressures, water pressures for the design maximum water level, and Design Basis Earthquake loads. For the final condition, gate and skimmer wall loads are added to the direct loads on the gate guide cells.

The closure cells were built by driving sheet steel piling through the overburden to rock and filling the enclosures above the overburden with crushed stone. Material tests were conducted to verify design requirements, including the density of the crushed rock, and for the steel sheet piling, chemical analysis, yield strength of steel, and strength of interlocks. Vertical shear is transferred from the piling to a concrete cap block. Design loads include dead load, earth pressures, water pressures for the design maximum water level, and Design Basis Earthquake loads.

A load diagram is shown in Figure 12.2-75a, and loads and design cases are defined in more detail in Table 12.2-37.

Factors of safety against overturning and sliding have been computed as follows.

Design Case	Factors of Safety Overturning	Factors of Safety Sliding
Gate Cells		
Initial condition	1.54	1.85
Final condition	1.13	1.33
Large Closure Cells	1.06	1.09
Small Closure Cells	1.10	1.50

Description of the Design Basis Earthquake is given in paragraph 2.5.4. Seismic water and lateral earth pressures are determined as described in paragraph 12.2.2.10. Cells partially embedded in the earth are assumed to have the same accelerations as the earth around the cells.

The accelerations of the earth were found by amplifying the accelerations of the rock. This amplification factor was found by considering the earth as an elastic medium

and making a dynamic analysis of a slice of unit thickness using only the horizontal shearing resistance of the earth. A damping ratio of 0.10 was used for the earth in this analysis.

Based on analyses for the design cases in Table 12.2-37, considering the cells individually and their interaction effects, the cells are stable.

In addition to the design cases with water at the design maximum water level, the cell loads were analyzed for the water at El. 529.0, based on the loss of the downstream dam in conjunction with the Design Basis Earthquake. This condition was less severe than the design maximum water level cases.

The gate guide cells were also investigated for the unlikely event of impact due to a runaway, fully-loaded coal barge at Maximum Probable Flood. Under these conditions the cells can absorb sufficient energy to stop a barge traveling at 5 mph without overturning. Even if a gate guide cell were to turn over so that it fell across the channel, flow to the intake pumping station would be through either or both remaining gate openings. The smallest opening with the gates closed provides an area of approximately 130 square feet to accommodate the 80 cfs flow needed for shutdown cooling of all three units, as noted in paragraph 2.4.2.2.2.

If a barge were to sink in the channel, flow of water to the intake pumping station would not be blocked since normal minimum pool level is El. 550, bottom of the intake channel is El. 523, and a maximum depth of a coal barge is less than 15 feet.

12.2.7.7 Alteration to Discharge Conduits for Cooling Tower Systems

The discharge conduits were constructed as shown in Figure 12.2-71a. In order to provide for discharge of condenser circulating water to the cooling towers (paragraph 12.2.7.6), alterations were made to 80 ft of discharge conduits immediately upstream of the stoplog structure.

These alterations are shown in Figure 12.2-72c. Two existing blocks (80 ft in length) were removed and replaced with a combined conduit and two gate structures (No. 1A, No. 1B). Gate structure No. 1A, will serve the function now performed by the existing stoplog structure. In addition, it will serve as a gate to divert discharge water to the cooling towers through gate structure No. 1B.

The altered discharge conduits are shown in Sections A3-A3, B3-B3, C3-C3 and D3-D3, of Figure 12.2-72c. These altered conduits permit the discharge water to pass to the diffuser or be diverted to the cooling towers.

Figure 12.2.-72c also shows gate structure No. 1 and openings cut into the top of each discharge conduit to allow cooled water to be discharged to the reservoir.

12.2.7.8 Auxiliary Condenser Cooling Water System

The Auxiliary Condenser Cooling Water System is shown on Figures 12.2-72a, 12.2-72b, and 12.2-72c. The system consists of waterways, control structures and cooling towers to permit helper system operation. The triple-box culvert leading from the original discharge area to the cooling tower area is shown in Figures 12.2-75b sheet 1, 12.2-75b sheet 2, and 12.2-75b sheet 3.

Concrete structures are designed in general in accordance with the Building Code of the American Concrete Institute (ACI 318-71).

Operation of the discharge control structure, gate structure No. 1, gate structure No. 2, and the cool water channel between the cooling towers and the pumping station was model tested. The needs for structure modifications, channel blocks, and protective paving and riprap were determined by these model tests.

12.2.7.8.1 Waterways

The waterways include a reinforced concrete triple-box culvert, precast pipe, and open channels. The culvert and precast pipe are designed for normal applicable dead, live, and surcharge loads with appropriate load factors. The triple-box culvert has also been investigated and determined safe for a fuel cask truck surcharge load.

The dike which forms the waterway between the discharge control structure and the intake channel (Figure 12.2-72a) is designed to be placed partly under water. Where this dike crosses the diffuser pipes, a pile supported protective slab is provided to protect the diffuser from the crushing load of the dike.

There are two operations in the construction of the cool water channel which relate to the intake channel design. The first is the addition of the earth dike for the new channel. This operation does not affect the design for the intake channel side slopes. The second operation is the removal of a portion of the west bank of the intake channel having the shape of the new channel. Removal of this material from the intake channel west bank increases the greater-than-1.5 safety factor referred to in paragraph 12.2.7.2.

12.2.7.8.2 Cooling Towers

Each of seven cooling towers is supported on a grade level shallow depth reinforced concrete basin which is designed as a spread footing for the cooling tower. The basins vary in depth from 2 to 4 feet. They are designed for normal, applicable dead and live loads and normal allowable differential settlement and soil bearing capacity.

12.2.7.8.3 Control Structures

The control structures consist of a pumping station, several gate structures, a discharge control structure, and a hump in the warm water conduits. All of these structures are founded directly or indirectly on rock, as indicated in Figures 12.2-72b, 12.2-72c, and 12.2-74. Excluding gate structures No. 2 and 3, these structures are seismically unclassified and were designed for normal applicable dead, live, and surcharge loads with appropriate load factors.

A vacuum breaker system, associated with the hump in the warm water discharge conduits, ensures that the vacuum established in the warm water conduits under normal operation can be broken under Design Bases Earthquake conditions. This is necessary to ensure that the water in the warm water channel can be prevented from flowing back to the intake channel, because of a loss of the downstream dam, and polluting the emergency raw cooling water supply.

The vacuum breaker system is housed in a vacuum pump building and a vacuum pipe building. Both buildings are below grade. The pump building is seismically unclassified. The pipe building is a Class I building and is located directly above the hump or steel-lined siphon portion of the warm water conduits. Earth backfill covers the top of this building to a depth of about 2 feet. The building is founded on earth backfill compacted to 95 percent of maximum Standard Proctor density at optimum moisture content.

The Vacuum Pipe Building is basically a single-barreled concrete box frame with closed ends. The walls and top and bottom slabs are designed as one-way slabs using coefficients from the ACI Building Code 318-71 for reinforced concrete and the basic strength design method. The Vacuum Pipe Building is dynamically rigid and, therefore, the seismic accelerations throughout the building are the same. The seismic accelerations at bedrock were amplified through the soil to the elevation of the Vacuum Pipe Building. The amplification factors used to calculate accelerations, displacements, and Accelerations Response Spectra (ARS) were 1.6 in the horizontal directions and 1.3 in the vertical direction. The static analysis is performed in a conventional manner with earth pressures determined as described in paragraph 12.2.2.10. Stresses resulting from the static analysis are combined by the method of superposition with stresses from the Design Basis Earthquake loading. The magnitude of the increase in lateral soil pressure due to seismic motion has been determined by shaking table experiments performed for the design of TVA's Kentucky hydro project. A ground acceleration of 0.4g is used in design based on a shear-wave-to-height ratio approximately equal to ten (10). The maximum accelerations at the top of the soil occur at this ratio. For ground acceleration of 0.4g, the static soil pressure is increased by a factor of 2.4 for dry soil. This incremental increase is combined with the static pressure as a triangle of pressure, whose apex is at the rock surface and maximum ordinate is at the ground surface. This structure is not designed for tornado conditions.

12.2.7.8.4 Gate Structure No. 2 (Class I)

Gate Structure No. 2 is located in the cool water channel between the discharge control structure and the intake channel (forebay) and is shown in Figures 12.2-84 and 12.2-85. The primary structural components of the gate structure are the concrete gravity section, the machinery deck, and the cellular cofferdams.

The concrete gravity section is a reinforced concrete structure founded on bedrock. This substructure supports the machinery deck and houses two submerged gates which are permanently in the closed position due to the elimination of the closed mode of CCW system operation.

The cellular cofferdams consist of steel sheet piling driven to bedrock and backfilled with crushed stone and/or flowable fill concrete. The cofferdams extend from the concrete gravity section to the north bank of the cool water channel and to the south to the earthen dike separating the cool water channel from the Wheeler reservoir.

Gate Structure No. 2 provides the protective safety function of limiting the potential thermal mixing of the warmer water discharged from the cooling towers during their operation with the water in the intake pumping station channel which provides the makeup for the RHRSW system. The potential for mixing of the two bodies of water is credible only with the cooling towers in operation during an extreme rainfall event. This limited thermal mixing has been evaluated and determined not to result in the temperature of the RHRSW makeup water exceeding the design temperature limit.

Gate Structure No. 2 was initially designed as a Class II structure for normal applicable dead, live, and surcharge loads with appropriate load factors. Subsequent to the initial design and construction, the structure has been evaluated for stability against sliding and overturning for the design cases in Table 12.2-46. The seismic accelerations were based on bedrock accelerations. This analysis determined that the gate structure would maintain its structural integrity during and after a DBE.

12.2.8 Diesel Generator Building, Units 1 and 2 (Class I)

12.2.8.1 Concrete Structure (Figure 12.2-76)

This structure is located adjacent to the west side of the Reactor Building and the south side of the Radwaste Building. It is separated from these buildings by a two-inch expansion joint filled with Fiberglas insulation. The south end of the building faces an earth backfill varying to a height of approximately 29 feet above the lower floor level. The west side of the building is exposed. The foundation for the structure is 3+ feet of earth backfill compacted to 95 percent of maximum Standard Proctor density at optimum moisture content. The only materials used for backfill are clay sand (SC) and lean clay (CL). Laboratory testing of soil samples has determined that

the allowable static soil bearing value is 1.5 tons/ft^2. Underlying this earth backfill is approximately 32 feet of crushed stone backfill replacing the material excavated for the construction of the Reactor Building.

This is a well graded stone placed in 4- to 6-inch layers and compacted with 3-wheel vibrating rollers.

The structure is a two-story concrete box, with one longitudinal dividing wall full length and height, and four transverse dividing walls full height terminating at the longitudinal wall. Each of the four bays encloses a diesel generator and its auxiliary equipment. A raised grated air intake and exhaust plenum over each generator bay provides for air intake and exhaust. The grade floor slab consists of diesel fuel storage tanks encased in concrete.

Two new air-cooled rotary screw chillers, two chiller pumps, two transformers, piping and associated valves and a concrete enclosure consisting of concrete walls and a steel/aluminum grating roof, have been added to the roof of the Unit 1 and 2 Diesel Generator Building. The impact of additional equipment and the concrete enclosure at roof elevation 594.5' necessitated the generation of a revised response spectra. The final model including the enclosure is shown in Figure 12.2-77a.

The Diesel Generator Building's dynamic behavior is simulated using beam elements representing floor to floor wall systems connected to lumped floor masses. The original 2-D models are converted to a single 3-D model and used in subsequent Soil Structure Interaction (SSI) analyses. The chiller enclosure itself is modeled with the same assumptions as those applied to the main structure. The predominant deformation of the enclosure is in shear due to the long concrete walls surrounding the chillers.

Along the side adjacent to the Radwaste Building a wall extends down from the structure to the bottom of the Radwaste Building. This wall is integral with the Diesel Generator Building and is designed to retain the fill under the building without any dependence on the Radwaste Building. Since the Radwaste Building is a Class II structure, this integral wall will guarantee that the Class I capability of the Diesel Generator Building is not compromised.

The analysis of the structure assumes the exterior walls fixed at El. 565.5, continuous across the El. 583.5 floor, and pinned at the roof El. 595.0. This frame is analyzed using the loading conditions as given in Table 12.2-29, with lateral earth pressures determined as described in paragraph 12.2.2.10. The El. 583.5 floor and roof El. 595.0 are one-way slabs continuous across interior walls and restrained at exterior walls. All horizontal loads are transmitted through floor and roof to parallel walls and thence to the foundation.

Stresses resulting from the static analysis are combined by the method of superposition with stresses resulting from moments, shears, deflections, and accelerations determined by dynamic analysis. Members are proportioned by the working stress method of ACI Code 318-63 so that the above stresses do not exceed the allowable stresses given in Table 12.2-29.

The expansion joint loads as listed in Table 12.2-29 are determined by two factors: (1) the lateral movement during earthquake of the Diesel Generator Building and Reactor Building, and (2) the fluid pressure of freshly placed concrete.

The lateral movement of the Reactor Building is as determined by the dynamic earthquake analysis described in paragraph 12.2.2.8. This movement in relation to the two-inch expansion joint between buildings is insignificant. The lateral movement of the diesel generator building is conservatively assumed to be the same as that for the surface of the overburden when the rock surface is subjected to earthquake accelerations of 0.1g or 0.2g. The amount of this movement is calculated to be 0.11 inch and 0.22 inch for Operational Basis Earthquake (0.1g) and Design Basis Earthquake (0.2g), respectively. It is assumed that the two buildings move in opposing directions, thereby reducing the width of the expansion joint by their movement. Using the compression characteristics of the Fiberglas insulation in the joint and the amount the material is compressed, the load imposed on the walls by lateral movement during earthquake can then be determined.

The fluid pressure of freshly placed concrete is determined by the methods given in the ACI Publication SP-4.[9] The rate of placement was limited to a maximum of two feet per hour and the temperature of concrete in the forms was limited to a minimum of 50°F. This results in a maximum lateral pressure of 510 psf.

The tornado depressurization load as listed in Table 12.2-29 is determined by an analysis that considers pressure versus time as a function of the vent area available. The computer program used is the same referred to in answer to question 2.1 in Amendment 2 of the Unit 3 Design and Analysis Report. The result of this analysis is that the pressure differential is less than 40 psf. This is the internal pressure load used for case V.

The static load of 230 lb/sq ft from tornado wind used for case VI is determined as described in paragraph 12.2.2.9.

The concrete walls are capable of resisting the spectrum of postulated tornado-generated missiles as described in paragraph 12.2.2.9.2 except for walls less than 18-inches thick above El. 583.5'. A probabilistic analysis of these walls and the Diesel Generator Exhaust Stacks shows that the frequency of occurrence of

[9] Hurd, M. K., 'Formwork for Concrete,' Special Publication No. 4, American Concrete Institute, Detroit, Michigan, 1963, pp. 75-75.

tornado-generated missile strike is less than or equal to 1.0 x 10^{-7} per year which meets the NUREG-0800 U.S. NRC Standard Review Plan acceptance criteria of 1.0 x 10^{-7} per year. The analysis demonstrated that tornado-generated missile strikes are not credible events.

Several walls less than 18-inches thick are not included in the probability calculation. The east side of the units 1 and 2 and the west side of unit 3 air intake and exhaust plenum are not considered as probable targets of a missile hit because of protection provided by the taller reactor building. The north, south, and east walls of the unit 3 mechanical equipment room above El. 597.6' are not included because even if there is a loss of air conditioning equipment due to tornado missiles, acceptable temperatures can be maintained in the 4KV shutdown board rooms 3EA, B, C, and D and the bus tie board room for up to 72 hours. Beyond that time, Plant Procedures (0-A0I-100-7 and 0-0I-31) will be followed.

Therefore, it is concluded that the integrity of system 82 and its associated components can be maintained. These conclusions have been documented in the closure of CAQR BFP890327.

12.2.8.2 Access Doors

12.2.8.2.1 Description

The doors are closures for the four 8 feet high x 9 feet 6 inches wide openings and one 8 feet high x 11 feet 6 inches wide opening in the Diesel-Generator Building. The four doors provide access to the diesel generator units, and the one door provides access to the CO_2 room. Doors at the rear of rooms for the diesel generator units and the CO_2 room connect to the pipe and electrical tunnel.

The two leaves of each door are hinged on the outside to an embedded frame and held at the top and bottom from swinging inward by shear bars. Both leaves are held near the center of the opening by latching pins. On one leaf, the inactive leaf, the latching pins are manually positioned from the inside only. On the other leaf, the active leaf, the pins are spring loaded to engage when the door is closed and are manually released from either side. The active leaf, which is used for normal access, is locked from the outside only, but may be unlocked and opened from either side. On units 1 and 2, the active leaf is manually opened and closed. On unit 3, the active leaf is opened and closed by a pneumatic system. Operation of the pneumatic system is initiated by moving the latching handle to the open or closed position. The inactive leaf on all three units is manually opened and closed. Each door is of welded steel construction and is fabricated from ASTM A-36 steel. Each door leaf has a structural steel framework with a solid steel skin plate on both sides.

Load combinations used in designing the doors with corresponding allowable stresses are listed in Table 12.2-30. For all combinations (except VI), the resulting

stresses do not exceed the allowable. For combination VI, the doors will deform, but will stop a missile.

12.2.8.2.2 <u>Safety Evaluation</u>

These access doors provide adequate protection for the diesel-generator units and are designed to withstand tornado conditions and missiles generated by tornadoes, flood conditions, or earthquakes, with only one of these conditions occurring at any one time. During plant operation, the doors are normally closed, latched, and locked. Replaceable rubber seals on the doors are always in place and seal anytime the doors are closed.

Tornado conditions consist of winds to 300 mph and missiles generated by the 300 mph wind with impact speeds of 100 mph. Flood conditions consist of floods up to and including a probable maximum flood to El. 572.5, with wave runup to El. 578.0. Broken waves and surge forces may occur during floods, and wave runup for floods less than the maximum may vary, but not exceed El. 578.0.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction. Certified mill tests, covering chemical analyses and physical properties, are on file for all materials.

12.2.8.2.3 <u>Inspection and Testing</u>

After initial installation, all doors are to be tested for operation of hinges, latches, and locks. Thereafter, all parts of the doors are to be inspected periodically. Doors, openers, and locking mechanisms are to be inspected for free operation, and the surface of the skin plate is to be inspected for paint deterioration. Seals are to be inspected for cracks, wear, and deterioration.

12.2.8.3 <u>Dynamic Earthquake and Analysis</u>

The structure is characterized by the mathematical model shown in Figure 12.2-77a consisting of lumped masses resting on weightless, elastic columns, with the linear springs K_T, K_R, and K_V. Here, K_T represents the resistance to horizontal motion offered by the soil, K_R represents the resistance to rotation offered by the soil, and K_V represents the resistance to vertical motion by the soil. The values of K_T, K_R, and K_V are found by using a plane strain finite element analysis of typical slices along the two axes of the building and subjecting the slice to horizontal and vertical forces, and moments.

The peak of the input time history for the diesel generator building models was determined by multiplying the acceleration time history at the top of sound rock by

amplification factors for the vertical and horizontal directions. These amplification factors were determined by calculating the first mode frequencies of the soil deposit in the horizontal and vertical directions. These frequencies were used to find the spectral accelerations from the design ground acceleration spectrum at 10% damping. For each direction, the ratio of the spectral acceleration to the zero period acceleration was multiplied by the participation factor and mode shape for the first mode to calculate the amplification factor. Amplification factors of 1.6 in the horizontal direction and 1.1 in the vertical direction were conservatively used in the analysis of the diesel generator building.

Using the model in Figure 12.2-77a (including soil springs) described above, two time history analyses were performed using 5% damping for all modes.

a) For the design of the primary structure as well as floors integrally connected to the primary structure, the analysis was performed using the El Centro earthquake record described in Section 2.5.4. The peak horizontal acceleration of .20 g for DBE and .10 g for OBE and the peak vertical acceleration of .13 g for DBE and .067 g for OBE are increased by the amplification factors described above. Resulting axial forces, moments and shears are used in the design of the structure and integrally connected walls and slabs.

b) For use in the design of the subsystems housed in the building, the analysis was performed using the artificial Time History ground acceleration record described in Section 2.5.4, multiplied by the amplification factor described above. Amplified Response Spectra (ARS) were developed at each mass location for frequencies ranging from 0.2 Hz to 20 Hz at intervals consistent with those given in the Standard Review Plan. These spectra were peak broadened by ±15 percent.

   Vertical responses due to rocking caused by the horizontal input motion are combined with vertical responses due to the vertical input motion using the SRSS method.

Since the dimensions of the diesel generator building for Units 1 and 2 and the depth of the underlying soil deposit differ from those of the Unit 3 diesel generator building, separate analyses are performed for the two structures.

The models for Units 1, 2, and 3 are shown in Figures 12.2-77a and 12.2-77b, respectively. Amplified Response Spectra (ARS) developed from the analyses of the two buildings (original ‘2D’ models) were enveloped for use in subsystem analyses of Unit 3.

12.2.8.4 Portable Bulkhead

12.2.8.4.1 Description

The portable bulkhead is part of the Diesel-Generator Building flood protection for the Probable Maximum Flood. It will be used to seal any one of the four rear interior doorways between the affected Diesel-Generator Room and the pipe and electrical corridor, to protect the other Diesel-Generator Rooms from the flood water. When an exterior watertight door to a Diesel-Generator Room is inoperable and not capable of performing its flood protection function and flooding is imminent, the rear interior door in the affected Diesel-Generator Room will be removed from its hinges and the portable bulkhead bolted over the doorway.

Flooding is considered imminent when the Lake Elevation High alarm at the Intake Pumping Station is activated at elevation 564 and all upstream dams have been breeched as discussed in Appendix 2.4A of Chapter 2 (maximum Possible Flood).

The bulkhead is approximately 7 feet 7 inches high and 3 feet 10 inches wide, of welded steel construction, and fabricated from ASTM A-36 steel. It consists of a structural steel framework with a solid steel skin plate on one side. The other side is equipped with eye bolts and channels for bolting to a doorway.

Load combinations and corresponding allowable stresses used in designing the bulkhead are listed in Table 12.2-42. The resulting stresses do not exceed the allowable for any load combination used.

12.2.8.4.2 Safety Evaluation

This bulkhead provides adequate protection against flooding of adjacent Diesel-Generator Rooms due to water traveling through an open access door, into the pipe and electrical corridor, and through the doorways between the corridor and other diesel generator rooms. It is equipped with replaceable seals to seal against static flood heads to El. 578.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction; and certified mill tests, covering chemical analyses and physical properties, are on file for all materials.

12.2.8.4.3 Inspection and Testing

The bulkhead is to be inspected for deterioration of seals and structural members on a periodic basis.

12.2.9 Equipment Access Lock (Class I)

12.2.9.1 Concrete Structure

The equipment access lock (EAL) is a concrete box structure approximately 26 feet wide by 26 feet high, extending outward from the south side of the Reactor Building approximately 106 feet. The structure is supported by a row of steel bearing piles to rock under each vertical wall, and another row at the midpoint of the ground level slab. In this manner, differential settlement and alignment problems are eliminated at the face of the rock-supported Reactor Building.

The structure is covered to a depth of approximately three feet by the 30-foot-high earth berm which surrounds the reactor building. A hardened wetwell vent for Unit 3 also runs across the top of the EAL in a section built-up berm. Separation from the Reactor Building is provided for by a two-inch expansion joint filled with Fiberglas insulation. Static and dynamic lateral earth pressures are determined as described in paragraph 12.2.2.10. Vertical accelerations from the Operating Basis Earthquake (0.067g) and Design Basis Earthquake (0.13g) are used to increase or decrease the vertical load, whichever is conservative.

The loading on the ground level slab consists of the Dry Cask Storage spent fuel cask overpack and carrier, or Low Profile Transporter (LPT). Provisions were made in the original design of the structure to accept a spent fuel cask on either a rubber tired trailer or a railcar. The design considered either of these loadings, with the result that the railcar loading for transport of a GE cask was the most critical load condition. However, for Dry Cask Storage operations, the transport of the Dry Cask Storage overpack using the LPT exceeds the original design parameters, which necessitated a reanalysis to confirm the adequacy of the EAL for dry cask storage operations.

The structure was originally analyzed as a box-type frame by a computer program for the design cases given in Design Criteria BFN-50-C-7100, Design of Civil Structures. For each design case, the loads were combined in various combinations so as to produce maximum design moments at any point in the frame. The stresses at each section did not exceed the allowable stresses given in Design Criteria BFN-50-C-7100.

The scope of this new EAL structural evaluation is limited to the EAL slab and the underlying supporting piles; both the OBE and DBE seismic loading conditions are considered in the analysis. Since Dry Cask Storage operations do not create new loading conditions for the EAL roof and walls. Therefore, these upper structural components are not evaluated in this reanalysis but are modeled in order to correctly account for the loads transferred from them to the slab. Additionally, the seismic stability of the freestanding overpack/LPT during transport operations in the

EAL is evaluated for the specified seismic events to provide bounding loads applied to the slab from the cask.

The EAL slab is evaluated by using the finite element method under the load combinations specified in Design Criteria BFN-50-C-7100, Design of Civil Structures. A 3-D finite element model is developed to evaluate the EAL slab responses to all applicable loads, except the lateral seismic loads, and includes all structural components including the underlying supporting piles. Peak EAL slab responses resulting from the soil structure interaction (SSI) under the specified lateral seismic loads are obtained by using a 2-D plain strain finite element model developed for transient SSI analysis. The finite element analyses are carried out with the commercial finite element code ANSYS.

In accordance with the Design Criteria BFN-50-C-7100, the working stress design method for concrete specified in ACI 318-63 is used in forming load combinations and setting allowable limits. The EAL slab loading capacities at critical cross sections are calculated and compared with the results from the EAL finite element analyses under all applicable load combinations.

The EAL rail local support is also evaluated with the finite element method. The EAL piles foundation analysis is performed by using the same approach as in the original calculations. The stability of the freestanding overpack/LPT is evaluated by performing rigid body dynamic simulations for the specified OBE and DBE seismic events. Seismic interface loads for the rigid body simulations are applied to the EAL rails and slab.

The rails extend outside onto an apron slab which sits on compacted soil at the entrance of the EAL. The apron slab and its rails are evaluated to ensure that they can support a loaded overpack/LPT and the transport crawler during the cask transport operation. Running on the existing rails, the LPT carries the loaded overpack cask to the apron slab, where the crawler picks it up and moves the cask to the ISFSI site. In addition to bearing the loads from the dolly and crawler, the apron slab is also subjected to loads applied along the east and west edges by the two walls that retain the soil tapering down from the roof of the buried EAL. Figure 12.2-71a shows the general layout of the apron slab and retaining walls relative to the EAL. The slab and rails are analyzed for credible loading conditions (i.e., normal, OBE, and DBE) for all credible scenarios of overpack cask/LPT/crawler.

12.2.9.2 Doors

12.2.9.2.1 Description

The equipment access lock doors as shown in Figure 12.2-80 provide a sealed closure for the 21-foot 6-inch by 24-foot 6-inch opening at each end of the access

lock. Each door consists of two leaves and one sub-door. Each leaf is hinged on the outer side, held at the center of the opening by manually-operated locking pins, and sealed on four sides by inflatable rubber seals. The seals are mounted on the door leaves and are pressurized after the door is closed. The structural portion of each door is airtight, of welded steel construction, and fabricated from A-36 steel.

Operation of the doors is interlocked so that only one door is open at any given time. Motor-driven units are sized so the doors will operate against a 10-psf wind and hold against a 30-psf wind. The separate drive units on the two leaves of each door operate simultaneously and provide simultaneous operation of the door leaves. Each door is controlled from a pushbutton station mounted on the wall in the vicinity of the door. The sub-door requires manual operation and will utilize a mechanical interlock system which prevents opening two doors at the same time.

Load combinations used in designing the structural portions of the doors with corresponding allowable stresses are listed in Table 12.2-33.

For all combinations, the resulting stresses do not exceed the allowable.

The rubber seals are designed for a wind load of 30-psf and a static pressure differential of 1/4-inch of water. Seals are to be inflated to an internal pressure of 8 to 10 psi.

For the mechanical portions of the doors, resulting stresses do not exceed allowable stresses. For load combinations with 30 psf winds, stresses do not exceed 90 percent of yield.

12.2.9.2.2 Safety Evaluation

The equipment access lock doors and sub-doors provide an effective air lock between the Reactor Building and outside entrance. During normal operation, the doors remain closed and at no time will more than one door be opened. While closed, the inflatable seals are kept inflated with a positive pressure. Main Control Room annunciation is provided for the inflatable seals if the sealing force is lost due to the reduced seal pressure.

The locking pins on both leaves of each door must be manually operated from the inside before a door can be opened. In addition, the two doors are electrically interlocked to provide the following conditions.

a. Only one door may be opened at any given time.

b. Before one door can be opened, the other door must be closed, locked, and sealed.

c. Doors cannot be opened until seals are deflated.

d. Seals on one door cannot be inflated until the door has been closed and locked.

e. Sub-door functions as stated in a. and b. (above) except sealing is accomplished by the use of mechanical seals.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction.

A personnel access lock is provided adjacent to the equipment access lock. This personnel lock provides an emergency access from the equipment access lock to the Reactor Building.

12.2.9.2.3 Inspection and Testing

The rubber seals are to be visually inspected on a quarterly basis, to see if any cracks or blemishes have developed. The surface of the skin plate is to be inspected annually for paint deterioration.

12.2.9.3 Flood Gate

12.2.9.3.1 Description

The equipment access flood gate is located on the outside face of the equipment access lock and is part of the Reactor Building flood protection for the Probable Maximum Flood. The gate will normally be in the open position for access into or from the equipment access lock, but may be lowered in the event of impending high water.

Hoisting and lowering of the gate are by a motorized, two-drum hoist unit equipped with means for manual operation. Control of the hoist unit is from a pushbutton station mounted in the equipment access lock near the outer pushbutton station for the lock doors. As a backup, the plant mobile crane can be used for lowering the gate.

The gate is approximately 23-feet-wide by 14-feet-high and of welded steel construction, with structural parts fabricated from A36 steel. It consists of a structural steel frame with a solid steel skin plate on one side. Rubber seals provide sealing to El. 578. The gate is guided during raising and lowering by wheels at each end, which operate in steel guides.

Power-operated, seismically-qualified dogging devices are provided as a backup for holding the gate in the open position.

Operating machinery for the gate was not seismically qualified or subject to the same quality assurance requirements as the gate, since the machinery has no function relative to flood protection.

Load combinations and corresponding allowable stresses used in designing the gate are listed in Table 12.2-43. The resulting stresses do not exceed the allowable for any load combination.

12.2.9.3.2 Safety Evaluation

This gate provides adequate protection against flooding of the Reactor Building. The gate will normally be in the open position for access into or from the equipment access lock. Gate operation from raised to lowered position takes less than 5 minutes. The gate can be lowered manually should the machinery fail to operate. Also, the plant mobile crane provides an additional means for lowering the gate. The dogging devices provide a backup for holding the gate in the open position, and provide additional protection against the remote possibility of the gate falling on a spent fuel cask. The dogs are equipped with electrical interlocks to prevent lowering the gate if the dogging devices fail to disengage properly. The closure mechanisms are part of the Plant Preventive Maintenance Program and are tested periodically. The gate is designed for static head to El. 578 and for all static heads, broken waves, and surge forces due to flood conditions ranging from mean flood levels of El. 556 to 568, with concurrent winds of 85 mph, to the probable maximum flood, with a mean flood level of El. 572.5 concurrent with 45-mph winds. Seals on the gate are the replaceable type.

All welding was done in accordance with the requirements of the American Welding Society in its Standard Code for Welding in Building Construction. All steel fabrication was in accordance with the applicable requirements of the American Institute of Steel Construction. Certified mill tests, covering chemical analyses and physical properties, are on file for all materials.

12.2.9.3.3 Inspection and Testing

After initial installation, the gate was raised and lowered sufficiently to determine that the gate seats and seals properly. Thereafter, the gate and its components are inspected periodically.

The rubber seals are to be inspected for cracks, blemishes, and wear. Lifting cables are to be inspected for wear and defects, and painted surfaces are to be inspected for paint deterioration. Lifting and lowering of the gate are to be observed for proper operation, seating, and sealing in the lowered position.

12.2.9.4 Watertight Personnel Access Door

12.2.9.4.1 Description

The watertight personnel access door is located at the south (outside) end of the personnel corridor, which is on the east side of the equipment access lock. This door is a part of the flood protection for the Reactor Building from the Probable Maximum Flood.

Physical description, design load combinations, safety evaluation, and inspection and testing criteria are the same as for the condenser cooling water system personnel access doors, as described in paragraph 12.2.7.1.2.

12.2.10 Standby Gas Treatment Building (Class I)
12.2.10.1 Concrete Structure (Figure 12.2-76)

There are two Standby Gas Treatment Buildings at Browns Ferry; Building No.1 and Building No.2. The buildings are located side-by-side adjacent to the southwest corner of the Reactor Building and lie within the earth berm surrounding the Reactor Building. They are isolated from each other and the Reactor Building by two inch wide expansion joints filled with fiber glass insulation. These structures are basically single-story, double-barreled concrete box frames with closed ends. The length of Building No. 2 is 77 feet, 10 inches, the width is 36 feet, 6 inches, and the height is about 20 feet. Building No. 1 is similar to Building No. 2 except there is an 8 feet, 10 inches by 12 feet, 2 inches recess in the northwest corner. Earth backfill surrounds all sides and covers the top of the structure in varying depths of 2 feet to 11 feet. The structure is founded on 10 ± feet of earth backfill compacted to 95 percent of maximum Standard Proctor density at optimum moisture content. The only materials to be used for backfill, as identified by the soils investigation program, are a clay sand (SC), or a lean clay (CL). Laboratory testing of soil samples has determined that the allowable soil-bearing value is 1.5 tons/ft^2. Underlying this earth backfill is a crushed stone backfill as described in paragraph 12.2.8.1.

The model used for the static analysis in the transverse direction is shown in Figure 12.2-79. This frame is analyzed by the moment distribution method using the loading conditions as given in Table 12.2-34, with lateral earth pressures determined as described in paragraph 12.2.2.10. The east and west end walls are analyzed as two-way slabs for lateral earth pressures determined in the same manner.

Stresses resulting from the static analysis are combined by the method of superposition with stresses resulting from the moments, shears, deflections, and accelerations determined by the dynamic earthquake analysis described in paragraph 12.2.10.2. Vertical accelerations of 0.067g and 0.133g are assumed to act simultaneously with the horizontal accelerations for the 0.1g Operating Basis

Earthquake and 0.2g Design Basis Earthquake, respectively. Members are proportioned by the working stress method of ACI Code 318-63, so that the above stresses do not exceed the allowable stresses given in Table 12.2-34.

12.2.10.2 Dynamic Earthquake Analysis

The dynamic earthquake analysis of the Standby Gas Treatment Buildings were done in the same manner as the Diesel Generator Buildings. Two analyses were performed for the two buildings. The results for both buildings were enveloped to produce a single set of data to be used in subsequent analyses of either building.

Each mathematical model consisted of two lumped masses, one at the center of the roof slab and one at the center of the base slab. The soil structure interaction was modeled by a translational spring K_T, rotational spring, K_R and vertical spring K_V. The values of K_T K_R and K_V were obtained from the values computed for the Diesel Generator Buildings (see paragraph 12.2.8.3), using the relations:

$$K_T = C_t A$$
$$K_R = C_\phi I$$
$$K_V = C_V A$$

Where C_t, C_ϕ and C_V are coefficients of the soil, and A and I are the area and moments of inertia of the slabs contact area, The spring constants were calculated for each building from:

$$K_T \text{ SGT BLDG} = \frac{A_{SGT\ BLDG}}{A_{DG\ BLDG}} \times K_T \text{ DG BLDG}$$

$$K_R \text{ SGT BLDG} = \frac{I_{SGT\ BLDG}}{I_{DG\ BLDG}} \times K_R \text{ DG BLDG}$$

AND

$$K_V \text{ SGT BLDG} = \frac{A_{SGT\ BLDG}}{A_{DG\ BLDG}} \times K_V \text{ DG BLDG}$$

The values of K_T, K_R, K_V and masses of roof and base used in the analysis is given in Table 12.2-35. Since the structures are supported on a soil deposit, the DBE and OBE accelerations at the top of rock are amplified to the foundation level using the method described in Section 12.2.8.3. The amplification factors used are 1.6 and 1.2 for the horizontal and vertical motions, respectively.

The analyses were performed using the normal mode time history method. The vertical motion of the structure due to horizontal input earthquake motion was included in the analyses. A structural damping value of 5 percent of critical damping was used for all modes. The results of the analyses are shown in Table 12.2-36. The lateral restraint of the soil along the sides of the buildings was neglected as far as retarding the motion of the buildings (the increased lateral pressure is used in analyzing stresses, however). The mass used in the analysis includes the weight of the earth on top of the building roof slab and all equipment.

Analyses as described above were performed using both the E1 Centro earthquake and artificial earthquake time histories input ground motion described in Section 2.5.4. Results of the analyses using the E1 Centro earthquake input were used in the design of the buildings. Results from the analyses using the artificial earthquake input were used in the analyses and design of subsystems housed in the building. ARS were generated at frequencies ranging from 0.2 Hz to 20 Hz at intervals consistent with those given in the Standard Review Plan. The ARS were peak broadened $\pm$ 15 percent.

12.2.11 (Deleted)

12.2.12 Guardhouse (Gatehouse) (Class II)

This is a conventional-type structure consisting of concrete slab and walls below grade and concrete slab at grade. Above grade, the building is framed by structural steel and enclosed by architectural panels.

12.2.13 Diesel Generator Building, Unit 3 (Class I)

12.2.13.1 Concrete Structure (Figure 12.2-81)

This structure is located adjacent to the east side of the Reactor Building as shown in Figure 2.2-4. It is separated from the Reactor Building by a 2-inch expansion joint filled with Fiberglas insulation. The north and south ends of the building face an earthfill which slopes in the east-west direction to a height of 30 feet above the lower floor level at the reactor building. The east side of the building is exposed.

The foundation for the Unit 3 structure is specified to be the same as that for the Units 1 and 2 structure as described in paragraph 12.2.8.1.

The structure is a two-story concrete box with two longitudinal dividing walls full length and height, and four main transverse dividing walls full height terminating at the east longitudinal wall. Each of the four bays encloses a diesel generator and its auxiliary equipment. A raised grated air intake and exhaust plenum over each generator bay provides for air intake and exhaust. The grade floor slab of each

generator bay consists of diesel fuel storage tanks encased in concrete. The west longitudinal wall separates the diesel generator portion of the building from the shutdown board rooms and contains no doors or any open penetrations. A corridor extending west to east on the roof of the building provides access to the shutdown board room area and the diesel generator area from the Reactor Building. Separate stairways lead from the corridor to the shutdown board area and diesel generator area. Mechanical equipment rooms located on the roof of the board room provide for heating and ventilating of the board room.

The description of the analysis of the structure is as given in paragraph 12.2.8.1. The diesel generator portion of the Unit 3 structure is of the same configuration as the Units 1 and 2 structure, and the designs performed in 1968 using the ACI Code 318-63 are applicable to this portion of the Unit 3 structure. Designs were made in 1972 for the shutdown board rooms and are in accordance with the alternate design method of ACI 318-71.

The Diesel Generator Building lower floor is at El. 565.5. The building is protected against flood-water and wave action and kept dry to El. 578. The emergency drain lines in the Diesel Generator Building are isolated in the event of a probable maximum flood using isolation valves. In the event that the valves are inoperable, drain plugs are installed to provide a temporary alternate means of isolation. There are five sets of large double doors in the east wall of the building for both personnel and maintenance access. These doors are designed for all static heads, broken waves, and surge forces up to El. 578 with replaceable seals which are always in place. The electrical boards are located in rooms in the building conforming to the appropriate separation criteria required. If a diesel compartment must be blocked open for maintenance, provisions are made to protect the remainder of the building.

12.2.13.2 Access Doors

The access doors for the Unit 3 building are identical to those described in paragraph 12.2.8.2 for the Units 1 and 2 building.

12.2.13.3 Dynamic Earthquake Analysis

The results of the analysis described in paragraph 12.2.8.3 for the Units 1 and 2 structure are accepted as being applicable to the Unit 3 structure.

The foundation material for the two structures is the same and is as described in paragraph 12.2.8.1. As can be seen by a comparison of Figures 12.2-76 and 12.2-81, the two structures are of the same configuration, with the exception that the Unit 3 structure is wider in the east-west direction.

The predominant motion of the Units 1 and 2 structure is rocking and translation of the base. The translatory motion at the base is more significant, since the majority

of the structure's mass is concentrated at the base. Therefore, the response of the structure depends mainly on the horizontal soil spring, K_T, and the mass of the structure. The effect of the increased width for the Unit 3 structure is twofold. First, the stiffness of the soil springs is increased, lowering the natural period of the structure and reducing the response of the structure, as shown by a comparison of the north-south and east-west direction responses for Units 1 and 2 structure. Second, the mass of the structure increases, tending to increase the response and the natural period of the structure. The soil spring, K_T, for the Unit 3 structure increases approximately 15 percent, while the mass of the structure increases only about 10 percent. The effect of the increased soil springs exceeds the effect of the increased mass, resulting in a reduction of the total responses. However, as stated above, the results determined for the Units 1 and 2 (original '2D' model) structure are applicable to, and are used for, the design of the Unit 3 structure.

12.2.14 Offgas Treatment Building (Class I)

Concrete Structure

The offgas treatment structure is a rectangular box-type concrete structure completely covered with an average of 10 feet of backfill. The structure is located between the stack and centerline of reactors (column line r). See general outline features (Figure 12.2-82). The access into the building is a covered stairway at one end and a spiral stairway at the other for emergency exit. The interior members are comprised predominantly of thick walls and slabs, the dimensions of which are determined by shielding requirements.

The building is sealed against flood to the elevation of the stairway entrance at EL. 568.0. PVC seals are provided in all construction joints in the exterior walls, roof, and base slabs to prevent leakage through the joint.

The structure is designed for water pressure, earth pressure with surcharge, equipment loads, and Design Basis Earthquake. The structure is designed to remain in the elastic range for the Design Basis Earthquake and remain waterproof. The roof is not designed to withstand the load caused from the collapse of the reinforced concrete chimney due to a tornado. The exterior walls and bottom slab are designed to remain in the elastic range, assuming the roof slab has collapsed, and they will not permit water leakage into or out of the building below EL. 566.25.

For loads and load combination with appropriate load factors used, see Table 12.2-44.

The base slab is poured on sound rock or a concrete subpour taken to sound rock. Reinforcing bars grouted into 3-inch-diameter holes are provided where required to resist the net uplift due to hydrostatic pressure. The exterior walls, acting as cantilever walls after the roof and interior slabs collapse, are anchored into rock with

reinforcing bars to resist the maximum moment which they have to resist. The interior walls are load bearing, but are separated from the exterior walls and base slab by a contraction joint, so the deformation or collapse of the top slab and interior member will not compromise the structural integrity of the exterior walls and base slab. Shear keys are provided in the contraction joints to resist horizontal loads.

The condition of rapid depressurization during a tornado is provided for in the following way. The latches on the two exterior doors will fail, allowing the interior to be vented; so the interior is designed for a negative pressure of 240 psf.

The structure is founded on rock and is rigid; hence there is no amplification of the motion through the structure. However, since the structure is buried, the roof slab is conservatively considered to have the same horizontal motion as the surrounding soil. The amplification factors used are 1.6 in the horizontal direction and 1.3 in the vertical direction using the method described in Section 12.2.8.3 The structure is designed for the increase in soil pressure on the structure from the horizontal and vertical soil accelerations. Subsystems housed in the Offgas Treatment Building are designed for the site design spectrum multiplied by the soil amplification factor.

12.2.15 Radwaste Evaporator Building (Class II)

Concrete Structure

The Radwaste Evaporator Building is a box-type concrete structure extending from grade to approximately 30 feet above grade, and is supported by steel H-piles driven to bedrock. It is located adjacent to the west side of the Radwaste Building as shown in Figure 12.2-83. The structure is separated from the radwaste building by a 1-l/2-inch expansion joint.

The building is comprised predominantly of thick walls and slabs, the dimensions of which are primarily determined by shielding requirements. About two-thirds of the roof system is a steel-framed structure supported on concrete wall brackets. The remainder of the roof system is a thick concrete slab required for shielding over the area that houses the radwaste evaporator equipment.

The structure is designed using the alternate design method in Section 8.10 of ACI 318-71 for normal design loads, with higher stresses allowed for load cases as shown in Table 12.2-45. Moments and shears for frame and one-way slabs are designed by the moment distribution method. Two-way slabs are designed using the publication, Moments and Reactions for Rectangular Plates, by W. T. Moody, also known as Bureau of Reclamation Engineering Monograph No. 27. The structure is designed using the load, loading conditions, and allowable stresses as shown in Table 12.2-45. The total horizontal force located at the top of the pile group is resisted by the passive earth pressure exerted by the soil against individual piles. Earth pressure and pile designs are based on Soil Mechanics in Engineering

Practice, by Terzaghi and Peck; Foundations of Structures, by Dunham; and Steel HPiles, by U.S. Steel Corporation.

The minimum percentage of reinforcing in the slabs is 0.0015bt in the top face and 0.0018bt in the bottom face.

The structure is designed to be flood-proof for the Maximum Possible Flood (water level EL. 572.5) plus waves. The construction interface between the structure and the radwaste building is sealed for flood protection to EL. 578.0.

12.2.16 Residual Heat Removal Service Water Intake (Class I)

The Residual Heat Removal Service Water (RHRSW) pumps on the deck of the intake structure are surrounded by concrete walls which extend from the deck elevation to a minimum elevation of 578. This protective structure provides permanent, in-place protection to EL. 578 and requires no operator action for its implementation. These pumps serve both the RHRSW system and the Emergency Equipment Cooling Water (EECW) System.

The walls are reinforced, poured-in-place concrete, including three inner partitions which provide four individual pump compartments. The walls are designed for static water pressure with water level to EL. 578, or for the dynamic force resulting from a wave whose height is 5.1 feet with reservoir water level at EL. 572.5, or for a wave whose height is 10 feet with reservoir water level at EL. 568. For the design of the walls, it is assumed that the wave strikes the wall with the force that would occur if the wall were located at the front face of the intake structure. The walls are also designed to withstand the Design Basis Earthquake (0.20g) or tornado winds to prevent damage to the enclosed equipment.

The outer and inner compartment walls, as erected, form an effective water barrier. All pipe or instrument openings are sleeved through the outer and inner walls and are sealed. The protective structure is open at the top to the weather, and access to each individual compartment is by a watertight door which will be normally closed. Each compartment presently contains three RHRSW pumps and their immediate piping and valving, an EECW strainer, one or more condenser circulating water pump valve access hatch(es), and a deck opening for a future RHRSW pump; each of the two central compartments contains a vent hold.

12.2.17 Low Level Radioactive Waste Storage Facility (LLRWSF)

12.2.17.1 Description

The LLRWSF is a group of rectangular box-type concrete storage modules designed to store Low Level Radioactive Waste (LLRW) resins or trash. The onsite storage facility consists of a minimum of four concrete modules placed on either in-situ soil

or compacted fill and sized to provide for storage of non-volume reduced LLRW generated at the plant. The resin storage modules and trash storage modules shall be above ground, safety-related structures constructed of reinforced concrete. Each storage module shall have five individual compartments. Each compartment shall be composed of five unit cells. All storage modules shall be designed to withstand the design basis events and shall be designed and constructed for the normal loads, severe environmental loads, and extreme environmental loads.

For these concrete structures, the ultimate strength design (USD) method will be used based on ACI 318-77, “Building Code Requirements for Reinforced Concrete.” The required section strength used in the design is the maximum value among the several values determined for the required loading combinations of ACI 318-77.

Details of the LLRWSF location and configuration of storage modules are provided in Figure 2.5-S1.

12.2.18 Independent Spent Fuel Storage Installation (ISFSI) PAD

12.2.18.1 Description

The ISFSI pad is part of the ISFSI facility and has been designed to be compliant with 10 CFR 72 federal regulations. The pad is located at the northeast side of the plant and adjacent to the switchyard as shown on Figure 2.2-4. The ISFSI pad is a reinforced concrete slab with a nominal thickness of 24 inches. The pad is divided by construction joints into four sections. There are two 98 feet by 68 feet sections which comprise the ends of the pad and two 96 feet by 68 feet sections which comprise the middle sections of the pad for an overall length of 388 feet and an overall width of 68 feet. Each pad section is designed to support up to 24 spent fuel storage casks for a total of 96 casks for the entire ISFSI pad.