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U. S. Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: T. R. QUAY

SUBJECT: RESPONSE TO RAIs RELATED TO AP600 STRUCTURAL MODULE DESIGN

Dear Mr. Quay:

The attached documents summarizes Westinghouse responses to issues in an NRC letter dated July 1, 1996. This response includes a markup of subsections 3.8.3 and 3.8.4 of the AP600 SSAR. These responses are in preparation for the meeting and design review on AP600 structural modules with the NRC staff scheduled for January 14, 15, and 16, 1997. It is expected that this review of this submittal will facilitate resolution and closure of the issues. Westinghouse will be prepared to discuss these issues during the meting.

If you have any questions please contact D. A. Lindgren at (412) 374-4856.

Brian A. McIntyre

Brian A. McIntyre, Manager
Advanced Plant Safety and Licensing

/jml

attachments

cc: T. Kenyon - NRC (w/attachment)
T. Cheng - NRC (w/attachment)
J. Braverman - BNL (w/attachment)

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The following summarizes Westinghouse responses to issues in the NRC letter dated July 1, 1996. Some issues are addressed by proposed changes in the SSAR. Others will be addresssed by NRC audit of the AP600 design.

1. SSAR revision shows use of ACI 349 and has deleted the use of AASHTO.

2. GW-SUP-005 evaluates effect of cracking on overall response. No impact on west side of CIS.

3. Case 4 module stiffness deleted from SAR revision.

4. Thermal loadings revised in SSAR. Documented in GW-SUP-006.

5. Description of liners expanded in paragraphs 3.8.3-1 and 3.8.3.5-6.

6. Reference to 3.7.3 added in subsection 3.8.3.5.5

7. N, F, H revised.

8. Reference added to ACI 347.

9. Paragraph revised to show stiffness assumptions and table added summarizing analyses.

10. Case 4 deleted and table added summarizing analyses.

11. Last paragraph of 3.8.3.2 and 3.8.4.2 revised to confirm inspection activities to AISC N-690.

12. Shell elements included in SSAR

13. Reference added to AISC N-690.

14.1 Reference to principal stresses deleted. Revised to follow ACI 349 methods.

14.2 Use of 3 S_{m1} has been clarified.

15. Final design will not use SAP90 results.

Attachment 2

1. Liners. See previous Item 5.

2. Drawings will be available during audit

3. Primarily piping or valves

4. Same as previous Item 11.

5. Loads. See previous Item 7.

6. Loads. See previous Item 7.

7. See previous Item 4.

8. Subsection 3.8.3.3.1 clarified

9. See subsection 3.8.3.3.2 revision.

10. See previous Item 9.

11. See previous Item 10

12. ADS_2 is less limiting - statement added in SSAR.

13. Documentation of analyses is available for audit.

14. Documentation of analyses is available for audit.

15. Documentation of analyses is available for audit..

16. Out-of-plane Shear. See previous Item 1.

17. Combining stresses in faceplates. See previous Item 14(1).

18. Trusses designed to AISC N-690.

19. Studs. See previous Item 13.

20. See revision on figure

21. Drawings will be available during audit

22. There are no pressures due to pipe break on modules outside containment.

23. Angle is about 35 degrees, but criteria does not use an angle but only a spacing criterion.

24. Reference added to ACI 347.

25. Separate response sent to NRC.

3.8.3 Concrete and Steel Internal Structures of Steel Containment

3.8.3.1 Description of the Containment Internal Structures

The containment internal structures are those concrete and steel structures inside (not part of) the containment pressure boundary that support the reactor coolant system components and related piping systems and equipment. The concrete and steel structures also provide radiation shielding. The containment internal structures are shown on the general arrangement drawings in Section 1.2. The containment internal structures consist of the primary shield wall, reactor cavity, secondary shield walls, in-containment refueling water storage tank (IRWST), refueling cavity walls, operating floor, intermediate floors, and various platforms. The polar crane girders are considered part of the containment vessel. They are described in subsection 3.8.2.

Component supports are those steel members designed to transmit loads from the reactor coolant system to the load-carrying building structures. The component configuration is described in this subsection including the local building structure backing up the component support. The design and construction of the component supports are described in subsection 5.4.10.

The containment internal structures are designed using reinforced concrete and structural steel. At the lower elevations conventional concrete and reinforcing steel are used, except that permanent steel forms are used in some areas in lieu of removable forms based on constructibility considerations. These steel form modules (liners) consist of plate reinforced with angle stiffeners and tee sections, as shown in Figure 3.8.3-16. The angles and the tee sections are on the concrete side of the plate. Welded studs, or similar embedded steel elements, are attached on the concrete face of the permanent steel form where surface attachments transfer loads into the concrete. Where these surface attachments are seismic Category I, the portion of the steel form module transferring the load into the concrete is classified as seismic Category I.

Above elevation 98′, the structure, walls, and some floors are structural modules. Figure 3.8.3-1 shows the location of the structural modules. Figures 3.8.3-2 and 3.8.3-3 show the typical structural configuration of the wall and floor modules, respectively. These structural modules are structural elements built up with welded steel structural shapes and plates. Concrete is used where required for shielding, but reinforcing steel is not normally used.

Walls and floors exposed to water during normal operation or refueling are constructed using stainless steel plates.

3.8.3.1.1 Reactor Coolant Loop Supports

3.8.3.1.1.1 Reactor Vessel Support System

The reactor vessel is supported by four supports located under the cold legs, which are spaced 90 degrees apart in the primary shield wall. The supports are designed to provide for radial thermal growth of the reactor coolant system, including the reactor vessel, but they prevent the vessel from lateral and torsional movement. The loads are carried by the reactor vessel supports through embedded steel weldments to the primary shield concrete. Figure 3.8.3-4 shows the reactor vessel supports.

3.8.3.1.1.2 Steam Generator Support System

The steam generator vertical support consists of a single vertical column extending from the steam generator compartment floor to the bottom of the steam generator channel head. The column is constructed of heavy plate sections and is pinned at both ends to permit unrestricted radial displacement of the steam generator during plant heatup and cooldown. The location of this column is such that it will allow full access to the steam generator for routine maintenance activities. It is located a sufficient distance away from the reactor coolant pump motors to permit pump maintenance and in-service inspection.

The lower steam generator horizontal support is located at the bottom of the channel head. It consists of a tension/compression strut oriented approximately perpendicular to the hot leg. The strut is pinned at both the wall bracket and the generator channel head to permit movement of the generator during plant heatup and cooldown.

The upper steam generator horizontal support is located on the lower shell just below the transition cone. It consists of two large hydraulic snubbers oriented parallel with the hot leg centerline and two rigid compression-only bumpers (one on each side of the generator) oriented perpendicular to the hot leg. The hydraulic snubbers are valved to permit steam generator movement for thermal transition conditions, and to "lock-up" and act as rigid struts under dynamic loads. The two rigid bumpers are mounted on the steam generator compartment wall at the elevation of the operating deck, and are shimmed to a nominal zero gap at the plant normal operating temperature. The steam generator loads are transferred to the bumpers and snubbers through a ring girder surrounding the generator shell. Figure 3.8.3-5 shows the steam generator supports.

3.8.3.1.1.3 Reactor Coolant Pump Support System

Because the reactor coolant pumps are integrated into the steam generator channel head, they do not have individual supports. They are supported by the steam generators.

3.8.3.1.1.4 Pressurizer Support System

The pressurizer is supported by four columns mounted from the pressurizer compartment floor. A lateral support is provided at the top of the columns. This lateral support consists of eight struts connecting it to the pressurizer compartment walls. A lateral support is also provided on the upper portion of the pressurizer. This lateral support consists of a ring girder around the pressurizer and eight struts connecting it to the pressurizer compartment walls. Figure 3.8.3-6 shows the pressurizer supports.

3.8.3.1.2 Containment Internal Structures Basemat

The containment internal structures basemat is the reinforced concrete structure filling the bottom head of the containment vessel. It extends from the bottom of the containment vessel head at elevation 66′-6″ up to the bottom of the structural modules that start between elevations 83′-0″ and 103′-0″. The basemat includes rooms as shown on Figure 1.2-5. The primary shield wall and reactor cavity extend from elevation 71′-6″ to elevation 107′-2″. They provide support for the reactor vessel and portions of the secondary shield walls and refueling cavity walls. The general arrangement drawings in Section 1.2 show the location and configuration of the primary shield wall and reactor cavity. Above elevation 98′ the walls are structural modules. Below elevation 98′ they are reinforced concrete. The reinforced concrete portions are shown in Figure 3.8.3-7.

3.8.3.1.3 Structural Wall Modules

Structural wall modules are used for the secondary shield walls around the steam generators and pressurizer, for the east side of the in-containment refueling water storage tank, and for the refueling cavity. The general arrangement drawings in Section 1.2 show the location and configuration. Locations of the structural modules are shown in Figure 3.8.3-1. The secondary shield ~~is~~ walls are a series of walls that, together with the refueling cavity wall, enclose the steam generators. Each of the two secondary shield wall compartments provides support and houses a steam generator and reactor coolant loop piping. The in-containment refueling water storage tank is approximately 30 feet high. The floor elevation of this tank is 103′-0″. The tank extends up to about elevation 133′-3″, directly below the operating deck. On the west side, along the containment vessel wall, the tank wall consists of a stainless steel plate stiffened with structural steel sections in the vertical direction and angles in the horizontal direction. Structural steel modules, filled with concrete and forming, in part the refueling cavity, steam generator compartment, and pressurizer compartment walls, compose the east wall. The refueling cavity has two floor elevations. The area around the reactor vessel flange is at elevation 107′-2″. The lower level is at elevation 98′-1″. The upper and lower reactor internals storage is at the lower elevation, as is the fuel transfer tube. The center line of the fuel transfer tube is at elevation 100′-8.75″.

Structural wall modules consist of steel faceplates connected by steel trusses. The primary purpose of the trusses is to stiffen and hold together the faceplates during handling, erection, and concrete placement. The nominal thickness of the steel faceplates is 1/2 inch. The nominal spacing of the trusses is 30 inches. Shear studs are welded to the inside faces of the steel faceplates. The structural wall modules are anchored to the concrete base by reinforcing steel dowels or other types of connections embedded in the reinforced concrete below. After erection, concrete is placed between the faceplates. Typical details of the structural modules are shown in Figures 3.8.3-2 and 3.8.3-8.

3.8.3.1.4 Structural Floor Modules

Structural floor modules are used for the operating floor at elevation 135′-3″ over the in-containment refueling water storage tank and for the 107′-2″ floor over the rooms in the containment internal structures basemat. The floors are shown on the general arrangement drawings in Section 1.2. The 107′-2″ floors and the floor above the in-containment refueling water storage tank consist of steel tee and wide flange sections, welded to horizontal steel bottom plates stiffened by transverse stiffeners. After erection, concrete is placed on top of the horizontal plate and around the structural steel section. The remaining region of the operating floor consists of a concrete slab, placed on Q decking supported by structural steel beams. The operating floor is supported by the in-containment refueling water storage tank walls, refueling cavity walls, the secondary shield walls, and steel columns originating at elevation 107′-2″. Structural details of the operating floor structural module are shown in Figure 3.8.3-3.

3.8.3.1.5 Internal Steel Framing

The region of the operating floor away from the in-containment refueling water storage tank consists of a concrete slab, placed on Q decking supported by structural steel beams. The floor at elevation 118′-6″ consists of steel grating supported by structural steel framing. In addition, a number of steel platforms are located above and below the operating floor. These platforms support either grating floors or equipment, such as piping and valves.

3.8.3.2 Applicable Codes, Standards, and Specifications

The following documents are applicable to the design, materials, fabrication, construction, inspection, or testing of the containment internal structures:

- American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, ACI-318-89 (refer to subsection 3.8.4.4.1 for applicability)
- American Concrete Institute (ACI), Code Requirements for Nuclear Safety Related Structures, ACI-349-90 (refer to subsection 3.8.4.5 for supplemental requirements)
- American Concrete Institute (ACI), Manual of Standard Practice for Detailing Reinforced Concrete Structures, ACI-315-88
- American Concrete Institute (ACI), Standard Specification for Tolerances for Concrete Construction and Materials, ACI-117-90
- American Concrete Institute (ACI), Guide to Formwork for Concrete, ACI-347-94
- American Institute of Steel Construction (AISC), Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690-1984 (refer to subsection 3.8.4.5 for supplemental requirements)
- American Welding Society (AWS), Structural Welding Code, AWS D 1.1-90
- American Welding Society (AWS), Reinforcing Steel Welding Code, AWS D 1.4-89

- National Construction Issues Group (NCIG), Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants, NCIG-01, Revision 2, May 7, 1985

- ~~American Association of State Highway and Transportation Officials (AASHTO), Bridge Design Specifications, 1994.~~

Nationally recognized industry standards, such as American Society for Testing and Materials, American Concrete Institute, and American Iron and Steel Institute, are used to specify material properties, testing procedures, fabrication, and construction methods. Section 1.9 describes conformance with the Regulatory Guides.

Welding and inspection activities for seismic Category I structural steel, including building structures, structural modules, cable tray supports, and heating, ventilating and air-conditioning (HVAC) duct supports are accomplished in accordance with written procedures and meet the requirements of the American Institute of Steel Construction (AISC N-690). The weld acceptance criteria is as defined in NCIG-01, Revision 2. The welded seams of the plates forming part of the leaktight boundary of the in-containment refueling water storage tank are examined by liquid penetrant and vacuum box after fabrication to confirm that the boundary does not leak.

3.8.3.3 Loads and Load Combinations

The loads and load combinations for the containment internal structures are the same as for other Category I structures described in subsection 3.8.4.3 and the associated tables, except for the following modifications:

Wind loads (W), tornado loads (W_t), and precipitation loads (N) are not applicable to the design of the containment internal structures because of the protection provided by the steel containment. Therefore, these loading terms have been excluded in the load combinations for the containment internal structures.

3.8.3.3.1 Passive Core Cooling System Loads

Structures are evaluated for pressure and thermal transients associated with operation of the passive core cooling system. The effects of temperatures higher than 100°F on the modulus of elasticity and yield strength of steel are considered.

The passive core cooling system and the automatic depressurization system (ADS) are described in Section 6.3. The automatic depressurization system is in part composed of two spargers that are submerged in the in-containment refueling water storage tank. The spargers provide a controlled distribution of steam flow to prevent imposing excessive dynamic loads on the tank structures. Capped vent pipes are installed in the roof of the tank on the side near the containment wall. These caps prevent debris from entering the tank from the containment operating deck, but they open under slight pressurization of the in-containment refueling water storage tank. This provides a path to vent steam released by the spargers. An overflow is provided from the in-containment refueling water storage tank to the refueling cavity to accommodate volume and mass increases during automatic depressurization system operation. Two sets of loads representing bounding operational or inadvertent transients are considered in the design of the in-containment refueling water storage tank.

- ADS_1 - This automatic depressurization system load is associated with blowdown of the primary system through the spargers when the water in the in-containment refueling water storage tank is cold and the tank is at ambient pressure. Dynamic loads on the in-containment refueling water storage tank due to automatic depressurization system operation are determined using the results from the automatic depressurization system hydraulic test as described in subsection 3.8.3.4.3. This automatic depressurization system transient is of short duration such that the concrete walls do not heat up significantly. It is combined with ambient thermal conditions. Long-term heating of the tank is bounded by the design for the ADS_2 load.

- ADS_2 - This automatic depressurization system transient considers heatup of the water in the in-containment refueling water storage tank. This may be due to prolonged operation of the passive residual heat removal heat exchanger or due to an automatic depressurization system discharge. Prolonged operation of the passive residual heat removal heat exchanger raises the water temperature from an ambient temperature of 120°F to saturation in about 2 hours, increasing to 240°F within about 5.5 hours. Steaming to the containment atmosphere initiates once the water reaches its saturation temperature. For structural design an extreme transient is defined starting at 50°F since this maximizes the temperature gradient across the concrete filled structural module walls. The water temperature rises from an ambient temperature of 50°F to saturation in 4 hours, increasing to 240°F at 6 hours. The containment atmosphere rises from 50°F at 4 hours to 240°F at 6 hours. Blowdown of the primary system through the spargers may occur during this transient and occurs prior to 24 hours after the initiation of the event. Since the flow through the sparger cannot fully condense in the saturated conditions, the pressure increases in the in-containment refueling water storage tank and steam is vented through the in-containment refueling water storage tank roof. The in-containment refueling water storage tank is designed for an equivalent static internal pressure of 5 psi in addition to the hydrostatic pressure occurring at any time up to 24 hours after the initiation of the event.

The ADS_1 and ADS_2 loads are considered as live loads. The dynamic ADS_1 load is combined with the safe shutdown earthquake by the square root sum of the squares (SRSS). The static ADS_2 load is combined with the safe shutdown earthquake by absolute sum.

3.8.3.3.2 Concrete Placement Loads

The steel faceplates of the structural wall modules, designed for the hydrostatic pressure of the concrete, act as concrete forms. The concrete placement loads are 1050 lbs per square foot determined in accordance with ACI-347. The bending stress in the faceplate due to theis hydrostatic pressure of the concrete is approximately 13 ksi, based on the assumption of a continuous faceplate, or 20 ksi based on the assumption of simple spans. The minimum yield strength of material for the faceplates is 36 ksi for A36 steel. Thise stress is well below the allowable, since the plate is designed to limit the out-of-plane deflection. After the concrete has gained strength~~, the steel plate is not required for support, and the full steel plate is available to carry other loads.~~ these stresses remain in the steel; however, since the average residual stress is zero and since the concrete no longer requires hydrostatic support, the ultimate strength of the composite section is not affected, and the full steel plate is available to carry other loads as described below.

The steel plates and the concrete act as a composite section after the concrete has reached sufficient strength. The composite section resists bending moment by one face resisting tension and the other face resisting compression. The steel plate resists the tension and behaves as reinforcing steel in reinforced concrete. The composite section is underreinforced so that the steel would yield before the concrete reaches its strain limit of 0.003 in/in. As the steel faceplates are strained beyond yield to allow the composite section to attain its ultimate capacity, the modest residual bending stress from concrete placement is relieved, since the stress across the entire faceplate in tension is at yield. The small residual strain induced by the concrete placement loads is secondary and has negligible effect on the ultimate bending capacity of the composite section. The stresses in the faceplates resulting from concrete placement are therefore not combined with the stresses in the post-construction load combinations.

3.8.3.4 Analysis Procedures

This subsection describes the modelling and overall analyses of the containment internal structures, including the concrete-filled structural modules. Concrete and steel composite structures are used extensively in conventional construction. Applications include concrete slabs on steel beams and concrete-filled steel columns. Testing of concrete-filled structural modules is described in References 27 through 29 for in-plane loading and in References 30 through 33 for out-of-plane loading. The tests indicate that these composite structures behave in a manner similar to reinforced concrete structures. The initial load deflection behavior is well predicted using the gross properties of the steel and concrete. This is similar to the behavior of reinforced concrete elements where the initial stiffness is predicted by the gross properties. As the load is increased on reinforced concrete members, cracking of the concrete occurs and the stiffness decreases. The behavior of concrete and steel composite structures is similar in its trends to reinforced concrete but has a superior performance. The results of the test program by Akiyama et al. (Reference 27) indicate that concrete and steel composites similar to the structural modules have significant advantages over reinforced concrete elements of equivalent thickness and reinforcement ratios:

- Over 50 percent higher ultimate load carrying capacity
- Three times higher ductility
- Less stiffness degradation under peak cyclic loads, 30 percent for concrete and steel composites versus 65 percent for reinforced concrete

Methods of analysis for the structural modules are similar to the methods used for reinforced concrete. Table 3.8.3-2 summarizes the finite element analyses of the containment internal structures and identifies the purpose of each analysis and the stiffness assumptions for the concrete filled steel modules. For static loads ~~member forces are dependent on the relative stiffnesses and~~ the analyses use the monolithic (uncracked) stiffness of each concrete element. For thermal and dynamic loads ~~member forces are dependent on both the absolute and the relative magnitudes of stiffness. These~~ the analyses consider the extent of concrete cracking as described in later subsections. Stiffnesses are established based on analyses of the behavior and review of the test data related to concrete-filled structural modules. The stiffnesses directly affect the member forces resulting from restraint of thermal growth. The in-plane shear stiffness of the module influences the fundamental horizontal natural frequencies of the containment internal structures in the nuclear island seismic analyses described in subsection 3.7.2. The out-of-plane flexural stiffness of the module influences the local wall frequencies in the seismic and hydrodynamic analyses of the in-containment refueling water storage tank. Member forces are evaluated against the strength of the section calculated as a reinforced concrete section with zero strength assigned to the concrete in tension.

ACI 349, Section 9.5.2.3 specifies an effective moment of inertia for calculating the deflection of reinforced concrete beams. For loads less than the cracking moment, the moment of inertia is the gross (uncracked) inertia of the section. The cracking moment is specified as the moment corresponding to a maximum flexural tensile stress of $7.5\sqrt{f_c'}$. For large loads, the moment of inertia is that of the cracked section transformed to concrete. The effective moment of inertia provides a transition between these two dependent on the ratio of the cracking moment to the maximum moment in the beam at the stage the deflection is to be computed.

Table 3.8.3-1 summarizes in-plane shear and out-of-plane flexural stiffness properties of the 48-inch and 30-inch walls based on a series of different assumptions. The stiffnesses are expressed for unit length and height of each wall. The ratio of the stiffness to the stiffness of the monolithic case is also shown.

- Case 1 assumes monolithic behavior of the steel plate and uncracked concrete. This stiffness is supported by the test data described in References 27 through 33 for loading that does not cause significant cracking. This stiffness is the basis for the stiffness of the concrete-filled steel module walls in the nuclear island seismic analyses and in the uncracked case for the hydrodynamic analyses.

- Case 2 considers the full thickness of the wall as uncracked concrete. This stiffness value is shown for comparison purposes. It is applicable for loads that do not result in significant cracking of the concrete and is the basis for the stiffness of the reinforced concrete walls in the nuclear island seismic analyses. This stiffness was used in the harmonic analyses of the internal structures described in subsection 3.8.3.4.2.2.

- Case 3 assumes that the concrete in tension has no stiffness. For the flexural stiffness this is the conventional stiffness value used in working stress design of reinforced concrete sections. For in-plane shear stiffness, a 45-degree diagonal concrete compression strut is assumed with tensile loads carried only by the steel plate. The in-plane stiffnesses calculated by these assumptions are lower than the stiffnesses measured in the tests described in References 27 through 29 for loading that causes cracking.

- ~~Case 4 considers concrete tensile stiffness of the concrete between cracks. The crack spacing is 15 inches based on estimates of the crack spacing for the walls of the in-containment refueling water storage tank subject to thermal transients that may crack the concrete. It also considers the flexibility of the shear connectors between the steel plate and the concrete. This case shows similar magnitudes of stiffness for closely spaced cracks, considering both concrete tensile stiffness and the flexibility of the shear connectors between the steel plate and the concrete.~~

3.8.3.4.1 Seismic Analyses

3.8.3.4.1.1 Finite Element Model

The three-dimensional (3D) lumped-mass stick model of the containment internal structure is developed based on the structural properties obtained from a 3D finite element model. The structural modules are simulated within the finite element model using 3D shell elements. Equivalent shell element thickness and modulus of elasticity of the structural modules are computed as shown below. The shell element properties are computed using the combined gross concrete section and the transformed steel faceplates of the structural modules. This representation models the composite behavior of the steel and concrete.

- Axial and Shear Stiffnesses of module:

$$\sum E\,A = E_c\,(L\,t + 2\,(n-1)\,L\,t_s)$$

- Bending Stiffness of module:

$$\sum E\,I = E_c\left[\frac{L}{12}\,t^3 + 2\,\frac{L}{12}\,(n-1)\,L\,t_s^3 + 2\,(n-1)\,L\,t_s\left(\frac{t}{2}\right)^2\right]$$

where:

E_c = concrete modulus of elasticity
n = modular ratio of steel to concrete
~~l~~ L = length of wall module
t = thickness of wall module
t_s = thickness of plate on each face of wall module

These equations lead to an equivalent thickness, t_m, and modulus of elasticity of the plate elements, E_m, as shown below:

$$t_m = \left[\frac{1 + 3\,\alpha\,(n-1)}{1 + \alpha\,(n-1)} \right]^{1/2} t$$

$$E_m = [\,1 + \alpha\,(n-1)\,] \left[\frac{1 + 3\,\alpha\,(n-1)}{1 + \alpha\,(n-1)} \right]^{-1/2} E_c$$

where $\alpha = 2t_s / t$ and terms of order α^3 are neglected (for a typical 30-inch thick wall with 1/2-inch steel plates, $\alpha = 0.033$).

3.8.3.4.1.2 Stiffness Assumptions for Global Seismic Analyses

The monolithic initial stiffness (Case 1 of Table 3.8.3-1) is used in the seismic analyses of the containment internal structures and the auxiliary building modules. This stiffness is used since the stresses due to mechanical loads including the safe shutdown earthquake are less than the cracking stress. The maximum in-plane concrete shear stresses in the containment internal structures modules are 97 psi for the 48-inch wall and 137 psi for the 30-inch wall due to the safe shutdown earthquake based on the monolithic section properties.

The broadening of the floor response spectra is sufficient to account for lower structural frequencies due to cracking of those portions of the structural modules that are boundaries of the in-containment refueling water storage tank exposed to abnormal thermal transients. Case 3 of Table 3.8.3-1 shows a calculated in-plane shear stiffness based on a 45-degree diagonal concrete compression strut with tensile loads carried only by the steel plate. These calculated stiffnesses are considerably lower than the test data described in References 27 and 28 where the overall stiffness reduced to 60 to 70 percent of the monolithic stiffness. If the calculated stiffnesses are conservatively used for the boundaries of the in-containment refueling water storage tank, the equivalent shear area of the containment internal structures stick model is reduced by about 30 percent with a corresponding reduction in frequency of about 16 percent.

3.8.3.4.1.3 Stiffness Assumptions for Local Seismic Analyses of In-Containment Refueling Water Storage Tank

The seismic analyses of the in-containment refueling water storage tank address the local response of the walls and water and are performed to verify the structural design of the tank. The lowest significant wall frequency is about 30 hertz using monolithic properties and would not be excited by the seismic input. The local analyses are therefore performed using the cracked section stiffness values based on composite behavior with zero stiffness for the concrete in tension (Case 3 of Table 3.8.3.1).

3.8.3.4.1.4 Damping of Structural Modules

Damping of the structural modules is reported in Reference 27 based on the cyclic load tests of a containment internal structure model. The equivalent viscous damping at the design load level was 5 percent for the concrete-filled steel model. This was almost constant up to the load level at which the steel plate started yielding. Dynamic analyses are performed using 7 percent damping for the reinforced concrete and 5 percent for the structural modules as shown in subsection 3.7.1.

3.8.3.4.2 Hydrodynamic Analyses

This subsection describes the hydrodynamic analyses for automatic depressurization system discharge into the in-containment refueling water storage tank. This discharge is designated as ADS_1 in the load description of subsection 3.8.3.3.1 and results in higher loading than ADS_2. The first three stages of the automatic depressurization system valves discharge into the tank through spargers under water, producing hydrodynamic loads on the tank walls and equipment. Hydrodynamic loads, measured in hydraulic tests of the automatic depressurization system sparger in a test tank, are evaluated using the source load approach (Reference 34). Analyses of the tests define source pressure loads that are then used in analyses of the in-containment refueling water storage tank to give the dynamic responses of the contaiment internal structures. The basic analysis approach consists of the following steps:

1. A pressure source, an impulsive forcing function at the sparger discharge, is selected from the tests using a coupled fluid structure finite element model of the test tank, taking into account fluid compressibility effects. This source development procedure is based on a comparison between analysis and test results, both near the sparger exit and at the boundaries of the test tank.

2. The pressure source is applied at each sparger location in a coupled fluid structure finite element model of the in-containment refueling water storage tank structure and of the contained water. The mesh characteristics of the model at the sparger locations and the applied forcing functions correspond to those of the test tank analysis.

3.8.3.4.2.1 Sparger Source Term Evaluation

A series of tests was conducted with discharge conditions representative of one sparger for the AP600 (References 35 and 36). Pressure traces measured during the test discharges were investigated, at both sparger exit and tank boundaries to (1) bound the expected discharge from the automatic depressurization system; (2) characterize the pressure wave transmission through the pool water; (3) determine the maximum pressure amplitudes and the frequency content; and (4) produce reference data for qualification of the analytical procedure. Pressure time histories and power spectrum densities were examined at reference sensors, both for the total duration of the discharge transient (about 50 seconds) and for critical time intervals.

Fluid-structure interaction analyses were performed with the ANSYS computer code (Reference 37). The mathematical model consists of a 3D sector finite element model, 15 degrees wide, as shown in Figure 3.8.3-9. It uses STIF30 fluid and STIF63 structural ANSYS finite elements, which take into account fluid compressibility and fluid-structure interaction. Rayleigh damping of 4 percent is used for the concrete structure, and fluid damping is neglected. Direct step-by-step time integration is used. The measured discharge pressures for single time intervals are imposed as uniform forcing functions on the idealized spherical surface of the steam/water interface, and pressures transmitted through the water to the tank boundary are calculated and compared with test measurements. The analyses of the test tank showed satisfactory agreement for the pressures at the tank boundary.

The examination of test results related to the structural design of the in-containment refueling water storage tank under automatic depressurization system hydrodynamic excitation and the comparison with the analytical procedure previously described, lead to the following conclusions regarding the sparger source term definition:

- The automatic depressurization system discharge into cold water produces the highest hydrodynamic pressures. The tests at higher water temperatures produce significantly lower pressures.

- Two pressure time histories, characterized by different shapes and frequency content, can be selected as representative of the sparger discharge pressures; they are assumed as acting on a spherical bubble centered on the sparger centerline and enveloping the ends of the sparger arms.

- The application of such time histories as forcing functions to an analytical model, simulating the fluid structure interaction effects in the test tank, has been found to predict the measured tank wall pressures, for the two selected reference time intervals.

- The two defined sparger source term pressure time histories can be used as forcing functions for global hydrodynamic analyses of the in-containment refueling water storage tank by developing a comprehensive fluid-structure finite element model and reproducing the test tank mesh pattern in the sparger region.

3.8.3.4.2.2 In-Containment Refueling Water Storage Tank Analyses

The in-containment refueling water storage tank is constructed as an integral part of the containment internal structures as described in subsection 3.8.3.1.3. It contains two depressurization spargers that are submerged approximately 9 feet below the normal water level. Transmission of the hydrodynamic pressures from the sparger discharge to the wetted in-containment refueling water storage tank is evaluated using the coupled fluid-structure interaction method similar to that described for the test tank analysis in the previous subsection.

The 3D ANSYS finite element model includes the in-containment refueling water storage tank boundary, the water within the in-containment refueling water storage tank, the adjacent structural walls of the containment internal structures, and the operating floor. The model of the in-containment refueling water storage tank, shown in Figures 3.8.3-10, 3.8.3-11, and

3.8.3-12, represents the outer steel structures, the inner concrete walls, and the water. The flexible steel outer wall is represented using beam and shell elements; isotropic plate elements are used to represent the inner structural module walls. The water is modelled as a compressible fluid to provide an acoustic medium to transmit the source pressure. The model has two bubble boundaries representing the spargers. Pressure loads are applied to the solid element faces adjacent to the air bubbles. The forcing functions at the sparger locations are conservatively assumed to be in phase. Rayleigh damping of 5 percent is used for the concrete-filled structural modules and fluid damping is neglected. All degrees of freedom were retained in the step-by-step direct integration solution procedure for the in-containment refueling water storage tank boundary and the water. Degrees of freedom in the adjacent walls and floor were condensed by Guyon reduction.

Significant structural frequencies of the containment internal structures were analyzed using the harmonic response option with the ANSYS model of the in-containment refueling water storage tank and containment internal structures. A harmonic unit pressure is applied at the surface of the spherical bubble representing the automatic depressurization system spargers. Material properties for the concrete elements are based on the uncracked gross concrete section (Case 2 of Table 3.8.3-1). The results of these harmonic response analyses show the response deflection as a function of input frequency at nodes in the containment internal structures. The harmonic response analyses show that the largest responses are close to the wetted boundary of the in-containment refueling water storage tank and that the significant frequencies are from 18 to 50 hertz.

Two time histories are identified for the structural hydrodynamic analyses; one has significant frequencies below 40 hertz while the other has significant frequencies in the range of 40 to 60 hertz. Both time history inputs are used in the hydrodynamic analyses with the monolithic uncracked section properties for all walls. The lower frequency input is also applied in lower bound analyses using the cracked section stiffness values (Case 3 of Table 3.8.3-1) for the concrete walls that are boundaries of the in-containment refueling water storage tank. Monolithic properties are used for the other walls. Results from these cases are enveloped, thereby accounting for variabilities in the structural analyses.

The analyses of the in-containment refueling water storage tank give wall pressures, displacements, accelerations, hydrodynamic floor response spectra, and member forces due to the automatic depressurization system discharge pressure forcing functions. Consideration of pressure wave transmission and fluid-structure interaction shows a significant wall pressure attenuation with distance from the sparger region and with increasing wall flexibilities, relative to the measured sparger pressure forcing function. The member stresses are evaluated against the allowable stresses specified in subsection 3.8.3.5 for seismic Category I structures, considering the hydrodynamic loads as live loads.

3.8.3.4.3 Thermal Analyses

The in-containment refueling water storage tank water and containment atmosphere are subject to temperature transients as described in subsection 3.8.3.3.1. The temperature transients result in a nonlinear temperature distribution within the wall modules. Temperatures within the concrete wall are calculated in a unidimensional heat flow analysis. The average and equivalent linear gradients are applied to a finite element model of the containment internal structures at selected times during the transient. The effect of concrete cracking is considered in the stiffness properties for the concrete elements subjected to the thermal transient.

Thermal transients for the design basis accidents are described in Section 6.3. The analyses for these transients are similar to those described above.

3.8.3.5 Design Procedures and Acceptance Criteria

The containment internal structures that contain reinforcing steel including most of the areas below elevation 98′, are designed by the strength method, as specified in the ACI Code Requirements for Nuclear Safety Related Structures, ACI-349. The ductility criteria of ACI-318, Chapter 21, are considered in detailing, placing, anchoring, and splicing of the reinforcing steel.

The internal steel framing is designed according to the AISC Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690, supplemented by the requirements given in subsection 3.8.4.5.

The secondary shield walls, in-containment refueling water storage tank, refueling cavity, and operating floor above the in-containment refueling water storage tank are designed using structural modules. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. Structural floor modules are designed as composite structures in accordance with AISC-N690.

Methods of analysis used are based on accepted principles of structural mechanics and are consistent with the geometry and boundary conditions of the structures.

The methods described in subsection 3.7.2 are employed to obtain the safe shutdown earthquake loads at various locations in the containment internal structures. The safe shutdown earthquake loads are derived from the response spectrum analysis of a three-dimensional, finite element model representing the entire containment internal structures.

The determination of pressure and temperature loads due to pipe breaks is described in subsections 3.6.1 and 6.2.1.2. Subcompartments inside containment containing high energy piping are designed for a pressurization load of 5 psi. The pipe tunnel in the CVS room (room 11209, Figure 1.2-6) is designed for a pressurization load of 7.5 psi. The design for the effects of postulated pipe breaks is performed as described in subsection 3.6.2. Determination of pressure loads resulting from actuation of the automatic depressurization system is described in subsection 3.8.3.4.3.

Determination of reactor coolant loop support loads is described in subsection 3.9.3. Design of the reactor coolant loop supports within the jurisdiction of ASME Code, Section III, Division 1, Subsection NF is described in subsections 3.9.3 and 5.4.10.

Computer codes used are general purpose codes. The code development, verification, validation, configuration control, and error reporting and resolution are according to the Quality Assurance requirements of Chapter 17.

3.8.3.5.1 Reactor Coolant Loop Supports

3.8.3.5.1.1 Reactor Vessel Support System

The embedded portions of the reactor vessel supports, which are outside the ASME jurisdictional boundary, are designed by elastic methods of analysis. They are analyzed and designed to resist the applicable loads and load combinations given in subsection 3.8.4.3. The design is according to AISC-N690 and ACI-349. Figure 3.8.3-4 shows the jurisdictional boundaries.

3.8.3.5.1.2 Steam Generator Support System

The embedded portions of the steam generator supports, which are outside the ASME jurisdictional boundary, are designed by elastic methods of analysis. They are analyzed and designed to resist the applicable loads and load combinations given in subsection 3.8.4.3. The design is according to AISC-N690 and ACI-349. Figure 3.8.3-5 shows the jurisdictional boundaries.

3.8.3.5.1.3 Reactor Coolant Pump Support System

The reactor coolant pumps are integrated into the steam generator channel head and consequently do not have a separate support system.

3.8.3.5.1.4 Pressurizer Support System

The embedded portions of the pressurizer supports, which are outside the ASME jurisdictional boundary, are designed by elastic methods of analysis. They are analyzed and designed to resist the applicable loads and load combinations given in subsection 3.8.4.3. The design is according to AISC-N690 and ACI-349. Figure 3.8.3-6 shows the jurisdictional boundaries.

3.8.3.5.2 Containment Internal Structures Basemat

The containment internal structures basemat including the primary shield wall and reactor cavity are designed for dead, live, thermal, pressure, and safe shutdown earthquake loads. Above elevation 98′ the primary shield wall is a structural module and is designed as described in subsection 3.8.3.5.3.

Below elevation 98′ the primary shield and reactor cavity are part of the reinforced concrete forming the base of the containment internal structures. They are designed according to ACI 349. Figure 3.8.3-7 shows the reinforcement.

3.8.3.5.3 Structural Wall Modules

Structural wall modules without concrete fill, such as the west wall of the in-containment refueling water storage tank, are designed as steel structures, according to the requirements of AISC-N690. This code is applicable since the module is constructed entirely out of structural steel plates and shapes. In local areas stresses due to restraint of thermal growth may exceed yield and the allowable stress intensity is 3 S_{m1}. This allowable is based on the allowable stress intensity for Service Level A loads given in ASME Code, Section III, Subsection NE, Paragraph NE-3221.4.

The concrete-filled steel module walls are designed for dead, live, thermal, pressure, safe shutdown earthquake, and loads due to postulated pipe breaks. The in-containment refueling water storage tank walls are also designed for the hydrostatic head due to the water in the tank and the hydrodynamic pressure effects of the water due to the safe shutdown earthquake, and automatic depressurization system pressure loads due to sparger operation. The walls of the refueling cavity are also designed for the hydrostatic head due to the water in the refueling cavity and the hydrodynamic pressure effects of the water due to the safe shutdown earthquake.

Figure 3.8.3-8 shows the typical design details of the structural modules, typical configuration of the wall modules, typical anchorages of the wall modules to the reinforced base concrete, and connections between adjacent modules. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The application of ACI-349 and the supplemental requirements are supported by the behavior studies described in subsection 3.8.3.4.1.

3.8.3.5.3.1 Design for Axial Loads and Bending

Design for axial load (tension and compression), in-plane bending, and out-of-plane bending is in accordance with the requirements of ACI-349, Chapters 10 and 14.

3.8.3.5.3.2 Design for In-Plane Shear

Design for in-plane shear is in accordance with the requirements of ACI-349, Chapters 11 and 14. The steel faceplates are treated as reinforcing steel, contributing as provided in Section 11.5 of ACI-349~~, except that the shear capacity is based on the shear yield~~.

3.8.3.5.3.3 Design for Out-of-Plane Shear

Design for out-of-plane shear is in accordance with the requirements of ACI-349, Chapter 11~~American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications. The acceptance criteria in the AASHTO specification are based on the modified compression field theory, resulting in a more rational approach for shear design than the conventional ACI methods. The modified compression field theory has been implemented~~

~~in a number of engineering codes of practice, both in North America and in Europe. The background is provided by Collins (Reference 38).~~

3.8.3.5.3.4 Evaluation for Thermal Loads

The effect of thermal loads on the concrete-filled structural wall modules is evaluated by using the working stress design method for the load combinations of Table 3.8.4-2 with the load factors taken as unity. This evaluation is in addition to the evaluation using the strength design method of ACI-349 for the load combination without the thermal load. Acceptance for the load combination with thermal loads is that the stress in general areas of the steel plate be less than yield. In local areas the stress may exceed yield and the allowable stress intensity is 3 S_{m1}. This allowable is based on the allowable stress intensity for Service Level A loads given in ASME Code, Section III, Subsection NE, Paragraph NE-3221.4.

3.8.3.5.3.5 Design of Trusses

The trusses provide a structural framework for the modules, maintain the separation between the faceplates, support the modules during transportation and erection, and act as "form ties" between the faceplates when concrete is being placed. After the concrete has cured, the trusses are not required to contribute to the strength or stiffness of the completed modules. However, they do provide additional strength similar to that provided by stirrups in reinforced concrete. The trusses are designed according to the requirements of AISC-N690.

3.8.3.5.3.6 Design of Shear Studs

The wall structural modules are designed as reinforced concrete elements, with the faceplates serving as reinforcing steel. Since the faceplates do not have deformation patterns typical of reinforcing steel, shear studs are provided to transfer the forces between the concrete and the steel faceplates. The shear studs make the concrete and steel faceplates behave compositely. In addition, the shear studs permit anchorage for piping and other items attached to the walls.

The size and spacing of the shear studs is based on Section Q1.11.4 of AISC-N690 to develop full composite action between the concrete and the steel faceplates.

3.8.3.5.4 Structural Floor Modules

Figure 3.8.3-3 shows the typical design details of the floor modules. The operating floor is designed for dead, live, thermal, safe shutdown earthquake, and pressure due to automatic depressurization system operation or due to postulated pipe break loads. The operating floor region above the in-containment refueling water storage tank is a series of structural modules. The remaining floor is designed as a composite structure of concrete slab and steel beams in accordance with AISC-N690.

For vertical downward loads, the floor modules are designed as a composite section, according to the requirements of Section Q1.11 of AISC-N690. Composite action of the steel section and concrete fill is assumed based on meeting the intent of Section Q1.11.1 for beams totally encased in concrete. Although the bottom flange of the steel section is not encased within concrete, the design configuration of the floor module provides complete concrete confinement to prevent spalling. It also provides a better natural bonding than the code-required configuration.

For vertical upward loads, no credit is taken for composite action. The steel members are relied upon to provide load-carrying capacity. Concrete, together with the embedded angle stiffeners, is assumed to provide stability to the plates.

Floor modules are designed using the following basic assumptions and related requirements:

- Concrete provides restraint against buckling of steel plates. The buckling unbraced length of the steel plate, therefore, is assumed to equal the span length between the fully embedded steel plates and shapes.

- Although the floor modules forming the top (ceiling) of the in-containment refueling water storage tank are not in contact with water, stainless steel plates are used for the tank boundary.

- The floor modules are designed as simply supported beams.

3.8.3.5.4.1 Design for Vertical Downward Loads

The floor modules are designed as a one-way composite concrete slab and steel beam system in supporting the vertical downward loads. The effective width of the concrete slab is determined according to Section Q1.11.1 of AISC-N690. The effective concrete compression area is extended to the neutral axis of the composite section. The concrete compression area is treated as an equivalent steel area based on the modular ratio between steel and concrete material. Figure 3.8.3-13 shows the effective composite sections. The steel section is proportioned to support the dead load and construction loads existing prior to hardening of the concrete. The allowable stresses are provided in Table 3.8.4-1.

3.8.3.5.4.2 Design for Vertical Upward Loads

For vertical upward loads, the floor modules are designed as noncomposite steel structures. The effective width, b_e, of the faceplate in compression is based on post-buckling strength of steel plates and is determined from Equation (4.16) of Reference 44. The faceplates of the structural floor modules are stiffened and supported by embedded horizontal angles. Hence, the buckling unbraced length of the faceplates is equal to the span length between the horizontal angles. Since concrete provides restraint against buckling of the steel plates, a value of 0.65 is used for k when calculating the effective length of the steel plates and stiffeners whenever the plate or stiffener is continuous. The buckling stress, f_{cr}, of the faceplates is determined from Sections 9.2 and 9.3 of Reference 45. The effective width of the faceplates of the structural floor modules in compression is shown in Figure 3.8.3-13. The allowable stresses are provided in Table 3.8.4-1.

3.8.3.5.4.3 Design for In-Plane Loads

In-plane shear loads acting on the floor modules are assumed to be resisted only by the steel faceplate without reliance on the concrete for strength. The stresses in the faceplate due to the in-plane loads are combined with those due to vertical loads. The critical stress locations of the floor faceplate are evaluated for the combined normal and shear stress, based on the von Mises yield criterion:

For the particular case of a two-dimension stress condition the equation is:

$$(\sigma_1)^2 - \sigma_1\sigma_2 + (\sigma_2)^2 = (f_y)^2$$

where σ_1 and σ_2 are the principal stresses and f_y is the uniaxial yield stress.

For the faceplate where normal, σ, and shear, τ, stresses are calculated, the principal stresses can be expressed as follows:

$$\sigma_1 = (\frac{\sigma}{2}) + \sqrt{\frac{\sigma^2}{4} + \tau^2}$$

$$\sigma_2 = (\frac{\sigma}{2}) - \sqrt{\frac{\sigma^2}{4} + \tau^2}$$

Therefore, the condition at yield becomes:

$$\sigma^2 + 3\tau^2 = (f_y)^2$$

For the design of the structural floor module faceplate, the allowable stresses for the various loading conditions are as follows:

Normal condition:

$$\sigma^2 + 3\tau^2 \leq (0.6\ f_y)^2$$

Severe condition:

$$\sigma^2 + 3\tau^2 \leq (0.6\ f_y)^2$$

Extreme/abnormal condition:

$$\sigma^2 + 3\tau^2 \leq (0.96\ f_y)^2$$

Thermal stresses in the faceplates result from restraint of growth during the thermal transients described in subsection 3.8.3.3.1. ~~The allowable stress intensity is 3 S_{mi} for normal loads including local thermal stresses. This allowable is based on the allowable stress intensity for Service Level A loads given in ASME Code, Section III, Subsection NE, Paragraph NE-3221.4.~~ Evaluation for thermal stresses is the same as discussed in subsection 3.8.3.5.3.4 for the wall modules.

3.8.3.5.5 Internal Steel Framing

Internal steel framing is analyzed and designed according to AISC-N690. Seismic analysis methods are described in subsection 3.7.3.

3.8.3.5.6 Steel Form Modules

The steel form modules consist of plate reinforced with angle stiffeners and tee sections as shown in Figure 3.8.3-16. The steel form modules are designed for concrete placement loads defined in subsection 3.8.3.3.2.

The steel form modules are designed as steel structures according to the requirements of AISC-N690. This code is applicable since the form modules are constructed entirely out of structural steel plates and shapes and the applied loads are resisted by the steel elements.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

Subsection 3.8.4.6 describes the materials and quality control program used in the construction of the containment internal structures. The structural steel modules are constructed using A36 plates and shapes. Nitronic 33 (American Society for Testing and Materials 240, designation S2400, Type XM-29) stainless steel plates are used on the surfaces of the modules in contact with water during normal operation or refueling. The structural wall and floor modules are fabricated and erected in accordance with AISC-N690. Loads during fabrication and erection due to handling and shipping are considered as normal loads as described in subsection 3.8.4.3.1.1. Packaging, shipping, receiving, storage and handling of structural modules are in accordance with NQA-2, Part 2.2 (formerly ANSI/ASME N45.2.2 as specified in AISC N690).

3.8.3.6.1 Fabrication, Erection, and Construction of Structural Modules

Modular construction techniques are used extensively in the containment internal structures (Figure 3.8.3-1). Subassemblies, sized for commercial rail shipment, are assembled offsite and transported to the site. Onsite fabrication consists of combining the subassemblies in structural modules, which are then installed in the plant. A typical modular construction technique is described in the following paragraphs for Module M1, which is the main structural module in the containment internal structures.

The M1 module is a multicompartmented structure which, in its final form, comprises the central walls of the containment internal structures. The vertical walls of the module house the refueling cavity, the reactor vessel compartment, and the two steam generator compartments. The module (Figure 3.8.3-14) is in the form of a "T" and is approximately 50 feet long, 65 feet wide and 60 feet high. The module is assembled from about 40 prefabricated wall sections called structural submodules (Figure 3.8.3-15). The submodules are designed for railroad transport from the fabricator's shop to the plant site with sizes up to 12 feet by 12 feet by 80 feet long, weighing up to 80 tons. A typical submodule weighs between 9 and 11 tons. The submodules are assembled outside the nuclear island and the completed M1 module is lifted to its final location within the containment vessel by the heavy lift construction crane. Following placement of the M1 module within the containment building, the hollow wall structures are filled with concrete, forming a portion of the structural walls of the containment internal structures.

Tolerances for fabrication, asembly and erection of the structural modules conform to the requirements of section 4 of ACI-117, sections 3.3 and 3.4 of AWS D1.1, and sections Q1.23 and Q1.25 of AISC-N690.

3.8.3.6.2 Nondestructive Examination

Nondestructive examination of the submodules and module is performed according to AISC-N690 and AWS D 1.1. Welds are visually examined for 100 percent of their length. Full penetration welds are inspected by ultrasonic or radiographic examination for 10 percent of their length. Partial penetration welds are inspected by magnetic particle or liquid penetrant examination for 10 percent of their length.

3.8.3.6.3 Concrete Placement

After installation of the M1 module in the containment, the hollow walls are filled with concrete. Concrete is placed in each wall continuously from bottom to top. The concrete is placed through multiple delivery trunks located along the top of the wall. It is placed in incremental layers with the placement rate based on the pressure of the wet concrete and its setting time. During concrete placement, workers and inspectors have access to the inside of the modules. The arrangement of the module internal trusses provides communication to aid in the free flow of concrete and movement of personnel.

3.8.3.7 In-Service Testing and Inspection Requirements

There are no in-service testing or inspection requirements for the containment internal structures.

3.8.4 Other Category I Structures

The other seismic Category I structures are the shield building and the auxiliary building.

General criteria in this section describing the loads, load combinations, materials, and quality control are also applicable to the containment internal structures described in subsection 3.8.3.

3.8.4.1 Description of the Structures

3.8.4.1.1 Shield Building

The shield building is the shield building structure and annulus area that surrounds the containment building. It shares a common basemat with the containment building and the auxiliary building. The shield building is a reinforced concrete structure. The figures in Section 1.2 show the layout of the shield building and its interface with the other buildings of the nuclear island.

The following are the significant features and the principal systems and components of the shield building:

- Shield building cylindrical structure
- Shield building roof structure
- Lower annulus area
- Middle annulus area
- Upper annulus area
- Passive containment cooling system air inlet
- Passive containment cooling system water storage tank
- Passive containment cooling system air diffuser
- Passive containment cooling system air baffle
- Passive containment cooling system air inlet plenum

The cylindrical section of the shield building provides a radiation shielding function, a missile barrier function, and a passive containment cooling function. Additionally, the cylindrical section structurally supports the roof with the passive containment cooling system water storage tank and serves as a major structural member for the nuclear island. The floor slabs and structural walls of the auxiliary building are structurally connected to the cylindrical section of the shield building.

The shield building roof is a reinforced concrete shell supporting the passive containment cooling system tank and air diffuser. Air intakes are located at the top of the cylindrical portion of the shield building. The conical roof supports the passive containment cooling system tank, which is constructed with a stainless steel liner on reinforced concrete walls. The air diffuser is located in the center of the roof and discharges containment cooling air upwards.

The upper annulus of the shield building is the volume of the annulus between elevation 132′-3″ and the bottom of the air diffuser. The middle annulus area, the volume of annulus between elevation 100′-0″ and elevation 132′-3″, contains the majority of the containment vessel penetrations. The area below elevation 100′-0″ is the lower annulus of the shield building. There is a concrete floor slab in the annulus at elevation 132'-3", which is incorporated with the stiffener attached to the containment vessel.

A permanent flexible watertight and airtight seal is provided between the concrete floor slab at elevation 132'-3" and the shield building to provide an environmental barrier between the upper and middle annulus sections. The flexible watertight seal is utilized to seal against water leakage from the upper annulus into the middle annulus. The seal is designated as nonsafety-related and nonseismic; it is not relied upon to mitigate design basis events. The seal is able to accommodate events resulting in containment temperature and pressure excursions that result in lateral shell movement inward or outward.

3.8.4.1.2 Auxiliary Building

The auxiliary building is a reinforced concrete and structural steel structure. Three floors are above grade and two are located below grade. It is one of the three buildings that make up the nuclear island and shares a common basemat with the containment building and the shield building.

The auxiliary building is a C-shaped section of the nuclear island that wraps around approximately 50 percent of the circumference of the shield building. The floor slabs and the structural walls of the auxiliary building are structurally connected to the cylindrical section of the shield building.

The figures in Section 1.2 show the layout of the auxiliary building and its interface with the other buildings of the nuclear island. The following are the significant features and the principal systems and components of the auxiliary building:

- Main control room
- Remote shutdown area
- Class 1E dc switchgear
- Class 1E batteries
- Reactor trip switchgear
- Reactor coolant pump trip switchgear
- Main steam and feedwater piping
- Main control room heating, ventilating, and air conditioning (HVAC)
- Class 1E switchgear rooms heating, ventilating, and air conditioning
- Spent fuel pool
- Fuel transfer canal
- Cask loading and washdown pits
- New fuel storage area
- Cask handling crane
- Fuel handling machine
- Chemical and volume control system (CVS) makeup pumps
- Normal residual heat removal system (RNS) pumps and heat exchangers
- Liquid radwaste tanks and components
- Spent fuel cooling system

- Gaseous radwaste processing system
- Mechanical and electrical containment penetrations

Structural modules are used for part of the south side of the auxiliary building. These structural modules are structural elements built up with welded steel structural shapes and plates. Concrete is used where required for shielding, but reinforcing steel is not normally used. These modules include the spent fuel pool, fuel transfer canal, and cask loading and cask washdown pits. The structural modules are similar to the structural modules described above for the containment internal structures. Figure 3.8.4-5 shows the location of the structural modules. The structural modules extend from elevation 66′-6″ to elevation 135′-3″.

The ceiling of the main control room (floor at elevation 135′-3″), and the instrumentation and control rooms (floor at elevation 117′-6″) are designed as finned floor modules (Figure 3.8.4-6). A finned floor consists of a 24-inch-thick concrete slab poured over a stiffened steel plate ceiling. The fins are rectangular plates welded perpendicular to the plate. Shear studs are welded on the other side of the steel plate, and the steel and concrete act as a composite section. The fins are exposed to the environment of the room, and enhance the heat-absorbing capacity of the ceiling (see Standard Safety Analysis Report (SSAR) subsection 6.4.2.2). Several shop-fabricated steel panels, placed side by side, are used to construct the stiffened plate ceiling in a modularized fashion. The stiffened plate is designed to withstand construction loads prior to concrete hardening.

The new fuel storage area is a separate reinforced concrete pit providing temporary dry storage for the new fuel assemblies.

A cask handling crane travels in the east-west direction. The location and travel of this crane prevents the crane from carrying loads over the spent fuel pool, thus precluding them from falling into the spent fuel pool.

3.8.4.1.3 Containment Air Baffle

The containment air baffle is located within the upper annulus of the shield building, providing an air flow path for the passive containment cooling system. The air baffle separates the downward air flow entering at the air inlets from the upward air flow that cools the containment vessel and flows out of the discharge stack. The upper portion is supported from the shield building roof and the remainder is supported from the containment vessel. The air baffle is a seismic Category I structure designed to withstand the wind and tornado loads defined in Section 3.3. The air baffle structural configuration is depicted in Figures 1.2-14 and 3.8.4-1. The baffle includes the following sections:

- A wall supported off the shield building roof (see Figure 1.2-14)

- A series of panels attached to the containment vessel cylindrical wall and the knuckle region of the dome

- A sliding plate closing the gap between the wall and the panels fixed to the containment vessel, designed to accommodate the differential movements between the containment vessel and shield building

- Flow guides attached at the bottom of the air baffle to minimize pressure drop.

The air baffle is designed to meet the following functional requirements:

- The baffle and its supports are configured to minimize pressure losses as air flows through the system

- The baffle and its supports have a design objective of 60 years

- The baffle and its supports are configured to permit visual inspection and maintenance of the air baffle as well as the containment vessel. Periodic visual inspections are primarily to inspect the condition of the coatings

- The baffle is designed to maintain its function during postulated design basis accidents

- The baffle is designed to maintain its function under specified external events including earthquakes, hurricanes and tornadoes.

The design of the containment air baffle is shown in Figure 3.8.4-1. The portion of the air baffle attached to the containment cylinder comprises 60 panels in each of five rows, with each panel subtending an arc of six degrees (approximately 6 feet 11 inches wide). Each panel is supported by horizontal beams spaced approximately 14 feet apart. These horizontal beams span the six-degree arc and are bolted to U-shaped attachments welded to the containment vessel. The attachment locations are established considering the containment vessel plate and ring assemblies, as shown in Figure 3.8.2-1. The lowest attachments are at the bottom of the middle containment ring subassembly. The upper attachments are on the head. The attachments can be installed in the subassembly area and, therefore, should not interfere with the containment vessel erection welds. The only penetrations through the containment vessel above the operating deck at elevation 135′-3″ are the main equipment hatch and personnel airlock. Ten panels are deleted at the equipment hatch and two flow guides at the personnel airlock.

Two rows of panels are attached to the containment vessel above the cylindrical portion. The panels are curved to follow the curvature of the knuckle region of the head and then become flat forming a conical baffle that provides a transitional flow region into the upper shield building. A vertical sliding plate is provided between this upper row of panels and the air baffle that is attached directly to the shield building roof as shown in sheet 4 of Figure 3.8.4-1. This sliding plate accommodates the differential movement between the containment vessel and the shield building, based on the absolute sum of the containment pressure and temperature deflections and of the seismic deflections.

The panels accommodate displacements between each panel due to containment pressure and thermal growth. Radial and circumferential growth of the containment vessel are accommodated by slip at the bolts between the horizontal beams and the U shaped attachment resulting in small gaps between adjacent panels. Vertical growth is accommodated by slip between the panel and the horizontal beam supporting the top of the panel. Cover plates between the panels limit leakage during and after occurrence of these differential displacer

3.8.4.1.4 Seismic Category I Cable Tray Supports

Electric cables are routed in horizontal and vertical steel trays supported by channel type struts made out of cold rolled channel type sections. Spacing of the supports is determined by allowable loads in the trays and stresses in the supports. The supports are attached to the walls, floors, and ceiling of the structures as required by the arrangement of the cable trays. Longitudinal and transverse bracing is provided where required.

3.8.4.1.5 Seismic Category I Heating, Ventilating, and Air Conditioning Duct Supports

Heating, ventilating, and air conditioning duct supports consist of structural steel members or cold rolled channel type sections attached to the walls, floors, and ceiling of the structures as required by the arrangement of the duct. Spacing of the supports is determined by allowable stresses in the duct work and supports. Longitudinal and transverse bracing is provided where required.

3.8.4.2 Applicable Codes, Standards, and Specifications

The following standards are applicable to the design, materials, fabrication, construction, inspection, or testing:

- American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, ACI-318-~~89~~95 (refer to subsection 3.8.4.4.1 for applicability)

- American Concrete Institute (ACI), Code Requirements for Nuclear Safety Related Structures, ACI-349-90 (refer to subsection 3.8.4.5 for supplemental requirements)

- American Concrete Institute (ACI), Manual of Standard Practice for Detailing Reinforced Concrete Structures, ACI-315-88

- American Institute of Steel Construction (AISC), Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690-1984 (refer to subsection 3.8.4.5 for supplemental requirements)

- American Iron and Steel Institute (AISI), Specification for the Design of Cold Formed Steel Structural Members, Parts 1 and 2, 1986

- American Welding Society (AWS), Structural Welding Code, AWS D 1.1-90

The panels accommodate displacements between each panel due to containment pressure and thermal growth. Radial and circumferential growth of the containment vessel are accommodated by slip at the bolts between the horizontal beams and the U shaped attachment resulting in small gaps between adjacent panels. Vertical growth is accommodated by slip between the panel and the horizontal beam supporting the top of the panel. Cover plates between the panels limit leakage during and after occurrence of these differential displacements.

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- American Concrete Institute (ACI), Code Requirements for Nuclear Safety Related Structures, ACI-349-90 (refer to subsection 3.8.4.5 for supplemental requirements)

- American Concrete Institute (ACI), Manual of Standard Practice for Detailing Reinforced Concrete Structures, ACI-315-88

- American Institute of Steel Construction (AISC), Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690-1984 (refer to subsection 3.8.4.5 for supplemental requirements)

- American Iron and Steel Institute (AISI), Specification for the Design of Cold Formed Steel Structural Members, Parts 1 and 2, 1986

- American Welding Society (AWS), Structural Welding Code, AWS D 1.1-90

- American Welding Society (AWS), Reinforcing Steel Welding Code, AWS D 1.4-89
- National Construction Issues Group (NCIG), Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants, NCIG-01, Revision 2, May 7, 1985
- American Association of State Highway and Transportation Officials (AASHTO), Bridge Design Specifications, 1994.

Section 1.9 describes conformance with the Regulatory Guides.

Welding and inspection activities for seismic Category I structural steel, including building structures, structural modules, cable tray supports and heating, ventilating, and air conditioning duct supports are accomplished in accordance with written procedures and meet the requirements of the American Institute of Steel Construction (AISC N-690). The weld acceptance criteria is as defined in NCIG-01 Revision 2. The welded seam of the plates forming part of the leaktight boundary of the spent fuel pool and fuel transfer canal are examined by liquid penetrant and vacuum box after fabrication to confirm that the boundary does not leak.

3.8.4.3 Loads and Load Combinations

3.8.4.3.1 Loads

The loads considered are normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

3.8.4.3.1.1 Normal Loads

Normal loads are those loads to be encountered, as specified, during initial construction stages, during test conditions, and later, during normal plant operation and shutdown. They include the following:

D = Dead loads or their related internal moments and forces, including any permanent piping and equipment loads

F = Lateral and vertical pressure of liquids or their related internal moments and forces

L = Live loads or their related internal moments and forces, including any movable equipment loads and other loads that vary with intensity and occurrence~~, such as soil pressure~~

H = Static earth pressure or its related internal moments and forces

T_o = Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition

R_o = Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

3.8.4.3.1.2 Severe Environmental Loads

The severe environmental load is the following:

W = Loads generated by the design wind specified for the plant in subsection 3.3.1.1

3.8.4.3.1.3 Extreme Environmental Loads

Extreme environmental loads are the following:

E_s = Loads generated by the safe shutdown earthquake specified for the plant, including the associated hydrodynamic and dynamic incremental soil pressure. Loads generated by the safe shutdown earthquake are specified in Section 3.7.

W_t = Loads generated by the design tornado specified for the plant in subsection 3.3.2, including loads due to tornado wind pressure, differential pressure, and tornado-generated missiles.

N = Loads generated by the probable maximum precipitation (provided previously in Table 2.0-1).

3.8.4.3.1.4 Abnormal Loads

Abnormal loads are those loads generated by a postulated high-energy pipe break accident for pipes not qualified for leak-before-break. Abnormal loads include the following:

P_a = Pressure load within or across a compartment generated by the postulated break. The main steam isolation valve (MSIV) and steam generator blowdown valve compartments are designed for a pressurization load of 5 psi. Determination of subcompartment pressure loads is discussed in subsection 6.2.1.2.

T_a = Thermal loads under thermal conditions generated by the postulated break and including T_o. Determination of subcompartment temperatures is discussed in subsection 6.2.1.2.

R_a = Pipe reactions under thermal conditions generated by the postulated break and including R_o. Determination of pipe reactions generated by postulated breaks is discussed in subsection 3.6.

Y_r = Load on the structure generated by the reaction on the broken high-energy pipe during the postulated break. Determination of the loads is discussed in Section 3.6.

Y_j = Jet impingement load on the structure generated by the postulated break. Determination of the loads is discussed in Section 3.6.

Y_m = Missile impact load on the structure generated by or during the postulated break, as from pipe whipping. Determination of the loads is discussed in Section 3.6.

3.8.4.3.1.5 Dynamic Effects of Abnormal Loads

The dynamic effects from the impulsive and impactive loads caused by P_a, R_a, Y_r, Y_j, Y_m, and tornado missiles are considered by one of the following methods:

- Applying an appropriate dynamic load factor to the peak value of the transient load
- Using impulse, momentum, and energy balance techniques
- Performing a time-history dynamic analysis

Elastoplastic behavior may be assumed with appropriate ductility ratios, provided excessive deflections will not result in loss of function of any safety-related system.

Dynamic increase factors appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength.

3.8.4.3.2 Load Combinations

3.8.4.3.2.1 Steel Structures

The steel structures and components are designed according to the elastic working stress design methods of the AISC-N690 specification using the load combinations specified in Table 3.8.4-1.

3.8.4.3.2.2 Concrete Structures

The concrete structures and components are designed according to the strength design methods of ACI-349 Code, using the load combinations specified in Table 3.8.4-2.

3.8.4.3.2.3 Live Load for Seismic Design

Live loads specified for design of the floors vary from 100 to 800 pounds per square foot and are based on plant construction and maintenance activities. Only a fraction of these live loads is present during plant operation. In combinations with seismic loads, floors are designed for live loads that might realistically be present during normal plant operation. Live loads in combination with the safe shutdown earthquake are taken as 25 percent of the specified floor live load. The mass of equipment and distributed systems is included in both the dead and seismic loads.

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Seismic Category I Structures

The design and analysis procedures for the seismic Category I structures (other than the containment vessel and containment internal structures), including assumptions on boundary conditions and expected behavior under loads, are in accordance with ACI-349 for concrete structures, with AISC-N690 for steel structures, and AISI for cold formed steel structures. The structural modules in the auxiliary building are designed similar to the structural modules in the containment internal structures described in subsection 3.8.3. The ductility criteria of ACI-318, Chapters 12 and 21, are considered in detailing, placing, anchoring, and splicing of the reinforcing steel. The bases of design for the tornado, pipe breaks, and seismic effects are discussed in Sections 3.3, 3.6, and 3.7, respectively. The foundation design is described in subsection 3.8.5.

The seismic Category I structures are reinforced concrete and structural module shear wall structures consisting of vertical shear/bearing walls and horizontal slabs supported by structural steel framing. In-plane seismic forces are obtained from the response spectrum analysis of the three dimensional finite element fixed base models described in Table 3.7.2-14. These results are modified to account for soil structure interaction and accidental torsion as described in subsection 3.7.2. Also evaluated and considered in the shear wall and floor slab design are out-of-plane bending and shear loads, such as live load, dead load, seismic, lateral earth pressure, hydrostatic, hydrodynamic, and wind pressure. These out-of-plane bending and shear loads are obtained from hand calculations. The exterior walls of the seismic Category I structures below the grade are designed to resist the worst case lateral earth pressure loads (static and dynamic), soil surcharge loads, and loads due to external flooding as described in Section 3.4. Appendix 2C describes the seismic analyses used to calculate the lateral earth pressures on the exterior walls below grade. The exterior walls are also designed for full passive earth pressure which is utilized in the sliding evaluation described in subsection 3.8.5.5.3. Figure 3.8.4-2 shows typical shear walls and the arrangement of the reinforcing steel. Figure 3.8.4-3 shows typical reinforcing for the slabs.

The shield building roof and the passive containment cooling water storage tank are analyzed using three-dimensional finite element models with the ANSYS and GTSTRUDL computer codes. Loads and load combinations are given in subsection 3.8.4.3 and include construction, dead, live, thermal, wind and seismic loads. Seismic loads are applied as equivalent static accelerations. The seismic response of the water in the tank is analyzed in a separate finite element response spectrum analysis with seismic input defined by the floor response spectrum.

The structural steel framing is used primarily to support the concrete slabs and roofs. Metal decking, supported by the steel framing, is used as form work for the concrete slabs and roofs. The structural steel framing is designed for vertical loads. Figure 3.8.4-4 shows typical structural steel framing in the auxiliary building.

Computer codes used are general purpose computer codes. The code development, verification, validation, configuration control, and error reporting and resolution are according to the quality assurance requirements of Chapter 17.

The finned floors for the main control room and the instrumentation and control room ceilings are designed as reinforced concrete slabs in accordance with ACI-349. The steel panels are designed and constructed in accordance with AISC-N690. For positive bending, the steel plate is in tension and the steel plate with fin stiffeners serves as the bottom reinforcement. For negative bending, compression is resisted by the stiffened plate and tension by top reinforcement in the concrete.

3.8.4.4.2 Seismic Category I Cable Tray Supports

The design and analysis procedures for seismic Category I cable trays and their supports are described in Appendix 3F.

3.8.4.4.3 Seismic Category I Heating, Ventilating, and Air Conditioning Duct Supports

The design and analysis procedures for seismic Category I heating, ventilating, and air conditioning ducts and their supports are described in Appendix 3A.

3.8.4.5 Structural Criteria

The analysis and design of concrete conform to ACI-349. The analysis and design of structural steel conform to AISC-N690. The analysis and design of cold-formed steel structures conform to AISI. The margins of structural safety are as specified by those codes.

3.8.4.5.1 Supplemental Requirements for Concrete Structures

Supplemental requirements for ACI-349 are given in the position on Regulatory Guide 1.142 in Appendix 1A. In addition, the criteria of ACI-318, Chapters 12 and 21, are considered in detailing, placing, anchoring, and splicing of the reinforcing steel.

Design of fastening to concrete is in accordance with ACI 349-90, Appendix B with supplementary criteria based on references 46, 47, and 48. Reference 46 provides background material and a recommended design approach for consideration in the ACI building code. References 47 and 48 evaluate the test data against various anchor bolt design approaches. These references are being considered by the code committee responsible for ACI 349. Pending revision to Appendix B, the AP600 criteria include the following considerations:

- The 45 degree cone assumption used in the Appendix B approach is eliminated.
- The basic single anchor capacities for tension and shear are calculated by empirical formulae based on test data.

- Edge effects consider the effect of edges within a distance of one and a half times the embedment depth. Edge distances are sufficient to prevent lateral bursting.

- Group effects consider the effect of adjacent anchors within a distance of three times the embedment depth.

- ~~Strength reduction factors and the steel strength are specified such that the steel yields prior to concrete failure.~~Anchors are designed wherever possible with sufficent depth of embedment and side cover such that the steel anchor yields prior to failure of the concrete.

- The effect of concrete cracking is considered for fasteners located within the tensile zone of supporting concrete.

3.8.4.5.42 Supplemental Requirements for Steel Structures

Supplemental requirements for use of AISC-N690 are as follows:

- In Section Q1.0.2, the definition of secondary stress applies to stresses developed by temperature loading only.

- In Section Q1.3, where the structural effects of differential settlement are present, they are included with the dead load, D.

- In Table Q1.5.7.1, the stress limit coefficients for compression are as follows:
  1.3 instead of 1.5 in load combinations 2, 5, and 6.
  1.4 instead of 1.6 in load combinations 7, 8, and 9.
  1.6 instead of 1.7 in load combination 11.

- In Section Q1.5.8, for constrained members (rotation and/or displacement constraint such that a thermal load causes significant stresses), supporting safety-related structures, systems, or components, the stresses under load combinations 9, 10, and 11 are limited to those allowed in Table Q1.5.7.1 as modified above.

- Sections Q1.24 and Q1.25.10 are supplemented as follows:
  Shop painting is in accordance with Section M of the Manual of Steel Construction, Load and Resistance Factor Design, First Edition. Exposed areas after installation are field painted in accordance with the applicable portion of Chapter M of the Manual of Steel Construction, Load and Resistance Factor Design, First Edition. See subsection 6.1.2.1 for additional description of the protective coatings.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

This subsection contains information relating to the materials, quality control program, and special construction techniques used in the construction of the other seismic Category I structures, as well as the containment internal structures.

3.8.4.6.1 Materials

3.8.4.6.1.1 Concrete

The compressive strength of concrete used in the seismic Category I structures and containment internal structures is f_c'=4000 psi. The test age of concrete containing pozzolan is 90 days. The test age of concrete without pozzolan is the normal 28 days. Concrete is batched and placed according to Reference 6, Reference 7, and ACI-349.

Portland cement conforms to Reference 8, Type II, with the sum of tricalcium silicate and tricalcium aluminate limited to no more than 58 percent. It is also limited to no more than 0.60 percent by weight of alkalies calculated as Na_2O plus 0.658 K_2O. Certified copies of mill test reports showing that the chemical composition and physical properties conform to the specification are obtained for each cement delivery.

Aggregates conform to Reference 9. The fineness modulus of fine aggregate (sand) is not less than 2.5, nor more than 3.1. In at least four of five successive test samples, such modulus is not allowed to vary more than 0.20 from the moving average established by the last five tests. Coarse aggregates may be rejected if the loss from the Los Angeles abrasion test, Reference 10, using Grading A or Reference 11, exceeds 40 percent by weight at 500 revolutions. Acceptance of source and aggregates is based on the tests specified in Table 3.8.4-3.

Water and ice used in mixing concrete do not contain more than 250 parts per million of chlorides (as Cl) as determined in accordance with Reference 12. They do not contain more than 2000 parts per million of total solids as determined in accordance with Reference 13. Water meets the criteria in Table 3.8.4-4 in regard to the effects of the proposed mixing water on hardened cement pastes and mortars compared with distilled water.

The concrete contains a pozzolan, an air entraining admixture, and a water-reducing admixture. Admixtures, except pozzolan, are stored in liquid solution.

Admixtures do not contain added chlorides except as contained in potable drinking water used for manufacture of the admixtures. The chloride content is stated in the manufacturer's material certification.

Pozzolan conforms to Reference 14, except that the ignition loss does not exceed 6 percent.

Pozzolan is sampled and tested in accordance with Reference 15 for source approval.

Air entraining admixture conforms to Reference 16 and is the vinsol resin type.

Water-reducing admixture conforms to Reference 17 and is types A and D. Use of types A and D as limited by concrete placing temperature, least dimension of member sizes, and type of placement is as shown in Table 3.8.4-5.

Manufacturer's certification for the air entraining admixture is required demonstrating compliance with Reference 16, Section 4 requirements.

Manufacturer's certification for the water-reducing admixture is required demonstrating compliance with Reference 17, Section 5 requirements.

Manufacturer's test reports are required for each delivery of pozzolan showing the chemical composition and physical properties and certifying that the pozzolan complies with the specification.

Proportioning of the concrete mix is in accordance with Reference 18 and Option B of Reference 6, except that in lieu of the requirements of Reference 6, Paragraph 5.3.1.2, the concrete has a specified slump of 3 inches.

A testing laboratory designs and tests the concrete mixes. Only mixes meeting the design requirements specified for concrete are used.

Forms for concrete are designed as recommended in ACI 347.

3.8.4.6.1.2 Reinforcing Steel

Reinforcing bars for concrete are deformed bars according to Reference 19, Grade 60, and Reference 20. Certified material test reports are provided by the supplier for each heat of reinforcing steel delivered showing physical (both tensile and bend test results) and chemical analysis. In addition, a minimum of one tensile test is performed for each 50 tons of each bar size produced from each heat of steel.

In areas where reinforcing steel splices are necessary and lap splices are not practical, threaded splices or cadwelds are used.

As stated in subsection 3.4.1.1.1, seismic Category I structures that are located below grade elevation are protected against flooding by a waterproofing system and waterstops. This, in conjunction with the 2 inches of concrete cover for the reinforcing steel, provides sufficient protection for the reinforcing steel. Therefore, the use of coated reinforcing steel is not planned.

3.8.4.6.1.3 Structural Steel

Basic materials used in the structural and miscellaneous steel construction conform to the ASTM standards listed in Table 3.8.4-6.

3.8.4.6.1.4 Masonry Walls

There are no safety-related masonry walls used in the nuclear island.

3.8.4.6.2 Quality Control

The quality assurance program is described in Chapter 17. Conformance to Regulatory Guide 1.94 is as described in Section 1.9.

3.8.4.6.3 Special Construction Techniques

Construction techniques for the structural modules are the same as special construction techniques for the containment internal structures discussed previously in subsection 3.8.3.6.1.

3.8.4.7 In-Service Testing and Inspection Requirements

There are no in-service testing or inspection requirements for the seismic Category I structures.

3.8.6 Combined License Information

This section has no requirement for additional information to be provided in support of the Combined License application.

3.8.7 References

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Table 3.8.3-1

SHEAR AND FLEXURAL STIFFNESSES OF STRUCTURAL MODULE WALLS

Case	Analysis Assumption	Shear Stiffness[1,2]				Flexural Stiffness[1,2]			
		48" Wall		30" Wall		48" Wall		30" Wall	
		GA x 10^6 lbs	Ratio	GA x 10^6 lbs	Ratio	EI x 10^9 lbs.in^2	Ratio	EI x 10^9 lbs.in^2	Ratio
1	Monolithic section considering steel plates and uncracked concrete. For shear stiffness this is ($A_c G_c + A_s G_s$).	83.5	1.0	55.8	1.0	47.5	1.0	13.6	1.0
2	Uncracked gross concrete section (full wall thickness considering steel plate as concrete)	73.9	0.89	46.2	0.83	33.2	0.70	8.1	0.60
3	Transformed cracked section considering steel plates and concrete (no concrete tension stiffness)	25.0	0.30	22.6	0.41	22.1	0.47	8.0	0.59
4	~~Steel plates and shear studs with cracked concrete with inclined cracks spaced at 15 inches (includes concrete tensile stiffness betweeen cracks)~~	~~25.6~~	~~0.31~~	~~21.9~~	~~0.39~~	~~25.6~~	~~0.54~~	9.0	0.66

Notes:

1. The shear stiffness, GA, is calculated for the full thickness of wall. The flexural stiffness is calculated per unit length of the wall.
2. Stiffness calculations are based on the following material properties: E_c = 3,605,000 psi, n = 8, ν_c = 0.17, ν_s = 0.30

Table 3.8.3-2

SUMMARY OF CONTAINMENT INTERNAL STRUCTURES MODELS AND ANALYSIS METHODS

Computer program and Model	Analysis Method	Purpose	Concrete Stiffness
3D BSAP finite element of containment internal structures fixed at elevation 82'6"	Response spectrum analysis	To obtain the in-plane seismic forces for the design of floors and walls	Monolithic Case 1
3D ANSYS finite element of containment internal structures fixed at elevation 103'	Static analyses	To obtain member forces in boundaries of IRWST for static loads (dead, live, hydrostatic, pressure)	Monolithic Case 1
	Static analyses	To obtain member forces in boundaries of IRWST for thermal loads	Cracked Case 3
	Harmonic analyses	To evaluate natural frequencies potentially excited by hydrodynamic loads	Uncracked Case 2
	Time history analyses	To obtain dynamic response of IRWST boundary for hydrodynamic loads	Monolithic and cracked Cases 1 & 3
	Response spectrum analyses	To obtain member forces and water slosh height for seismic loads	Monolithic Case 1

Table 3.8.4-1

LOAD COMBINATIONS AND LOAD FACTORS FOR SEISMIC CATEGORY I STEEL STRUCTURES

		Load Combination and Factors								
Combination No.		1	2	3	4	5	6	7	8	9
Load description										
Dead	D	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Liquid	F	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live	L	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Earth pressure	H	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Normal reaction	R_o	1.0	1.0	1.0	1.0				1.0	1.0
Normal thermal	T_o			1.0	1.0				1.0	1.0
Wind	W		1.0							1.0
Safe shutdown earthquake	E_s			1.0				1.0		
Tornado	W_t				1.0					
Accident pressure	P_a					1.0	1.0	1.0		
Accident thermal	T_a					1.0	1.0	1.0		
Accident thermal reactions	R_a					1.0	1.0	1.0		
Accident pipe reactions	Y_r						1.0	1.0		
Jet impingement	Y_j						1.0	1.0		
Pipe impact	Y_m						1.0	1.0		
Stress Limit[(1), (3)] Coefficient		1.0	1.0	1.6	1.6	1.6	1.6	1.7	1.5	1.5

Notes:

1. Allowable stress limits coefficients are applied to the basic stress allowables of AISI or AISC. The coefficients for AISC-N690 are supplemented by the requirements identified in subsection 3.8.4.5.
2. Where any load reduces the effects of other loads, the coefficient for that load is taken as zero unless it can be demonstrated that the load is always present or occurs simultaneously with the other loads.
3. In no instance does the allowable stress exceed $0.7F_u$ in axial tension nor $0.7F_u$ times the ratio of the plastic to elastic section modulus for tension plus bending.
4. Loads due to maximum precipitation are evaluated using load combination 4 with the maximum precipitation in place of the tornado load.

Table 3.8.4-2

LOAD COMBINATIONS AND LOAD FACTORS FOR SEISMIC CATEGORY I CONCRETE STRUCTURES

		Load Combination and Factors								
Combination No.		1	2	3	4	5	6	7	8	9
Load Description										
Dead	D	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05
Liquid	F	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05
Live	L	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3
Earth	H	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3
Normal reaction	R_o	1.7	1.7	1.0	1.0				1.3	1.3
Normal thermal	T_o			1.0	1.0				1.3	1.3
Wind	W		1.7							1.3
Safe shutdown earthquake	E_s			1.0				1.0		
Tornado	W_t				1.0					
Accident pressure	P_a					1.5	1.25	1.0		
Accident thermal	T_a					1.0	1.0	1.0		
Accident thermal reactions	R_a					1.0	1.0	1.0		
Accident pipe reactions	Y_r						1.0	1.0		
Jet impingement	Y_j						1.0	1.0		
Pipe impact	Y_m						1.0	1.0		

Notes:

1. Design for mechanical loads is in accordance with ACI-349 Strength Design Method for all load combinations. Design for combinations including thermal loads is desccribed in subsection 3.8.3.5.3.4.

2. Where any load reduces the effects of other loads, the corresponding coefficient for that load is taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with the other loads. Otherwise the coefficient for the load is taken as zero.

3. Loads due to maximum precipitation are evaluated using load combination 4 with the maximum precipitation in place of the tornado load.

Table 3.8.4-3

ACCEPTANCE TESTS FOR CONCRETE AGGREGATES

Method of Test	Designation
Organic impurities in sand	ASTM C 40-1984
Effect of organic impurities on strength of mortar	ASTM C 87-1990
Soundness of aggregates	ASTM C 88-1983
Material finer than No. 200 sieve	ASTM C 117-1987
Specific gravity and absorption - coarse aggregates	ASTM C 127-1988
Specific gravity and absorption - fine aggregates	ASTM C 128-1988
Los Angeles abrasion of small-size coarse aggregates	ASTM C 131-1989
Sieve analysis	ASTM C 136-1984
Friable particles	ASTM C 142-1978
Potential reactivity of aggregates (chemical)	ASTM C 289-1987
Petrographic examination of aggregates	ASTM C 295-1988
Resistance to degradation of large-size coarse aggregates by abrasion and impact in the Los Angeles machine	ASTM C 535-1989
Potential alkali reactivity of carbonate rocks for concrete aggregates	ASTM C 586-1986
Resistance of concrete to rapid freezing and thawing	ASTM C 666-1984
Flat and elongated particles	CRD C 119-1953

Table 3.8.4-4

CRITERIA FOR WATER USED IN PRODUCTION OF CONCRETE

Requirements and Test Method	Criteria
Compressive strength ASTM C 109-1988	Reduction in strength not in excess of 10 percent
Soundness ASTM C 151-1989	Increase in length limited to 0.10 percent
Time of setting ASTM C 191-1982	± 10 min for initial set, ± 1 hour for final set

Table 3.8.4-5

TYPES OF WATER REDUCING AGENTS USED IN PRODUCTION OF CONCRETE

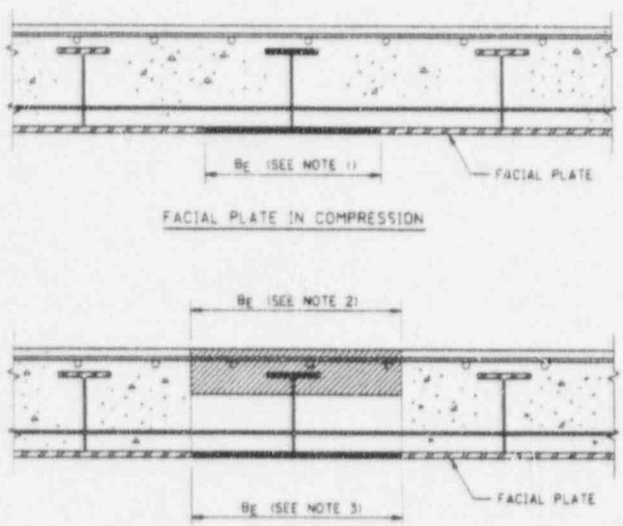
Concrete Placing Temperature	Placement Description	WRA[1] Type
70°F or less	For normal conditions	A
70°F or less	For additional retardation for members with least dimension of 3.0 feet or more	D
More than 70°F	For members except floor slabs	D
More than 70°F	For floor slabs	A

Note:
1. Water reducing agent

Table 3.8.4-6

MATERIALS USED IN STRUCTURAL AND MISCELLANEOUS STEEL

Standard	Construction Material
ASTM A1-84	Carbon steel rails
ASTM A36/A36M-89	Rolled shapes, plates, and bars
ASTM A108-90	Weld studs
ASTM A123-89a	Zinc coatings (hot galvanized)
ASTM A240-90	Nitronic 33 stainless steel (designation S2400, Type XM-29)
ASTM A307-90	Low carbon steel bolts
ASTM A325-90	High strength bolts
ASTM A354-90	Quenched and tempered alloy steel bolts (Grade BC)
ASTM A588-88	High-strength low alloy structural steel



FACIAL PLATE IN TENSION

EFFECTIVE SECTIONS FOR FLOOR MODULES

NOTES:

1. FOR FACIAL PLATE IN COMPRESSION, b_e, IS DETERMINED PER SECTION 3A.5.5.2. — 3.8.3.5.4.2
2. EFFECTIVE WIDTH OF CONCRETE, b_e, IS DETERMINED PER SECTION Q 1.11.1 OF AISC N690.
3. FOR FACIAL PLATE IN TENSION, b_e, IS TAKEN TO BE ONE HALF OF THE DISTANCE TO THE ADJACENT BEAMS.

Figure 3.8.3-13

Effective Sections For Floor Modules

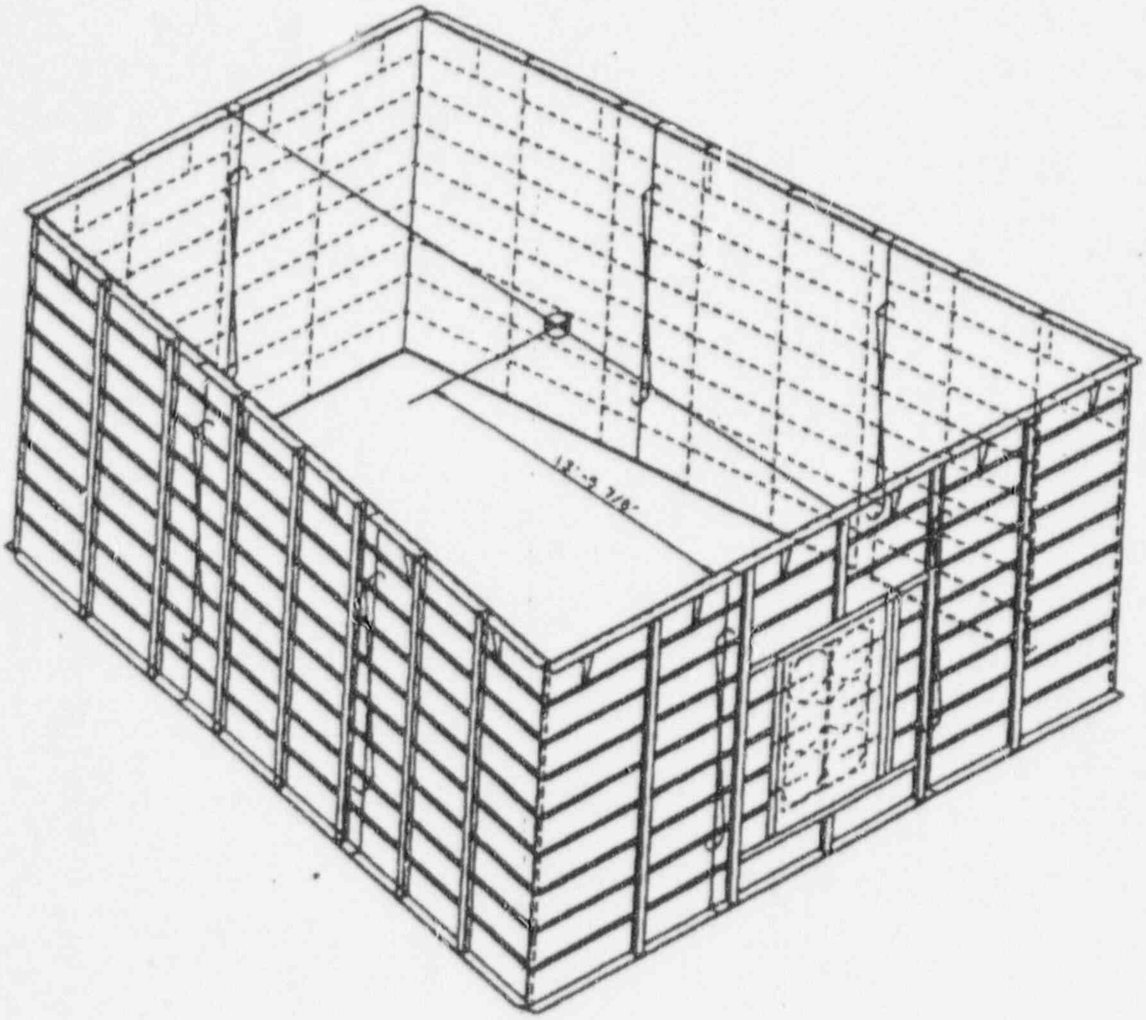


FIGURE 3.8.3_16

"ISOMETRIC OF A TYPICAL "L" MODULE ASSEMBLEY"

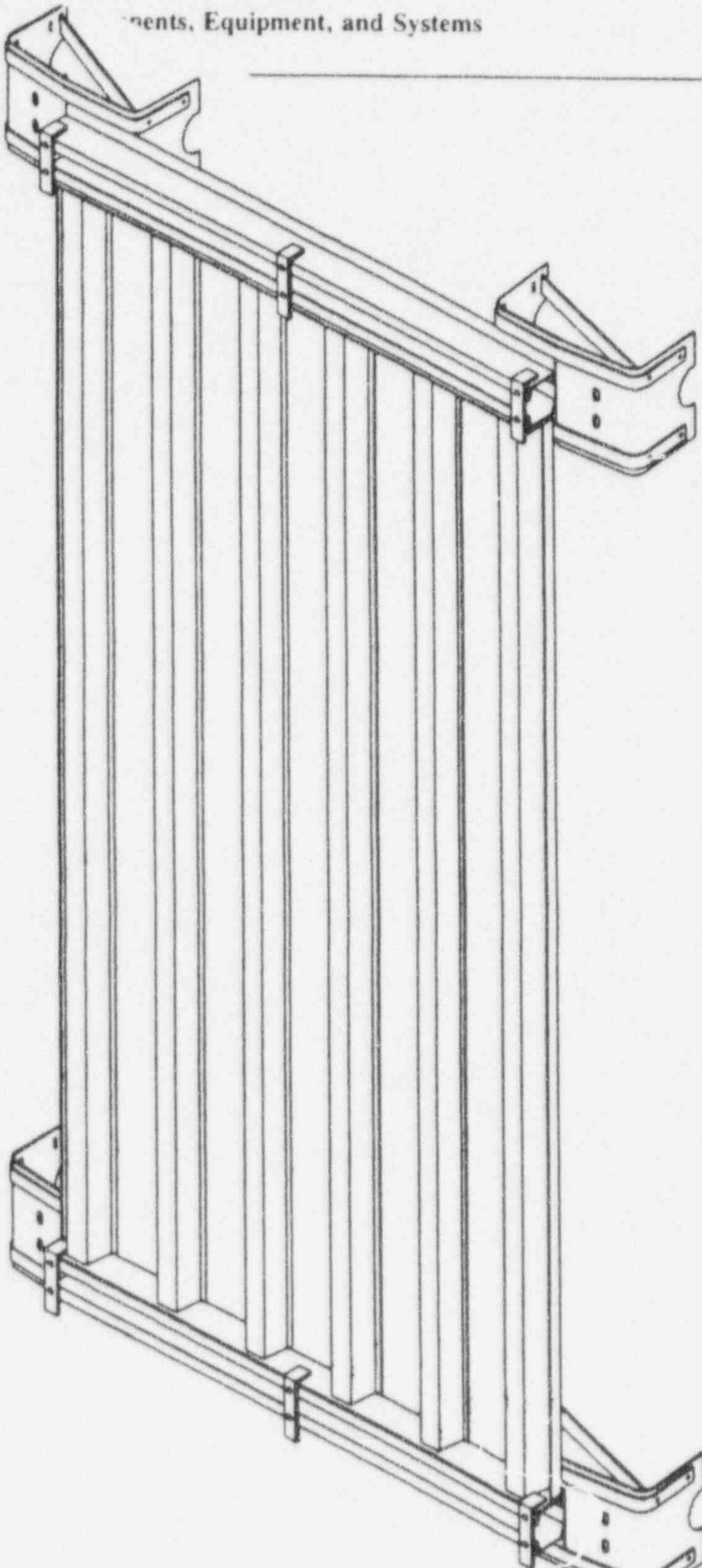


Figure 3.8.4-1 (Sheet 3 of 4)

Containment Air Baffle
Typical Panel on Cylinder