The concrete shell inner surface is lined with a minimum 1/4-in. carbon steel plate that is anchored to the concrete shell and dome to provide the required pressure boundary leak tightness. Areas around penetrations, support brackets, inner walls, and heavy components bases have thickened steel liner plates. The other items integrally welded to the liner form part of the overall pressure boundary, including but not limited to, the equipment hatch at elevation 86 ft, 3 in. (with ground level elevation 2 ft, 8 in.), an airlock at elevation 28 ft, 10 in. and a personnel airlock at elevation 80 ft, 2 in., various piping and electrical penetrations, and miscellaneous supports that are embedded in the concrete shell such as the polar crane brackets. The liner plate system is not designed or considered as a structural member in providing for the overall PCCV load resistance. The liner plate system is attached to the PCCV shell with an anchorage system that is depicted on Figure 3.8.1-2. In the cylinder portion of the PCCV, the liner is anchored with WT5x11s running vertically at a pitch of 1.6° (approximately 25 in. spacing along the inside face of the PCCV shell), and stiffened with 1/2 in. by 6 in. rib plates running horizontally in the hoop direction. In the dome portion of the PCCV, except the lowest panel portion where the cylinder liner anchorage system is also adopted, the upper portion of dome liner is anchored with 3/8 in. by 6 in. rib plates (spaced at approximately 32-1/4 in. maximum) which are oriented in a radial pattern originating at the dome apex. The rib plates are stiffened with 5 in. by 3 in. by 1/4 in. angles running horizontally in the hoop direction, spaced approximately at 34 in. ~~maximum.~~ Where acceptable based on the results of design analyses performed for the liner-and-anchorage system (discussed in Subsection 3.8.1.4), the liner anchors are connected to the liner using discontinuous welds such as stitched fillet welds. MIC-03-03-00066

Figure 3.8.1-1 provides the overall dimensions of PCCV and Figure 3.8.1-5 provides GA of prestressing tendons and conventional reinforcement of the PCCV shell. Figure 3.8.1-3 and 4 also show the liner anchorage system arrangement.

3.8.1.1.2 Equipment Hatch

Figure 3.8.1-6 provides the equipment hatch general layout. The hatch is located at centerline elevation 86 ft, 3 in., azimuth 40 degrees, and is a 27 ft, 11 in. diameter spherical dish with a convex profile projecting into the PCCV volume. The containment internal pressure places the hatch head into compression against a double-sealed seat on the frame. The space between the two seals is capable of pressure testing for leakage across either seal.

A lifting rig with an electrically powered hoist is provided to disengage, raise, and store the hatch in a secure position above the opening during outages. When required to seal the opening, the hatch is lowered back by hoist, repositioned, refastened, and pressure tested for leaks.

3.8.1.1.3 Personnel Airlocks

Figure 3.8.1-7 provides the general layout for the two personnel airlocks. The lower airlock at centerline elevation 28 ft, 10 in. is located at azimuth 24 degrees, and upper airlock at centerline elevation 80 ft, 2 in. is located at azimuth 120 degrees. The airlock inside diameter is 8 ft, 6-3/8 in.

Determining Prestressing Forces for Inspection of Prestressed Concrete Containments, RG 1.35.1 U.S. Nuclear Regulatory Commission, Washington, DC, July 1990 (Reference 3.8-6).

Concrete Containment, NUREG-0800 SRP Section 3.8.1, U.S. Nuclear Regulatory Commission, Washington, DC, March, 2007 (Reference 3.8-7).

3.8.1.3 Loads and Load Combinations

The PCCV is designed for the loads and load combinations defined in the ASME Code, Section III (Reference 3.8-2), in Article CC-3200 “Load Criteria” and Table CC-3230-1 “Load Combinations and Load Factors,” except as noted in RG 1.136 (Reference 3.8-3) Regulatory Position 5:

- The post LOCA flooding combined with the OBE set at one-third or less of the plant SSE is eliminated, since the load combination is less severe than the post-LOCA flooding combined with a SSE.

- ASME Code, Section III, Subarticle CC-3720 is satisfied by addressing an accident that releases hydrogen generated from 100% fuel clad-coolant reaction accompanied by hydrogen burning, including the effects of temperature and prestress. See Subsection 3.8.1.3.2.2 for further discussion of this design condition.

Load combinations and factors based on ASME Table CC-3230-1 are presented in Table 3.8.1-2. Load combinations involving wind and tornado have been determined to be less severe than other cases through comparison calculations to the design-basis earthquake loads and, therefore, load combinations involving wind and tornado are not used in the full detailed design analyses of the overall PCCV structure and its liner.

3.8.1.3.1 Loads

The following is a brief description of loads unique to the PCCV and liner used in Table 3.8.1-2 for design and analysis. Subsection 3.8.4.3 gives definitions and descriptions of other loads based on the ACI 349-06 (Reference 3.8-8) and AISC N690-1994, including Supplement 2 (Reference 3.8-9), definitions and descriptions, which are consistent with the ASME Code, Section III. MIC-03-03-00066

- **Prestress Load**

  For purposes of the US-APWR PCCV design, prestress is defined as the load on the PCCV dome and cylinder walls that, when applied by mechanical force from tendons after the concrete has hardened, results in the introduction of internal stresses to reduce potential tensile stresses in concrete resulting from other loads. The initial prestress governs the cylinder wall and dome thickness. It is not governed by radiation shielding. The minimum prestress level including all losses after design life applied to the PCCV is 1.20 times the design pressure. ~~The minimum prestress level including all losses after design life applied to the PCCV is 1.20 times the design pressure.~~ MIC-03-03-00066

- **Design-Basis Accident Pressure (P_a) and Test Pressure (P_t)**

The DBA pressure is 68 psig. The DBA pressure is increased for structural design purposes using load factors as shown in ASME Code, Section III, Table CC-3230-1, depending on the particular load combination considered.

The structural integrity test pressure P_t is 1.15 times the design pressure (P_t = 78.2 psig).

External or internal events such as containment spray actuation may induce a negative pressure on the PCCV. See Chapter 6 for further discussion. Therefore, the PCCV is designed for a negative pressure of 3.9 psig as a separate event.

With respect to accident pressure loads, 10 CFR 50.44 (Reference 3.8-10) requires that an analysis be performed that demonstrates that the containment structural integrity is maintained under loads resulting from combustible gases generated from metal-water reaction of the fuel cladding. In determining loads from combustible gases, the US-APWR design follows the guidance of RG 1.7 (Reference 3.8-11), in determining and analyzing the design accident pressure loads.

- **Thermal Loads (T_o) and Accident Thermal Loads (T_a)**

The normal operating environment inside and outside the PCCV is specified in Table 3.8.1-3 and Figures 3.8.1-9 through 3.8.1-13. Normal thermal loads for the exterior walls and roofs are addressed in the design of the PCCV. For the effects of transient loads such as T_a, the overall behavior of the PCCV is first determined. A portion of the PCCV shell can then be analyzed for local effects using the results obtained from the global analysis as boundary conditions, for example at penetrations and/or at its anchorages to the basemat.

During normal operation, a linear temperature gradient develops across the PCCV wall thickness. After a LOCA, however, the sudden increase in temperature in the liner and adjacent concrete produces a nonlinear transient temperature gradient. The temperature versus time is considered when combining with accident pressure in the specified load combinations, and worst case temperature gradients within the volume of the PCCV are used in the thermal analyses as discussed in Subsection 3.8.1.4. The calculated thermal gradients are developed in a manner consistent with the methodology of ~~ACI 349~~ACI 349-06 (Reference 3.8-8) Appendix ~~A~~E and its corresponding commentary. MIC-03-03-00066

- **Earthquake Loads (E_{ss})**

For the PCCV, earthquake loads E_{ss} and the seismic analysis are discussed and summarized in Section 3.7. There are two horizontal and one vertical earthquake components that require combination as discussed in Subsection 3.7.2.6.

engineered safety features from the effects induced by an accident, such as jet impingement forces and whipping pipes.

3.8.3.1 Description of the Structures Internal to Containment

3.8.3.1.1 Reactor Vessel Support System

The RV support system consists of eight steel support pads which are integrated with the inlet and outlet nozzle forgings. The support pads are placed on support brackets, which are supported by an embedded steel structure on the primary shield wall elevation 35 ft, 10.87in. The support system is designed for operating and accident load cases caused by seismic and postulated pipe rupture, including LOCAs. The supports are formed by sliding surfaces between the shim plates and support pads to allow radial thermal growth of the RCS and RV. The vessel position is maintained unchanged by controlling the horizontal load through the support brackets and the base plate. Figure 3.8.3-1 provides the detail of the RV supports and relationship with the primary shield wall.

3.8.3.1.2 Steam Generator Support System

The SG support system consists of an upper shell support structure at centerline elevation 96 ft, 7 in., an intermediate shell support structure at centerline elevation 75 ft, 5 in., and a lower shell support structure at centerline elevation 45 ft, 7.64 in.

The upper and intermediate shell supports are lateral restraints utilizing snubbers attached to structural steel brackets, while the lower support structure is constructed entirely of structural steel and provides both vertical and lateral support. All support systems are designed considering thermal expansion of piping. The support system also restrains horizontal movement of the SG in the event of earthquake or other DBAs.

Four columns ~~support~~transfer the vertical loads of the SG ~~from~~to the reinforced concrete slab at elevation 25 ft, 3 in. The upper and lower ends of the columns are pin-jointed to permit movement of the SGs caused by thermal expansion of piping. Figure 3.8.3-2 depicts the SG support system. MIC-03-03-00066

3.8.3.1.3 Reactor Coolant Pump Support System

Each RCP support system consists of a lateral support structure, and three support columns.

The lateral support structure at centerline elevation 42 ft, 7.69 in. is constructed entirely of structural steel. Both support structures are designed considering thermal expansion of piping. The support structure also restrains horizontal movement of the RCPs in the event of an earthquake or other DBAs.

The three support columns carry the vertical loads of the RCP from the reinforced concrete slab at elevation 25 ft, 3 in. The upper and lower ends of the supports are pin-jointed to permit movement of the pumps caused by thermal expansion of piping. Figure 3.8.3-3 depicts the RCP support system.

3.8.3.1.4 Pressurizer Support System

The pressurizer is supported by an upper support structure and a lower support skirt. The upper support structure constructed of four structural steel struts at centerline elevation 110 ft, 9 in. does not restrain movement by thermal expansion, but restrains horizontal movements in the event of design-basis earthquakes or accidents. The lower support structure supports the vertical load through a continuous structural steel skirt welded to the bottom of the pressurizer supported at elevation 59 ft, 1 in. Figure 3.8.3-4 depicts the pressurizer support system.

3.8.3.1.5 Primary Shield Wall

The RV is located at the center of the PCCV. Primary shield walls form the perimeter around the RV, which also serve to support the RV at elevation 35 ft, 10.87in. The top of primary shield wall elevation is 46 ft, 11in. The general ~~primary~~arrangement drawings in Chapter 1 show the location and configuration. Isometrics of the primary shield walls are shown in Figure 3.8.3-5. MIC-03-03-00066

The primary shield wall and other walls inside containment are fabricated as steel-concrete (SC) module walls. The modules are formed using permanently placed carbon steel faceplates and web-plates with a nominal thickness of 1/2 in. The faceplates, connected by tie-bars, fabricated from ~~solid~~ carbon steel ~~round~~deformed reinforcing bars, or by carbon steel web-plates, also function as formwork for concrete placed in the interior. The primary purpose of the tie-bars and web-plates is to stiffen and hold together the faceplates during handling, erection, and concrete placement, and to provide out-of-plane shear strength. The nominal pitch of the tie-bar is 24 in. for the ~~secondary shield~~SC module walls. The primary functions of the web-plates are to mitigate faceplate stress concentration, maintain the SC module configuration, and stiffen corners of faceplates. Shear studs are welded to the inside faces of the steel faceplates. Where SC modules intersect, web-plates are installed in-line with faceplates to maintain continuity across the point of intersection. The nominal pitch of studs is 8 in. to 12 in. in both directions. Face plates are welded to adjacent plates with full penetration welds so that the weld is at least as strong as the plate. The SC module walls are welded at the base to a continuous embedded plate in the basemat. After erection, concrete is placed between the faceplates. Typical details of the SC modules are shown in Figure 3.8.3-7. MIC-03-03-00066 MIC-03-03-00066

3.8.3.1.6 Secondary Shield Walls

The secondary shield walls surround the primary loops from the SG compartments. SC modules also form supports for intermediate floors and operating floors. The secondary shield walls are a series of walls that enclose the SGs and the pressurizer. Each of the four secondary shield wall compartments provides supports and houses a SG and RCL piping. The GA drawings in Chapter 1 show the location and configuration. Isometrics of secondary shield walls are shown in Figure 3.8.3-5.

3.8.3.1.7 Refueling Cavity

The cavity space directly above the RV and between ~~concrete~~SC module walls to the north is referred to as the refueling cavity. The refueling cavity connects to the fuel transfer tubes that penetrate the north end of PCCV. The floor of the refueling cavity MIC-03-03-00066 varies in elevation from 19 ft, 4 in. to 46 ft, 11 in. The top of the refueling cavity is 76 ft, 5 in.

The walls of the refueling cavity are formed by SC modules, which are lined with stainless steel over the 1/2-in. thick carbon steel plates, referred to as “clad steel.” The ceiling and floor slabs are also lined with clad steel.

3.8.3.1.8 RWSP

The RWSP is located at the lowest part of the PCCV. The RWSP is formed by wall of SC modules using clad steel. A floor at elevation 3 ft, 7 in. is formed of clad steel in a layer of concrete that covers the containment liner and basemat. The ceiling is similarly lined with stainless steel. Subsection 6.2.1.1 provides a description of the RWSP layout and design features.

3.8.3.1.9 Interior Compartments

The containment internal structure includes several subcompartments designed to provide containment, radiation shielding, and protection of safety-related components. These compartments are formed by the secondary shield walls surrounding the primary loops from the SGs. They also protect the containment from postulated pipe ruptures inside the containment. These SC wall modules also form supports for intermediate floors and the operating deck at elevation 76 ft, 5 in. The walls are designed for load cases including earthquake and DBAs.

Subcompartments and/or rooms comprising the containment internal structure are summarized as follows:

- reactor cavity — EL. -9 ft, 2 in.
- containment drain sump room — EL. 9 ft, 6 in.
- letdown heat exchanger room — EL. 25 ft, 3 in.
- regenerative heat exchanger room — EL. 50 ft, 2 in.
- excess letdown heat exchanger room — EL. 50 ft, 2 in.

Labyrinths are provided beside the shield wall openings at several elevations for radiation protection, which consist of SC modules and reinforced concrete walls, floors, and ceilings.

Reinforced concrete slabs are used for the floor above the RWSP at elevation 25 ft, 3 in., the intermediate floor at elevation 50 ft, 2 in., and the operating floor at elevation 76 ft, 5 in. The floors are shown on the GA drawings in Chapter 1. The floor ~~is~~ at elevation 25 ft, 3 in.~~, and~~ is supported by the primary shield wall, the secondary shield wall, and the RWSP. MIC-03-03-00066 The floors at elevations 50 ft, 2 in. and 76 ft, 5 in. are supported by the secondary shield wall and the structural steel framing (beams and columns) arranged between the secondary shield wall and the PCCV. The floors consist of reinforced concrete slabs, MIC-03-03-00066 placed on steel beams and deck plate.

3.8.3.1.10 SC Modules

Figure 3.8.3-5 provides isometric views of the SC modules.

The module framework, consisting of the steel faceplates prior to concrete placement, is positioned on the supporting reinforced concrete basemat. The SC modules are anchored to the basemat through reinforcement doweled with the slab. Seaming of adjacent plates is accomplished using full penetration welding that maintains full design strength of the plate units. The interior of the modular unit is filled with concrete to complete the installation process. Figure 3.8.3-6 depicts the containment internal structure compartment wall layout and configuration. Figure 3.8.3-7 provides typical details for the SC module construction including connection details and anchorage connection details to the reinforced concrete basemat.

3.8.3.1.11 Polar Crane Supports

An internal polar crane is supported by the PCCV. A continuous crane girder transfers the polar crane loads to the PCCV wall. Refer to Subsection 3.8.4.3 for loads applicable to the polar crane supports. Figure 3.8.3-8 depicts the polar crane supports layout and construction.

3.8.3.1.12 Structural Steel Framing

Structural steel framing within the interior of PCCV is primarily for support of floor slab, equipment, distribution systems such as piping, valves, and cable trays, and access platforms. Service platforms and secondary intermediate floors consist of steel grating or checkered plate supported by structural steel framing. All structural steel members are capable of resisting the loads and load combinations to which they may be subjected. MIC-03-03-00066

3.8.3.2 Applicable Codes, Standards, and Specifications

Refer to Subsection 3.8.4.2 for industry standards applicable to the design and construction of seismic category I structures inside containment. Other codes, standards and specifications applicable to materials, testing and inspections are identified in Subsections 3.8.4.6 and 3.8.4.7.

3.8.3.3 Loads and Load Combinations

Typical loads and load combinations are detailed in Subsection 3.8.4.3. Load combinations to be utilized for the design of the containment internal structure include hydrostatic, pressure, and thermal loads as summarized below. Hydrostatic loads reflect the water inventory and its location during various plant conditions.

Seismic category I concrete structures are designed for impulsive and impactive loads in accordance with the ACI 349-06 Code (Reference 3.8-8), ~~and special provisions of Appendix C of the same code,~~ with exceptions given in RG 1.142 (Reference 3.8-19). Impactive and impulsive loads must be considered concurrent with seismic and other loads (i.e., dead and live load) in determining the required load resistance of structural elements. MIC-03-03-00066 normal operating liquid load. In the event of a SSE, the containment internal structure is designed with the water inventory in any one of the above locations.

~~Out-of-plane seismic loads due to the water in the RWSP are applied as linearly increasing from 22.2 psi at the ceiling to 32.2 psi at the base of the pit SC module walls. These pressures include the seismic sloshing (convective) pressures, as well as the seismic inertia (impulsive) pressures.~~ The RWSP design also considers the hydrodynamic response of the refueling water under seismic excitation. The manner in which the impulsive and convective response components are considered is discussed in Subsection 3.8.3.4.2. MIC-03-03-00066

3.8.3.3.3 Accident Pressure Load (P_a)

Accident pressure loads within or across a compartment and/or building are considered in the design. Differential pressure is generated by postulated pipe rupture and includes the dynamic effects due to pressure time-history. The containment internal structure subcompartments are designed to the pressures shown in Table 3.8.3-2 and identified on Figure 3.8.3-9. These pressures are combined by SRSS with SSE loads, including sloshing loads, or by using more conservative combinations. The water inventory is assumed to be in the RWSP. Steel floors with grating need not be designed for differential pressure.

3.8.3.3.4 Operating Thermal Loads (T_o)

The normal operating environment inside and outside the PCCV is specified in Table 3.8.1-3. Under the normal operating condition, the primary shield wall, and the secondary shield wall (in the proximity of the main steam and feedwater pipes) experience temperature rises, including temperature distribution through the wall thicknesses. The loads resulting from these thermal gradients provided in Table 3.8.1-3 are combined with other loads for the containment internal structure as specified in the load combinations in Table 3.8.4-3.

3.8.3.3.5 Accident Thermal Load (T_a)

Thermal loads due to temperature gradients caused by the postulated pipe breaks are considered in the design. The temperature gradients are calculated using the temperatures corresponding to LOCA and MSLB, or a spent fuel pit accident, and are presented in Table 3.8.1-3. Local areas are designed for the elevated temperature effects and the loads resulting from the postulated accidents.

~~During a postulated pipe break, the concrete walls in the vicinity experience temperature increases at the surface following the accident. However, since the concrete is a poor heat conductor, considerable time must elapse before the entire wall experiences an increase in the temperature. Other loads such as accident pressure load, seismic load, etc., are of very short duration. This difference in the transients is considered when combining~~ ~~T_a~~ ~~with other loads.~~ MIC-03-03-00066

Temperatures during an accident do not exceed 350°F at the surface. However, local areas are allowed to reach 650°F from steam or water jets in the event of a pipe failure.

~~General design requirements for concrete subject to thermal loads may be found in Appendix A of ACI 349 (Reference 3.8-8).~~ MIC-03-03-00066

3.8.3.3.6 Accident Thermal Pipe Reaction (R_a)

Pipe and equipment reactions under thermal conditions are generated by the postulated pipe break and includes R_o (see Subsection 3.8.4.3).

3.8.3.3.7 Reaction Due to Pipe Ruptures (Y_r)

The load on a structure generated by the reaction of a ruptured high-energy pipe during the postulated event includes an appropriate dynamic load factor. The time dependent nature of the load and the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the effects of Y_r.

3.8.3.3.8 Jet Impingement (Y_j)

The load on a structure generated by the jet impingement from a ruptured high-energy pipe during a postulated event includes an appropriate dynamic load factor. The time-dependent nature of the load and the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the effects of Y_j. The dynamic load factor is calculated using a long duration step function for the load. The target resistance is idealized as bilinear elasto-perfectly plastic.

3.8.3.3.9 Impact of Ruptured Pipe (Y_m)

The load on a structure or a pipe restraint resulting from the impact of a ruptured high-energy pipe during the postulated event includes an appropriate dynamic load factor. The type of impact (i.e., plastic, elastic), together with the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the impact.

3.8.3.4 Design and Analysis Procedures

Concrete and steel composites are commonly used in construction because of the inherent benefits of the steel tensile strength in concrete sections. The fundamental difference between the conventional reinforced concrete and SC modular construction is that the reinforcement and formwork of conventional reinforced concrete is replaced by the steel faceplates of the SC. For walls within the US-APWR, additional benefits are realized by providing formwork during construction, improved construction staging and schedule, continuous steel surfaces for welding of field attachments, and impactive/impulsive capacities as applicable. If required to be qualified as radiation shielding, the requirements and recommended practices are maintained in accordance with RG 1.69 (Reference 3.8-20). ~~Assurances that SC modules for interior compartments of the US-APWR meet or exceed the requirements of ACI 349 (Reference 3.8-8) are provided by the following design and analysis procedures.~~ Assurances that SC modules for interior compartments of the US-APWR meet or exceed the strength requirements of ACI 349-06 (Reference 3.8-8) are provided by the following design and analysis procedures. DCD_03.07.02-35 MIC-03-03-00066

The ~~permanently-placed~~stay-in-place faceplates act as forms during the placement of concrete. Plate stresses occurring during concrete placement are conservatively assumed to be simply supported spans between tie bars. Faceplates fabricated from A572 high-strength low-alloy Columbium-Vanadium structural steel provide minimum yield strength of 50 ksi or greater, and maintain out-of-plane plate deflection to within code allowables. MIC-03-03-00066 MIC-03-03-00066

~~Stresses are induced on faceplates acting as formwork during concrete placement, however, they are not applicable during other load combinations.~~ After concrete curing, the SC module performs as a composite section of concrete with outer faceplates acting as ~~either~~ compression ~~or~~and tension reinforcement. The composite section is designed to allow faceplate yielding prior to the concrete reaching its strain limit of 0.003 in. per in. ~~Under tensile straining, the residual stress that was initiated by concrete placement is naturally relieved. While the formwork is permanently placed, the stresses generated by construction activities are therefore not applicable during other load combinations.~~ MIC-03-03-00066 MIC-03-03-00066 MIC-03-03-00066

The SC module forms a composite section once the concrete has reached sufficient strength, consisting of steel faceplates that ~~carry in-plane tension or compression from axial loads and out-of-plane bending~~resist in-plane shear and axial tension, as well as out-of-plane moments. Structural behavior of composite sections used as SC modules inside containment is, therefore, similar to conventional concrete reinforced by steel. Research regarding in-plane loading of composite sections consisting of steel faceplates and concrete infill is described in “Experimental Study on Steel Plate Reinforced Concrete Shear Walls with Joint Bars” (Reference 3.8-21) and “A Compression and Shear Loading Test of Concrete Filled Steel Bearing Wall” (Reference 3.8-22). Out-of-plane loading research is provided by “Experimental Studies on Composite Members for Artic Offshore Structures, Steel/Concrete Composite Structural Systems” (Reference 3.8-23), “Strength of Composite System Ice-Resisting Structures, Steel/Concrete Composite Structural Systems” (Reference 3.8-24), “Design and Behaviour of Composite Ice-Resisting Walls, Steel/Concrete Composite Structural Systems” (Reference 3.8-25), and “Tests on Composite Ice-Resisting Walls Steel/Concrete Composite Structural Systems” (Reference 3.8-26). In addition, “1/10th Scale Model Test of Inner Concrete Structure Composed of Concrete Filled Steel Bearing Wall” (Reference 3.8-27) provides research regarding in-plane loading of composite sections, and supports the conclusion there are significant advantages of SC modules over conventional reinforced concrete, such as high strength, high ductility, and less decrease of stiffness, over reinforced concrete elements of equivalent thickness and reinforcement ratios. Further, “A Study on the Structural Performance of SC Thick Walls” (Reference 3.8-69) reflects the experimental results of a 1/6th scale test which demonstrates the seismic behavior of the primary shield wall. MIC-03-03-00066 DCD_03.07.02-35

Shear connector spacing to plate thickness ratio is sufficient to prevent local buckling and allow development of full compressive strength. MIC-03-03-00066

~~Methods of analysis for the SC modules are similar to the methods used for reinforced concrete. Table 3.8.3-3 summarizes the modeling and analytical methods used for SC modules inside containment. The determination of section properties are in accordance with ACI 349 (Reference 3.8-8). For all loads, the analyses use the monolithic (uncracked) stiffness of each concrete element. For thermal loads, design forces are~~ DCD_03.07.02-35

~~calculated by multiplying the reduction ratio α, considering the reduction of stiffness by cracking to the result values of above analysis. The reduction ratio α is set to 0.5 as the reduction ratio of flexural stiffness caused by cracking for the typical member. For example, the flexural stiffness of cracked section for 48 in. wall with 0.5 in. plates assuming zero tensile strength of concrete is 22.2 by 10^9 lbs-in.^{2}/in., and the reduction ratio calculated by this value and elastic flexural stiffness (47.5 x 10^9 lbs-in.^{2}/in.) is 0.47.~~ DCD_03.07.02-35

~~Table 3.8.3-4 summarizes axial, in-plane shear and out-of-plane flexural stiffness properties of the 56-in., 48-in. and 39-in. walls based on a series of different assumptions. The stiffnesses are expressed for unit length and height of each wall.~~

~~Case 1 assumes monolithic behavior of the steel plate and uncracked concrete. This stiffness is the basis for the stiffness of the SC modules in the seismic analyses and the stress analysis.~~

~~Case 2 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.~~

Stiffness and Damping for Analysis: MIC-03-03-00066

The containment internal structure is unique among the R/B complex structures in that it is comprised of a number of different structural types. The structural types include composite SC walls of varying thickness, massive reinforced concrete sections, and reinforced concrete slabs. These structures experience varying levels of stress and resultant concrete cracking under the seismic and accident thermal loading applied to the containment internal structure. Each structural type exhibits unique stiffness and damping characteristics before and after cracking. Thus, it is not appropriate to apply a uniform stiffness reduction to the entire containment internal structure for the SSI analyses of the R/B complex. Each structural component is assigned stiffness and damping values appropriate for its structural type and estimated cracking levels. This assignment is simplified by grouping structural components into six structural categories with common behavior. Stiffness and damping values are then defined for each category under two basic loading conditions that encompass the full range of stresses and resultant cracking anticipated for the containment internal structure seismic response. DCD_03.07.02-35 MIC-03-03-00066

The six structural categories defined for stiffness and damping characterization are described below and summarized in Table 3.8.3-4. The values are derived from supporting experimental data for the SC modules and from industry standards for reinforced concrete structures. Plan and elevation views illustrating the use of each of the six structural categories are presented in Figures 3.8.3-12 through 3.8.3-18. DCD_03.07.02-35

Overall thicknesses of the single-celled SC walls vary from 36” to 67”, while the multi-celled primary shielding walls have overall thickness in excess of 9’-11”. The range of experimental data establishing the composite stiffness characteristics of SC walls is applicable to sections with overall thickness less than or equal to 56” and steel plate reinforcement ratio (ρ) greater than 1.5%. MIC-03-03-00066

$$\rho = 2 \cdot t_p / T > 0.015$$

Where MIC-03-03-00066

t_p = plate thickness,

T = overall wall thickness

The SC walls are separated into three categories, as follows:

Category 1: SC Walls with thickness less than or equal to 56 in. These SC walls have material and geometric parameters that are within the range of the experimental database. This category includes the majority of the secondary shielding walls in the containment internal structure. The most common SC wall is 48 in. thick with 0.5 in. thick steel faceplates.

Category 2: SC Walls with thickness greater than 56 in. This category includes a relatively small portion of the containment internal structure SC walls with thicknesses ranging from 58.5 in. to 67 in.

Category 3: Primary Shield Walls. The primary shield walls below elevation 35'-11" range in thickness from 9'-11" to 15'-4". They have a multi-cellular arrangement comprised of two steel faceplates, a mid-thickness steel plate, and numerous transverse web plates.

Non-SC structural components are separated into three additional categories, as follows:

Category 4: Reinforced concrete slabs. Standard reinforced concrete floor slabs are used at various elevations throughout the containment internal structure.

Category 5: Massive reinforced concrete. This category includes the thick reinforced concrete blocks at the base of the containment internal structure that support the steam generators and reactor coolant pumps. These blocks are nominally 8 to 32 feet deep and are anchored to the basemat of the reactor building complex with steel reinforcement.

Category 6: Steel structures with nonstructural concrete infill. These structures consist of steel plates or steel shape grillages with nonstructural concrete provided for shielding purposes.

~~The report "1/10th Scale Model Test of Inner Concrete Structure Composed of Concrete Filled Steel Bearing Wall" (Reference 3.8-27) provides damping of the SC modules based on the cyclic load tests of an containment internal structure model. The SC module exhibited 5 % equivalent viscous damping at the design load level. This remained nearly constant up to the load level where yielding was reached in the steel plate. Therefore, dynamic analyses as described in Subsection 3.7.1 are performed using 7 % damping for the reinforced concrete and 5 % for the SC modules.~~

Discussion of Basic Loading Conditions:

The containment internal structure seismic analysis must consider the stiffness and damping levels appropriate for two basic loading conditions: MIC-03-03-00066

Condition A: Seismic + Operating Thermal. The normal operating thermal loading involves ambient temperatures of 105°F to 120°F, which are not anticipated to cause cracking that would significantly reduce the stiffness of the SC modules or any of the reinforced concrete structures. The operating temperature of the reactor cavity is 150°F, such that a linear temperature distribution is postulated through the nominally 10-ft thickness of the primary shielding walls, varying from 150°F at the interior face to 105-120°F at the exterior face. This shallow linear gradient is not anticipated to cause significant cracking of the primary shielding walls. Thus, the stiffness for Condition A is estimated by evaluating stresses resulting from the seismic loading condition only. MIC-03-03-00066 DCD_03.07.02-35

Condition B: Seismic + Accident Thermal. The accident thermal conditions postulated involve initial temperatures of 580°F on the pipe-rupture side of a given wall, with an immediate increase of temperature on the opposite face to 300°F. Within approximately 1000 seconds (17 minutes) the two face temperatures equilibrate to 300°F within 10 seconds, which sets up a parabolic (U-shaped) temperature distribution through the thickness of the SC walls. MIC-03-03-00066

This distribution will cause through-thickness cracks in the SC walls. These cracks will reduce the in-plane shear stiffness, cause overall thermal deformations and out-of-plane flexural cracking at restraints.

Estimated Stiffness for Each Category and Loading Condition: DCD_03.07.02-35

Category 1, Condition A: An assessment of the maximum seismic in-plane shear demands in each SC wall of the containment internal structures indicated that these demands were generally lower than the cracking threshold for in-plane shear. Thus, the best estimate in-plane shear stiffness for Condition A is that of the uncracked composite section (i.e., $G_cA_c + G_sA_s$).

where MIC-03-03-00066

G_c = shear modulus of concrete

A_c = gross area of concrete

G_s = shear modulus of steel

A_s = 2·(face plate thickness)

Note that the cracking threshold for SC walls was assumed at a concrete stress of $2\sqrt{f'_c}$. Typically the cracking threshold for concrete is related to concrete stress of $4\sqrt{f'_c}$, but the limit for SC walls is reduced to account for shrinkage and other effects, as described in In-Plane Behavior of Concrete Filled Steel (CFS) Elements, Presentation, Enclosure 1 to DCP_NRC_00278 (Reference 3.8-67). This reduction is also corroborated by experimental data found in Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-plane Shear (Reference 3.8-61). In addition, the uncracked stiffness estimated for this condition takes into account the recommendation to increase calculated secant stiffness values by a factor of 1.25 to obtain effective in-plane shear stiffness values appropriate for use in an equivalent linear model as described in Relationship Between Effective Linear Stiffness and Secant Stiffness for Pinched In-Plane Shear Behavior or Shear Walls (Reference 3.8-62). Note that an effective stiffness value that results from calculation of 1.25 times the secant stiffness must not be taken to exceed the initial uncracked stiffness.

Experimental data indicates there is little to no uncracked out-of-plane flexural stiffness manifest in SC walls. This is due to effects of shrinkage cracking and partial composite action resulting from the discrete nature of the shear connectors (studs) between the face plates and the concrete core, as described by In-plane Shear Stiffness Recommendation (Reference 3.8-59). Instead, the stiffness ($E_c I_{ct}$) associated with the cracked-transformed section is exhibited very early during the application of out-of-plane moments to SC walls. DCD_03.07.02-35 MIC-03-03-00066

where

E_c = modulus of elasticity of concrete

I_{ct} = cracked-transformed moment of inertia of concrete

Category 1, Condition B: The through-thickness temperature gradient resulting from the accident thermal loading can cause significant cracking that reduces the in-plane shear stiffness of the SC walls. An empirical relationship providing a best-estimate of secant in-plane shear stiffness of cracked SC walls is as follows, and as described by In-plane Shear Stiffness Recommendation (Reference 3.8-59):

$$K_{cr} = 0.5(\bar{\rho}^{-0.42})G_s A_s$$

where

$$\bar{\rho} = \frac{A_s F_y}{\sqrt{f'_c} A_c}$$

G_s = shear modulus of steel

A_s = 2·(face plate thickness)

F_y = yield strength of steel plates

f'_c = specified compressive strength of concrete

A_c = unit area of concrete core

Category 2, Condition A: Stress evaluation indicates these thick walls remain uncracked for Condition A. Thus, uncracked stiffness values of the concrete section shall be used; i.e., $G_c A_c$ for in-plane shear and $E_c I_c$ for out-of-plane flexure.

where

G_c = shear modulus of concrete

A_c = gross area of concrete

E_c = modulus of elasticity of concrete

I_c = moment of inertia of concrete MIC-03-03-00066

Category 2, Condition B: Stiffness of these walls shall account for cracking due to accidental thermal loading. Stiffness values of $0.5G_cA_c$ and $0.5E_cI_c$ are assigned per the recommendations for cracked concrete walls as shown in Seismic Design Criteria for Structures, Systems, and Components (Reference 3.8-60). DCD_03.07.02-35 MIC-03-03-00066

Category 3, Condition A: The linear temperature gradient through the primary shield walls for normal operating conditions is not anticipated to cause significant cracking, and seismic demands on these walls are limited. Thus the primary shield wall stiffness shall be modeled as that of uncracked concrete (G_cA_c and E_cI_c). No credit is taken for the stiffness of the steel plates.

Category 3, Condition B: The accident thermal loading conditions is anticipated to cause only localized cracking in the thick primary shielding walls, which are largely enclosed by the mass concrete (Category 5) at the base of the containment internal structures. Thus, the stiffness for this condition is the same as that assigned for Condition A (uncracked).

Category 4, Condition A: In-plane shear stiffness of the reinforced concrete slabs shall be that of the gross concrete section (G_cA_c, in accordance with Seismic Design Criteria for Structures, Systems, and Components (Reference 3.8-60)). Out-of-plane flexural stiffness is equal to that of the gross concrete section (E_cI_c), as seismic-induced moments in the slabs are shown generally to be less than cracking moments (M_{cr}): MIC-03-03-00066

$$M_{cr} = f_r \cdot S$$

where

S = gross section modulus

f_r = modulus of rupture, taken equal to $7.5\sqrt{f'_c}$

Category 4, Condition B: In-plane shear stiffness of the reinforced concrete slabs for this condition shall also be that of the gross concrete section (G_cA_c). Out-of-plane flexural stiffness is taken as $0.5E_cI_c$, as described by Seismic Design Criteria for Structures, Systems, and Components (Reference 3.8-60).

Category 5 (both conditions): No significant cracking is anticipated in the massive reinforced concrete at the base of the structure as a result of either seismic or accident thermal loading. Thus, the stiffness is taken to be equal to that of uncracked concrete for both loading conditions.

Category 6 (both conditions): The stiffness of in-fill concrete provided for shielding purposes is not modeled for either loading condition; only the mass of these sections is included. For the pressurizer support platform, which is comprised of a grillage of steel shapes with in-fill concrete, only the stiffness of the steel members is modeled. DCD_03.07.02-35

Damping values are assigned to each structural category based on the estimated level of cracking. A damping value of 4% is assigned to composite SC walls with uncracked conditions (Condition A), and 5% when significant cracking is anticipated (Condition B). This is based on the results of the 1/10th scale test discussed in Technical Report MUAP-11005-P (Reference 3.8-63). For walls and slabs modeled as reinforced concrete structures, 4% damping is specified in RG 1.61 (Reference 3.8-64) for the limited levels of cracking associated with the OBE, while 7% damping is specified for cracked response exhibited during SSE loading. Finally, the massive concrete in the containment internal structures (Category 5) is not expected to exhibit significant cracking, such that 4% damping is considered appropriate in all cases. Recognizing that the amplified seismic response of the containment internal structure is dominated by the response of the SC walls, constant damping ratios of 4% for Condition A and 5% for Condition B are conservatively used for the seismic response analyses (See Table 3.8.3-4). MIC-03-03-00066

3.8.3.4.1 SC Module Stress Analyses

The design forces and moments for each member of the containment internal structure are calculated by the stress analysis using a three-dimensional FE model. The model is shown in Figure 3.8.3-10. The SC modules are simulated within the FE model using three-dimensional shell ~~plate bending~~ elements. Equivalent elastic stiffnesses of the SC modules are computed as shown ~~below~~above. The application of more detailed FE analysis is acceptable for qualifying modules subject to extreme conditions such as high accident temperatures. ~~The shell element properties are computed using the combined concrete section and the steel faceplates of the SC modules. This representation models the composite behavior of the steel and concrete.~~ DCD_03.07.02-35 MIC-03-03-00066 MIC-03-03-00066 DCD_03.07.02-35

- ~~Axial and Shear Stiffnesses of SC Modules:~~

$$\cancel{\Sigma EA = E_c A_c + E_s A_s,\ \Sigma GA = G_c A_c + G_s A_s}$$

$$\cancel{A_c = L(t - 2t_s),\ A_s = 2Lt_s,\ G_c = E_c/2(1+\nu_c),\ G_s = E_s/2(1+\nu_s)}$$

- ~~Bending Stiffness of SC Modules:~~

$$\cancel{\Sigma EI = E_c I_c + E_s I_s}$$

$$\cancel{I_c = L(t - 2t_s)^3/12,\ I_s = Lt^3/12 - I_c}$$

~~where:~~

~~E_c or E_s = modulus of elasticity for concrete or steel~~

~~ν_c or ν_s = Poisson's ratio for concrete or steel~~

~~L = length of SC module~~ DCD_03.07.02-35

~~t = thickness of SC module~~

~~t_s = thickness of plate on each face of SC module~~

3.8.3.4.2 Hydrodynamic Analyses

The vertical and lateral pressures of liquids inside containment are treated as dead loads. Structures supporting fluid loads during normal operation and accident conditions are designed for the hydrostatic as well as hydrodynamic loads as discussed in Subsection 3.7.3.9. The hydrodynamic analyses take into account the flexibility of walls in considering fluid-structure interaction. Sloshing height, however, is calculated using a conservative simplified assumption of a rigid tank shell in accordance with guidance provided in ASCE 4-98 (Reference 3.8-34), Subsection 3.5.4.3.

3.8.3.4.3 Thermal Analyses

The RWSP water and containment operating atmosphere's temperature is considered stable. The operating thermal load for each concrete member is calculated as the average and gradient based on this condition. The stress analysis is carried out by inputting these loads ~~into the corresponding part of R/B whole FE model. The normal thermal stresses for design are calculated in accordance with Appendix A of ACI 349 (Reference 3.8-8). The analysis reduction factor and modeling methods are shown in Table 3.8.3-3 and Table 3.8.3-4~~into a 3D FE model of the containment internal structures and the R/B basemat. For thermal effects on dynamic response, see Subsection 3.8.3.4. MIC-03-03-00066 DCD_03.07.02-35

The RWSP water and containment atmosphere are subject to temperature transients in the event of a LOCA as described in Subsection 3.8.3.3. The accident temperature transients result in a nonlinear temperature distribution within the members. Temperatures within the concrete members are calculated in a unidimensional heat flow analysis. The accident thermal load (average and equivalent linear gradients) is calculated from this analysis, at selected times during the transient.

The stress analysis for accident thermal loading is carried out by inputting the accident thermal load into a 3D FE model of the containment internal structures and the R/B basemat.~~the corresponding part of R/B whole FE model, as well as other parts. The stresses of containment are used for containment design. Though the stresses of containment internal structure are also obtained at the same time, since these self-limiting stresses are released in ultimate condition under such as extreme and abnormal load conditions, they are not taken into account in calculation of required reinforcement steel.~~ This is necessary to obtain realistic restraint of the structure walls at the basemat connection. The SC walls and reinforced concrete slabs in the containment internal structures are assigned the reduced stiffness values resulting from thermally induced cracking, as identified for Condition "B" in Technical Report MUAP-11018 (Reference 3.8-70) and in Subsection 3.8.3.4 above. The moments and forces induced in the modeled structure are then included in the ACI 349-06 design load combinations that involve accident thermal loading. MIC-03-03-00066 MIC-03-03-00066

Thermal transients for the DBAs are described in Section 6.3.

3.8.3.4.4 Design Procedures

~~The concrete members of the containment internal structure are designed by the strength method, as specified in the ACI "Code Requirements for Nuclear Safety-Related Structures", ACI 349 (Reference 3.8-8).~~ DCD_03.07.02-35

~~The primary and secondary shield walls, RWSP, refueling cavity, and other structural walls are designed using SC modules. SC modules are designed as reinforced concrete structures in accordance with the requirements of ACI 349 (Reference 3.8-8), as supplemented in the following paragraphs.~~The reinforced concrete members of the containment internal structure are designed by the strength method, as specified in the ACI 349-06 (Reference 3.8-8). MIC-03-03-00066

The primary and secondary shield walls, RWSP, refueling cavity, and other structural walls are designed using SC modules. SC modules are designed using the methodology of reinforced concrete structures in accordance with ACI 349-06 (Reference 3.8-8), as supplemented in Technical Reports MUAP-11019 (Reference 3.8-71) and MUAP-11020 (Reference 3.8-72). DCD_03.07.02-35 MIC-03-03-00066

Floor slabs of reinforced concrete are designed as reinforced concrete structures in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8). The floors of elevation 76 ft, 5 in. (Operating floor) and elevation 50 ft, 2 in. are supported by structural steel framing. MIC-03-03-00066

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 safe shutdown earthquake loads are determined from the results of seismic response analysis described in Section 3.7.

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 pressurization loads of 2 to 39 psi.

Determination of RCL support loads is described in Subsection 3.9.3. Design of the RCL supports are in accordance with ASME Code, Section III, Division 1, Subsection NF (Reference 3.8-2) as described in Subsections 3.9.3.

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.

3.8.3.4.5 SC Modules Design and Analysis

The SC modules are designed for dead, live, thermal, pressure, and safe shutdown earthquake loads. The RWSP walls are also designed for the hydrostatic head due to the water in the pit and the hydrodynamic pressure effects of the water due to the safe shutdown earthquake loads. The walls of the refueling cavity are also designed for the hydrostatic head due to the water in the refueling cavity.

~~Figure 3.8.3-7 shows the typical design details of the SC modules, typical configuration of the SC modules, typical anchorages of the SC modules to the reinforced base concrete,~~ DCD_03.07.02-35

~~and connections between adjacent walls. SC modules are designed as reinforced concrete structures in accordance with the requirements of ACI 349 (Reference 3.8-8), as supplemented in the following paragraphs. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The design of critical sections is described in Subsection 3.8.3.5.~~Figure 3.8.3-7 shows the typical design details of the SC modules, typical configuration of the SC modules, typical anchorages of the SC modules to the reinforced concrete basemat, and connections between adjacent walls. SC modules are designed using the methodology of reinforced concrete structures in accordance with ACI 349-06 (Reference 3.8-8), as supplemented in Technical Reports MUAP-11019 (Reference 3.8-71) and MUAP-11020 (Reference 3.8-72). The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The design of critical sections is described in Subsection 3.8.3.5. DCD_03.07.02-35 MIC-03-03-00066

Technical Report MUAP-11019 (Reference 3.8-71) is used to design the interior regions of the walls. Technical Report MUAP-11020 (Reference 3.8-72) is used to design the connections. The baseplate connecting the SC walls to the basemat is part of the liner and is therefore evaluated in accordance with the ASME Section III Division 2 code.

3.8.3.4.5.1 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 (Reference 3.8-8).~~Design for axial load (tension and compression), in-plane bending, and out-of-plane bending is in accordance with the methodology of ACI 349-06 (Reference 3.8-8) Chapters 10 and 14, as supplemented by Sections 3, 4, and 5 of Technical Report MUAP-11019 (Reference 3.8-71). DCD_03.07.02-35 MIC-03-03-00066

~~This~~The design approach recognizes behavior of the SC module is similar to that of reinforced concrete. The steel plate is similar to standard tensile reinforcement in each of ~~2~~two ~~designing~~ orthogonal directions, as concluded by the test results of ~~References 3.8-21 through 3.8-27~~references listed in Subsection 3.8.3.4. MIC-03-03-00066 MIC-03-03-00066

3.8.3.4.5.2 Design for In-Plane Shear

~~Design for in-plane shear is in accordance with the requirements of ACI 349, Chapters 11 and 14 (Reference 3.8-8). The steel faceplates are treated as reinforcement for the concrete, and satisfy the requirements of Section 11.10 of ACI 349 (Reference 3.8-8).~~Design for in-plane shear is in accordance with the methodology of ACI 349-06 (Reference 3.8-8) Chapters 11 and 21, as supplemented by Section 7 of Technical Report MUAP-11019 (Reference 3.8-71). The steel faceplates are treated as reinforcement for the concrete which satisfy the provisions of Section 11.10 of ACI 349-06 (Reference 3.8-8). DCD_03.07.02-35 MIC-03-03-00066

~~This~~The design approach is based on behavior of the SC module that is similar to reinforced concrete, which is supported by the test results of References listed in Subsection 3.8.3.4. The steel plate acts as shear reinforcement in each of ~~2~~two ~~designing~~ orthogonal directions, similar to that of standard concrete reinforcement. MIC-03-03-00066 MIC-03-03-00066

3.8.3.4.5.3 Design for Out-of-Plane Shear

~~Design for out-of-plane shear is in accordance with the requirements of ACI 349, Chapter 11 (Reference 3.8-8).~~Design for out-of-plane shear is in accordance with the methodology of ACI 349-06 (Reference 3.8-8) Chapter 11, as supplemented by Section 6 of Technical Report MUAP-11019 (Reference 3.8-71). DCD_03.07.02-35 MIC-03-03-00066

The design approach is based on the premise that the behavior against out-of-plane shear and the effect of shear reinforcement of the SC module are similar to those of reinforced concrete. This methodology is supported by the test results of References listed in Subsection 3.8.3.4.

3.8.3.4.5.4 Evaluation for Thermal Loads

~~The acceptance criterion for the load combination with normal thermal loads, which includes the thermal transients described in Subsection 3.8.3.4, is that the overall stress in general areas of the steel plate be less than yield. In local areas where the stress may exceed yield, the total stress intensity range is less than twice yield. This evaluation of thermal loads is based on the ASME Code philosophy for Level A service loads given in ASME Code, Section III (Reference 3.8-2), Subsection NE, Paragraphs NE-3213.13 and NE-3221.4.~~The forces and moments induced in the SC walls due to restraint of thermal growth are included in the design load combinations in accordance with ACI 349-06. As discussed in Section 9 of Technical Report MUAP-11019 (Reference 3.8-71), empirical data derived from experiments demonstrates that design basis accident thermal conditions cause no significant reduction in SC wall design strength. Thus, SC walls are evaluated and designed to resist combined design basis accident mechanical and therman loads consistent with provisions of ACI 349-06, as supplemented by Technical Report MUAP-11019 (Reference 3.8-71). MIC-03-03-00066

3.8.3.4.5.5 Design of Tie Bars

The tie bars provide a structural framework for the SC modules with faceplates, maintain the separation between the faceplates, support the SC modules during transportation and erection, ~~and~~ act as "form ties" between the faceplates when concrete is being placed. ~~After the concrete has cured, the tie bars~~Additionally, they ~~are not required to contribute to the strength or stiffness of the completed SC modules. However, they do~~ provide additional shear capacity between the steel plates and concrete as well as additional strength similar to that provided by stirrups in reinforced concrete. The tie bars are ~~designed as "form ties" according to the requirements of AISC-N690 (Reference 3.8-9) and designed as out-of-plane shear reinforcement according to the requirements of ACI 349 (Reference 3.8-8)~~designed as out-of-plane shear reinforcement according to the requirements of ACI 349-06 (Reference 3.8-8), as supplemented by Sections 3.6 and 3.7 of Technical Report MUAP-11019 (Reference 3.8-71). Tie bars are connected to steel faceplates in accordance with American Welding Society (AWS D1.1) requirements. Tie bar segments welded to each faceplate are spliced together using mechanical reinforcing bar couplers that meet requirements of ACI 349-06 Sections 12.14.3 and 21.2.6. MIC-03-03-00066 DCD_03.07.02-35 MIC-03-03-00066 MIC-03-03-00066

3.8.3.4.5.6 Design of Shear Studs

The SC 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 are designed according to Sections 2.1, 2.2, and 2.3 of Technical Report MUAP-11019 (Reference 3.8-71). The shear studs make the concrete and steel faceplates interact compositely. In addition, the shear studs permit anchorage for piping and other items attached to the walls. MIC-03-03-00066

3.8.3.4.6 Floor Slab

The floor slab of reinforced concrete is analyzed and designed according to ACI 349-06 (Reference 3.8-8) considering the same loads as for the SC modules. The floor design does not rely on composite action with supporting structural steel beams. MIC-03-03-00066

3.8.3.4.7 Structural Steel Design and Analysis

Structural steel framing within the interior of the PCCV is primarily for support of floor slabs, equipment, distribution systems, and access platforms. Design and analysis procedures, including assumptions on boundary conditions and expected behavior under loads, are in accordance with the allowable stress design (ASD) method in AISC-N690 (Reference 3.8-9). Analysis methods are generally simple calculations using seismic ~~accelerations~~loads obtained from Section 3.7 methodologies in load combinations. Frame connections are detailed for simply-supported beams unless otherwise analyzed and detailed. MIC-03-03-00066

3.8.3.4.8 RCL Supports

The RCL piping and support system is analyzed for the dynamic effects of a SSE. A coupled model of the containment internal~~s~~ structure and the RCS is dynamically evaluated using a time-history integration method of analysis. Appendix 3C provides additional information regarding the qualification of RCL supports. MIC-03-03-00066

3.8.3.5 Structural Acceptance Criteria

Structural acceptance criteria is reflected in Table 3.8.4-3 for concrete structures and Table 3.8.4-4 for steel structures, and are in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and AISC-N690 (Reference 3.8-9), except as provided in the table notes. MIC-03-03-00066

3.8.3.5.1 Design Report

A Design Report of the containment internal structure is provided separately from the DCD. The Design Report has sufficient detail to show that the applicable stress limitations are satisfied when components are subjected to the design loading conditions.

Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Section 3.7 and 3.8 provided the following acceptance criteria are met.

3.8.4.1.4 Heating, Ventilating and Air Conditioning Ducts and Duct Supports

Seismic category I HVAC ducts and duct supports are routed as necessary to supply safety-related functions of air distribution. Appendix 3A describes the qualification of HVAC ducts and duct supports.

3.8.4.1.5 Conduits and Conduit Supports

Seismic category I conduits and conduit supports are routed as necessary to supply safety-related Class-1E cable. The conduit consists of a metal wall of minimum thickness as specific, and is assembled using standard industry fittings and clips. Appendix 3F describes the qualification of conduits and conduit supports.

3.8.4.1.6 Cable Trays and Cable Tray Supports

Seismic category I cable trays and cable tray supports are routed as necessary to supply safety-related Class-1E cable. Cable trays are manufactured using thin-gauge steel channels, and supports are constructed using cold formed or rolled steel shapes. Appendix 3G describes the qualification of cable trays and cable tray supports.

3.8.4.2 Applicable Codes, Standards, and Specifications

The following industry standards are applicable for the design and construction of seismic category I structures and subsystems. Other codes, standards and specifications applicable to materials, testing and inspections are provided in Subsections 3.8.4.6 and 3.8.4.7.

- ~~ACI 349-01, Code Requirements for Nuclear Safety-Related Concrete Structures, American Concrete Institute, 2001~~Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary, American Concrete Institute, 2006 (Reference 3.8-8). MIC-03-03-00066

- ANSI/AISC N690-1994, Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004), American National Standards Institute/American Institute of Steel Construction, 1994 & 2004 (Reference 3.8-9).

- ANSI/ANS-57.7 Design Criteria for an Independent Spent Fuel Storage Installation (Water Pool Type), American National Standards Institute/American Nuclear Society, 1997 (Reference 3.8-33).

- ASCE 7-05, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2005 (Reference 3.8-35).

- ASCE 37-02, Design Loads on Structures During Construction, American Society of Civil Engineers, 2002 (Reference 3.8-36).

- ASME BPVC-III, Rules for Construction of Nuclear Facility Components - Section III Division 1 - Subsection NF - Supports, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda (Reference 3.8-2).

The structural evaluation considers a double-ended break and a longitudinal break (equal to the pipe cross-sectional area) for calculating the jet impingement load from the main steam and feedwater lines. This evaluation is applicable to the floor at elevation 65 ft, 0 in. of the Main Steam Isolation Valve (MSIV) subcompartment in the R/B break exclusion zone. The design pressure for LOCA and MSLB is considered for 100% power operation.

3.8.4.3.8.6 Impact of Ruptured Pipe (Y_m)

The load resulting from the impact of a ruptured high-energy pipe on a structure or a pipe restraint during the postulated event includes an appropriate dynamic load factor. The type of impact (i.e., plastic, elastic), together with the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the impact.

3.8.4.3.9 Load Combinations

Concrete structures are designed in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and the provisions of RG 1.142 (Reference 3.8-19) where applicable, with the load combinations and load factors provided in Table 3.8.4-3. MIC-03-03-00066

Steel structures are designed using the allowable strength design method in accordance with AISC N690 (Reference 3.8-9) for the load combinations and allowable strength factors provided in Table 3.8.4-4.

3.8.4.4 Design and Analysis Procedures

The following discussion describes the design and analysis procedures used for seismic category I structures in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8), with ~~supplement~~supplemental guidance by RG 1.142 (Reference 3.8-19) for concrete structures, and AISC N690 (Reference 3.8-9) for steel structures. This subsection also discusses items such as general assumptions on boundary conditions, expected behavior under loads, methods by which loads and forces are transmitted to supports and ultimately the structure foundation, and computer programs used. Table 3.8.4-5 summarizes the modeling and analytical methods of R/B and PS/Bs. MIC-03-03-00066

A Design Report prepared in accordance with guidance from Appendix C to SRP 3.8.4 provides design and construction information more specific than contained within this DCD. The Design Report information quantitatively represents the actual design computations and the final design results. In addition, the Design Report provides criteria for reconciliation between design and as-built conditions.

3.8.4.4.1 R/B

The R/B includes the MCR and the fuel storage area, and is a reinforced concrete structure consisting of vertical shear/bearing walls and horizontal slabs. The walls carry the vertical loads from the structure to the basemat. Lateral loads are transferred to the walls by the roof and floor slabs.

The fuel handling area is a reinforced concrete structure supported by structural steel framing. The new fuel is stored in racks in a dry, unlined pit. The spent fuel pit is lined with stainless steel and is normally flooded to an elevation 1 ft, 2 in. below the operating floor deck. Subsection 9.1.2 describes the design bases and layout of the fuel storage area.

The design and analysis procedures for the R/B, other than the PCCV and containment internal structure, including assumptions on boundary conditions and expected behavior under loads, are in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) for concrete structures, with AISC N690 (Reference 3.8-9) for steel structures, and with American Iron and Steel Institute (AISI) specification for cold formed steel structures (Reference 3.8-38). MIC-03-03-00066

The design considers normal loads (including construction, dead, live, and thermal), and the SSE. Seismic forces are obtained from the dynamic analysis described in Subsection 3.7.2. These loads are applied to the linear elastic FE model, which extends to the base of the R/B foundation, as equivalent static forces. Soil stiffnesses derived from the standard plant soil profiles are assigned to the subgrade for the design of the overall R/B, and the design of the R/B superstructure is also performed considering a fixed-base condition at the bottom of the foundation. Loads and load combinations are given in Subsection 3.8.4.3.

The design of the R/B's flexible shear walls and floor slabs, like that of the main steam piping room with many openings, takes into account the out-of-plane bending and shear loads, such as live load, dead load, and seismic load. Also, the walls and slabs of the spent fuel pit and the emergency feedwater pit are designed to resist the out-of-plane bending and shear loads, such as live load, dead load, seismic, hydrostatic, and hydrodynamic pressure.

The R/B is analyzed using a three-dimensional FE model with the ANSYS computer codes (Reference 3.8-14). The FE model is shown in Figure 3.8.4-2.

The basemat design is described in Subsection 3.8.5.

3.8.4.4.1.1 Structural Design of Critical Sections

This subsection summarizes the structural design of representative seismic category I structural elements in the R/B. These structural elements are listed below and the corresponding location numbers are shown on Figure 3.8.4-3.

SECTION 1 West exterior wall of R/B, elevation 3 ft, 7 in. to elevation 101 ft, 0 in. This exterior wall illustrates typical loads such as temperature gradients, seismic, and tornado missile.

SECTION 2 South interior wall of R/B, elevation 3 ft, 7 in. to elevation 101 ft, 0 in. This is one of the most highly stressed shear walls.

SECTION 3 The north exterior wall of spent fuel pit, elevation 30 ft, 1 in. to elevation 76 ft, 5 in. and the slab of spent fuel pit at elevation 30 ft, 1 in. The wall is subjected to temperature gradients, hydrostatic and hydrodynamic loads.

AREA 3 The slab of spent fuel pit at elevation 30 ft, 1 in. The slab is subjected to temperature gradients, hydrostatic and hydrodynamic loads.

thermal loads. As shown in Figure 3.8.4-6, the wall is divided in 3 segments for design purposes. Table 3.8.4-8 presents the typical details of the reinforcement for each SECTION 3 wall zone. Figure 3.8.4-6 shows the typical reinforcement for the west exterior wall at SECTION 3.

South Exterior Wall

The south exterior reinforced concrete wall extends from the top of the basemat area at elevation 3 ft, 7 in. to the roof at elevation 115 ft, 6 in. The wall segments are typically 40 in. to 44 in. thick. The wall is designed for the applicable loads including dead load, live load, hydrostatic and hydrodynamic loads (for Emergency Feedwater Pit wall), seismic loads, thermal loads, and tornado missile load. As shown in Figure 3.8.4-7, the wall is divided in 5 segments for design purposes. Table 3.8.4-9 presents the typical details of the reinforcement for each SECTION 4 wall zone. Figure 3.8.4-7 shows the typical reinforcement for the west exterior wall at Section 4.

3.8.4.4.1.3 Floor and Roof

Design Approach

The concrete slab and the steel reinforcement of the composite section are evaluated for normal and extreme environmental conditions. The slab concrete and the reinforcement are designed to meet the requirements of the ACI 349-06 Code (Reference 3.8-8). The slab design considers the in-plane and out-of-plane seismic forces. The global in-plane and out-of-plane forces are obtained from the equivalent static analysis of the three-dimensional FE model of the R/B. MIC-03-03-00066

Spent Fuel Pit Slab at Elevation 30 ft, 1 in., AREA 3

This concrete slab is designed for the applicable loads including dead load, live load, hydrostatic and hydrodynamic loads, seismic loads, spent fuel rack reaction loads, and thermal loads. The concrete slab is 126 in. thick. Table 3.8.4-10 presents the typical details of the reinforcement for AREA 3. Figure 3.8.4-8 shows the typical reinforcement at AREA 3.

Emergency Feedwater Pit Slab at Elevation 76 ft, 5 in., AREA 4

This concrete slab is designed for the applicable loads including dead load, live load, hydrostatic and hydrodynamic loads, seismic loads, and thermal loads. The concrete slab is 52 in. thick. Table 3.8.4-11 presents the typical details of the reinforcement for AREA 4. Figure 3.8.4-9 shows the typical reinforcement at AREA 4.

3.8.4.4.2 East and West PS/Bs

The east and west PS/Bs provide two emergency power sources, and are reinforced concrete structures consisting of vertical shear/bearing walls and horizontal slabs. The walls carry the vertical loads from the structure to the basemat. Lateral loads are transferred to the walls by the roof and floor slabs.

The design and analysis procedures for the PS/Bs, as described above for the R/B above including assumptions on boundary conditions and expected behavior under loads, are in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-~~7~~8) for concrete structures, with AISC N690 (Reference 3.8-~~8~~9) for steel structures, and AISI specification for cold formed steel structures Reference 3.8-38). MIC-03-03-00066

The design considers normal loads including construction, dead, live, thermal, and the SSE. Seismic forces are obtained from the dynamic analysis described in Subsection 3.7.2. Loads and load combinations are provided in Subsection 3.8.4.3.

The PS/Bs are analyzed using a three-dimensional FE model with the ANSYS computer codes (Reference 3.8-14). The FE model is shown in Figure 3.8.4-10. The basemat design is described in Subsection 3.8.5.

3.8.4.4.2.1 Structural Design of Critical Sections

This subsection summarizes the structural design of representative seismic category I structural elements in the PS/Bs. These structural elements listed below are for the west PS/B, which is the worst case configuration and contains the most critical sections. Locations within the west PS/B are shown with corresponding sections and area on Figure 3.8.4-11 and Figure 3.8.4-12.

SECTION 1 South exterior wall of west PS/B, elevation -26 ft, 4 in. to elevation 39 ft, 6 in. This exterior wall illustrates typical loads such as temperature gradients, seismic, and tornado missile.

SECTION 2 Typical interior wall of PS/Bs, elevation -26 ft, 4 in. to elevation 3 ft, 7 in. This is one of the most highly stressed shear walls.

AREA 1 The slab of PS/B at elevation 3 ft, 7 in. The slab is subjected to live loads and temperature gradients.

3.8.4.4.2.2 Shear Walls

<u>Structural Description</u>

All exterior walls are shear walls, however internal shear walls exist only in the north-south axis. The stress levels in shear walls depend on thickness, configuration, aspect ratio, amount of reinforcement and the seismic acceleration level. The walls are monolithically cast with the concrete floor slabs. The in-plane behavior of these shear walls, including the large openings, is adequately represented in the analytical models for the global seismic response. The shear walls are used as the primary system for resisting the lateral loads, such as earthquakes.

<u>Design Approach</u>

The PS/B shear walls are designed to withstand the loads specified in Subsection 3.8.4.3. Dead, live, thermal, and other normal operating condition loads are considered in the shear wall design.

South Exterior Wall

The south exterior reinforced concrete wall extends from the top of the basemat area at elevation -26 ft, 4 in. to the roof at elevation 39 ft, 6 in. The walls are typically 21 in. above elevation 3 ft, 7 in. and 32 in. below elevation 3 ft, 7 in. The wall is designed for the applicable loads including dead load, live load, seismic loads, thermal loads, and tornado missile load. As shown in Figure 3.8.4-13, the wall is divided in 4 segments each design purposes. Table 3.8.4-12 presents the typical details of the reinforcement for the SECTION 1 wall zone. Figure 3.8.4-13 shows the typical reinforcement of the south exterior wall at SECTION 1.

Typical Interior Wall

The typical interior reinforced concrete wall extends from the top of the basemat area at elevation -26 ft, 4 in. to the slab at elevation 3 ft, 7 in. The walls are 20 in. thick. The wall is designed for the applicable loads including dead load, live load, seismic loads, and thermal loads. Table 3.8.4-13 presents the typical details of the reinforcement for SECTION 2 wall zone 1, which is applicable for all interior walls. Figure 3.8.4-14 shows the typical reinforcement for the interior wall at SECTION 2.

3.8.4.4.2.3 Floor

Design Approach

The concrete slab and the steel reinforcement of the composite section are evaluated for normal and extreme environmental conditions. The slab concrete and the reinforcement are designed to meet the requirements of American Concrete Institute standard ACI 349-06 (Reference 3.8-8). The slab design considers the in-plane and out-of-plane seismic forces. The global in-plane and out-of-plane forces are obtained from the equivalent static analysis of the worst case three-dimensional FE model of the PS/B. MIC-03-03-00066

Slab at Elevation 3 ft, 7 in., AREA 1

The concrete slab is designed for the applicable loads including dead load, live load, seismic loads, and thermal loads. The concrete slab is 32 in. thick. Table 3.8.4-14 presents the typical details of the reinforcement for AREA 1. Figure 3.8.4-15 shows the typical reinforcement at AREA 1, which is applicable for the entire floor slab area in the PS/Bs

3.8.4.4.3 Other Seismic Category I Structures

The design and analysis procedures for other seismic category I concrete structures are in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8). The design and analysis procedures for seismic category I steel structures are in accordance with AISC N690 (Reference 3.8-9). MIC-03-03-00066

Seismic category I structures are modeled globally using applicable loads, including equivalent dead and live loads, in load combinations that include design-basis earthquake accelerations as described in Section 3.7. Computer modeling utilizes three-dimensional FE models to globally analyze the beams, columns, slabs, and shear walls.

Individual structural members are further analyzed for localized loading as described in specific load cases.

Concrete components such as walls, slabs, and foundations are evaluated for the effects of frame interaction when the flexural moment from seismic loads is a large percentage of the flexural capacity. When at least two-thirds of the flexural capacity of a component is from seismic loads alone, the component is designed as a frame to assure design capacity even under a seismic margin earthquake equal to 150% of the SSE, in accordance with RG 1.142 (Reference 3.8-19), Regulatory Position 3.

Concrete members that are subject to torsion and combined shear and torsion are evaluated to the standards of Section 11.6 of ACI 349-06 (Reference 3.8-8). MIC-03-03-00066

Design and analysis of the spent fuel pit, the spent fuel racks, and the fuel handling system is in accordance with Appendix D of NUREG-0800, SRP 3.8.4 (Reference 3.8-40). Additional general information is provided by ANSI/ANS-57.7 (Reference 3.8-33). Subsection 9.1.2 describes the design bases and layout of the spent fuel pit, the spent fuel racks, and the fuel handling system.

Exterior concrete walls below grade and basemat of seismic category I structures are designed using load combinations accounting for sub-grade loads including static and dynamic lateral earth pressure, soil surcharges, and effects of maximum water table. Dynamic lateral earth pressure is calculated ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~as described in Section 3.7 of Technical Report MUAP-10006 (Reference 3.7-48). The calculation approach follows guidance given in ASCE 4-98 (Reference 3.8-34) for computing dynamic lateral earth pressure, and also accounts for increases in horizontal pressure due to the vertical component of earthquake excitation. The static and seismic lateral earth pressures due to the vertical and horizontal components of the earthquake are combined by conservatively assuming that the peak vertical and horizontal response accelerations in the embedment soil occur simultaneously. The use of saturated unit weight for the soil provides the most conservative case for including the effects of groundwater in the calculations of the dynamic earth pressures because it considers that the response of the two phases of the system, the groundwater and soil, to be completely in-phase and does not consider the dissipation of energy due to the viscous flow of the groundwater. The total dynamic lateral pressure computed in this manner envelops the in-phase sum of the Wood's soil pressure (per ASCE 4-98) and the Westergaard formula for computing hydrodynamic groundwater pressure under seismic loads on a vertical wall bordering a free body of water (e.g. reservoir). DCD_03.07.02-98 DCD_03.07.02-98

Structural steel framing in seismic category I structures is primarily for the support of distribution systems, access platforms, and other plant appurtenances. Steel members are sized and detailed based on maximum stresses and reactions determined through conservative manual calculations and computer models based on pinned-end connections, including slotted hole clip angle connections, to relieve thermal expansion forces where appropriate, unless detailed to develop end moments in accordance with AISC N690 (Reference 3.8-9). The design of the support anchorage to the concrete structure is in accordance with ACI 349-06, Appendix ~~B~~D (Reference 3.8-8), RG 1.142 (Reference 3.8-19), and RG 1.199 (Reference 3.8-41). MIC-03-03-00066

The design and analysis procedures for seismic category I distribution systems, such as HVAC ducts, conduits, and cable trays including their respective seismic category I supports, are in accordance with AISC N690 (Reference 3.8-~~8~~9) and AISI Specification for Design of Cold-Formed Steel Members (Reference 3.8-~~34~~38). The following appendices provide additional discussion of the design and analysis of these subsystems. MIC-03-03-00066

- Appendix 3A Heating, Ventilation, and Air Conditioning Ducts and Duct Supports
- Appendix 3F Design of Conduits and Conduit Supports
- Appendix 3G Seismic Qualification of Cable Trays and Supports

The COL Applicant is to provide design and analysis procedures for the ESWPT, UHSRS, and PSFSVs.

3.8.4.4.4 Seismic Category II Structures

Seismic category II structures need not remain functional during and after an SSE. However, such structures must not fall or displace to the point they could damage seismic category I SSCs.

Seismic category II structures and subsystems are analyzed and designed using the same methods and stress limits specified for seismic category I structures and subsystems, and the same load combinations and stress coefficients given in Table 3.8.4-4.

3.8.4.5 Structural Acceptance Criteria

Structural acceptance criteria are listed in Table 3.8.4-3 for concrete structures and in Table 3.8.4-4 for steel structures, and are in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and AISC N690 (Reference 3.8-9), except as provided in the table notes. MIC-03-03-00066

The deflection of the structural members is limited to the maximum values as specified in ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and AISC N690 (Reference 3.8-9), as applicable. MIC-03-03-00066

Subsection 3.8.5.5 identifies acceptance criteria applicable to additional basemat load combinations.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

The following information pertains to the materials, quality control programs, and any special construction techniques utilized in the construction of the seismic category I structures for the US-APWR.

3.8.4.6.1 Materials

The major materials of construction in seismic category I structures are concrete, grout, steel reinforcement bars, splices of steel reinforcing bars, structural steel shapes, and anchors.

3.8.4.6.1.1 Concrete

Concrete utilized in standard plant seismic category I structures, other than PCCV and upper part of the tendon gallery in the basemat, has a compressive strength of f'_c = 4,000 psi. Concrete utilized in the PCCV and upper part of the tendon gallery in the basemat has a compressive strength of f'_c = 7,000 psi and is subject to the PCCV material requirements in Subsection 3.8.1.6, including the requirements of ASME III, Division 2 (Reference 3.8-2), as shown in Figure 3.8.5-4. The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures. A test age of 28 days is used for normal concrete. Batching and placement of concrete is performed in accordance with ACI 349-06 (Reference 3.8-8), ACI 304R (Reference 3.8-~~38~~39), and ASTM C 94 (Reference 3.8-42). During construction, volume changes in mass concrete are controlled where necessary by applying measures and provisions outlined in ACI 207.2R (Reference 3.8-52) and ACI 207.4R (Reference 3.8-53). MIC-03-03-00066

Portland cement is used in the concrete conforms to ASTM C 150, Type II (Reference 3.8-43) standards. The confirmation of the chemical composition of the cement properties is validated by certified copies of test reports showing the chemical composition of each Portland cement shipment.

Aggregates used in the concrete conform to ASTM C 33 (Reference 3.8-44). Aggregate and source acceptance is based on documented test results for each source and random sampling of shipments based on MIL-STD-1916 (Reference 3.8-45).

Water and ice used in the concrete conform to the requirements of ~~ACI 349~~ACI 349-06 (Reference 3.8-8). MIC-03-03-00066

Admixtures include an air entraining admixture, pozzolans, and a water reducing admixture. The admixtures, except the pozzolans, are stored in a liquid state.

Admixtures and concrete mix conform to the following requirements:

Pozzolans	ASTM C 618
Sampling and Testing of Pozzolans	ASTM C 311
Air Entraining Admixtures	ASTM C 260
Water Reducing Admixtures	ASTM C 494
Concrete Mix	ACI 211.1 and ASTM C 94 (Reference 3.8-45)
Concrete Mix Testing	ASTM C 172, ASTM C 192, and ASTM C 39
Minimum Number of Strength Tests~~[1]~~	ACI 349-06 (Reference 3.8-~~7~~8) ~~and ASME NQA-2 (Reference 3.8-37)~~

MIC-03-03-00066

~~Note 1: In lieu of frequency of compressive strength testing specified by Section 5.6.1.1 of ACI 349-97 (Reference 3.8-8) or that specified by ASME NQA-2 (Reference 3.8-37), the following is acceptable per RG 1.142, Regulatory Position 5 (Reference 3.8-19).~~ MIC-03-03-00066

Samples for strength tests of concrete should be taken at least once per day for each class of concrete placed or at least once for each 100 cubic yards of concrete placed. When the standard deviation for 30 consecutive tests of a given class is less than 600 psi, the amount of concrete placed between tests may be increased by 50 cubic yards for each 100 psi the standard deviation is below 600 psi, except that the minimum testing rate should not be less than one test for each shift when the concrete is placed on more than one shift per day or not less than one test for each 200 cubic yards of concrete placed. The test frequency should revert to once for each 100 cubic yards placed if the data for any 30 consecutive tests indicate a higher standard deviation than the value controlling the decreased test frequency.

3.8.4.6.1.2 Grout

Grout is used to transfer load from machinery, equipment, and column bases to their foundations, and to anchor the reinforcing bars, dowels, and anchor rods into hardened concrete. Grout generally consists of Portland cement, sand, water, and admixtures. Epoxy grout is only used in areas where radiation levels and temperature levels are compatible with epoxy use.

Portland cement used in the concrete conforms to ASTM C 150, Type II (Reference 3.8-43). Sand must be clean with gradation and fineness in accordance with ASTM C33 (Reference 3.8-44). Water and ice used in the grout conforms to the requirements of ACI 349-06 (Reference 3.8-8). Water-reducing and/or retarding admixtures conform to ASTM C494. MIC-03-03-00066

3.8.4.6.1.3 Steel for Concrete Reinforcement

Steel bars for concrete reinforcement are deformed bars conforming to ASTM A 6l5, Grade 60, or ASTM A 706, Grade 60 (minimum yield strength of 60,000 psi). For each heat (batch) of reinforcing steel bars, certified mill test reports are provided. Additionally, for each 50 tons/bar size/heat, a minimum of one tensile test is performed. Where mechanical anchorage can not be achieved through the use of deformed bars, headed steel bars conforming to ASTM A 970 are used.

Coated reinforcing steel is not used. Placement of concrete reinforcement is in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8), Sections 7.5 and 7.6. MIC-03-03-00066

3.8.4.6.1.4 Splices

Reinforcement splices comply with ~~ACI 349~~ACI 349-06, Chapter 12 (Reference 3.8-8). MIC-03-03-00066 All bars are sheared or cut to the correct length shown on the bar bending schedules from continuous rolled bar stock. In general, all splices are made with a wire-tied lap of length in accordance with ACI 408R. Mechanical splices used are in conformance with ACI 493.3R. Mechanical splices develop 125% of the specified yield strength of the spliced bar. Welding of reinforcing steel, other than in the PCCV, is performed in accordance with American Welding Society (AWS) D1.4 (Reference 3.8-46).

Edition through the 2003 Addenda (hereafter referred to as ASME Code). (Reference 3.8-2).

Note: Articles CC-1000 through CC-6000 of Section III, Division 2 are acceptable for the scope, material, design, construction, examination, and testing of concrete containments of nuclear power plants subject to the regulatory positions provided by RG 1.136 (Reference 3.8-3).

The following industry standards are applicable for the design and construction of seismic category I basemats not required as a pressure retention boundary. Other codes, standards and specifications applicable to materials, testing and inspections are provided in Subsections 3.8.4.6 and 3.8.4.7.

- ACI 349-0~~1~~6, Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary, ~~American Concrete Institute,~~ 200~~1~~6 (Reference 3.8-8) MIC-03-03-00066
- RG 1.142, Rev. 2, Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments), U.S. Nuclear Regulatory Commission, Washington, DC, November 2001. (Reference 3.8-19)

3.8.5.3 Loads and Load Combinations

Loads and load combinations are discussed in detail in Subsections 3.8.1.3 and 3.8.4.3. The containment design pressure P_d of 68 psi is included as an accident pressure in these load cases. Other load combinations applicable to the design of the basemat include acceptance criteria for overturning, sliding, and flotation as detailed in Table 3.8.5-1. The non-ASME portion of the basemat is designed in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and the provisions of RG 1.142 (Reference 3.8-19), where applicable. The reinforced concrete basemat for the PCCV and enveloped containment internal structure are designed in accordance with ASME Code Section III, Division 2, Subsection CC (Reference 3.8-2). Figure 3.8.5-4 delineates basemat regions applicable to each Code. MIC-03-03-00066

3.8.5.4 Design and Analysis Procedures

Based on the premise that seismic category I buildings basemats are not supported on bedrock, a computer analysis of the SSI is performed for static and dynamic loads. Subsection 3.7.2 provides further information.

The seismic category I structures are concrete, shear-wall structures consisting of vertical shear/bearing walls and horizontal floor slabs designed to SSE accelerations as discussed in Section 3.7. The walls carry the vertical loads from the structure to the basemat. Lateral loads are transferred to the walls by the roof and floor slabs. The walls then transmit the loads to the basemat. The walls also provide stiffness to the basemat and distribute the loads between them.

The reinforced concrete basemat for the PCCV and enveloped containment internal structure are designed in accordance with ASME Code Section III, Division 2, Subsection

CC (Reference 3.8-2). Other seismic category I basemats of reinforced concrete are designed in accordance with ~~ACI 349~~ACI 349-06 (Reference 3.8-8) and the provisions of RG 1.142 (Reference 3.8-19) where applicable. Table 3.8.5-2 identifies the material properties of concrete and Figure 3.8.5-4 delineates the governing codes based on region of the R/B, PCCV and containment internal structure basemat. MIC-03-03-00066

3.8.5.4.1 Properties of Subgrade

For the purposes of the US-APWR standard design, the SSI effects are captured using a representative suite of soil profiles and depths to baserock material with frequency dependent properties. Section 3.7.2.4 provides further discussion relating to SSI and the selection of subgrade types.

The soil profiles, due to the frequency-dependency of multiple soil layers, are variable opposed to fixed values. Documenting of typical (generic) subgrade conditions is not applicable.

A set of eight (8) generic layered profiles are selected for SSI analyses, which strain-compatible properties, shear wave and compression wave velocities (Vs and Vp) and corresponding hysteretic damping values provide a wide variation of properties that addresses soil properties. The development of frequency dependent properties used in the seismic analyses is described further in Section 3.7.2.4.

The minimum allowable subgrade bearing capacity of 15,000 psf represents the maximum bearing pressures resulting from static load cases for the R/B-PCCV-containment internal structure common foundation, while the minimum allowable dynamic soil bearing capacity of 60,000 psf represents the maximum bearing pressure resulting from Normal plus SSE loads. These bearing pressures envelope the foundation bearing pressures for all other standard plant building structures.

The foundation depth-to-equivalent-radius ratio for the R/B-PCCV basemat is less than 0.3, which indicates a shallow embedment foundation for purposes of SSI as defined in ASCE 4-98, Subsection 3.3.4.2 (Reference 3.8-34). SSI analyses performed for the generic soil cases consider the R/B complex common basemat and PS/B basemats as resting on the surface of the subgrade, which is located approximately 40 feet below the plant nominal ground surface elevation. The R/B complex embedment effects study is conducted by performing site-independent SSI analyses on the R/B complex lumped mass stick model using cracked-concrete stiffness reduction factors in accordance with Technical Report MUAP-11007 (Reference 3.8-58). Embedment effects for site-specific applications are addressed on a site-specific basis as stated in Subsection 3.7.2.4.~~Embedment effects on the R/B and PCCV SSI analysis are neglected in the US-APWR standard plant design in obtaining the soil impedance functions. Therefore, the R/B-PCCV seismic models are not coupled with any subgrade or backfill material at the sides of the basemat or along the faces of below-grade exterior walls, and no credit is taken in the seismic analysis for restraint due to the presence of these materials. Subsequently, there are no explicit requirements for shear wave velocity or other material characteristics requirements for the subgrade and/or backfill materials present on the sides of the basemat and R/B below-grade exterior walls. Subsection 3.7.2.4 provides additional discussion on the SSI analysis.~~ DCD_03.07.02-35

3.8.5.4.2 Analyses for Basemat Loads during Operation MIC-03-03-00066

The major seismic category I structures basemat analyses use 3-dimensional ANSYS FE models of the major seismic category I structures, which are described in Subsection 3.7.2.3. ~~Soil springs are assigned in the model to determine the interaction of the basemat with the overlying structures and with the subgrade. The model is capable of determining the possibility of uplift of the basemat from the subgrade during postulated SSE events. The vertical spring at each node in the analytical model act in compression only. The horizontal springs are active when the vertical spring is in compression and inactive when the vertical spring lifts off.~~ MIC-03-03-00066

The three-dimensional FE model of the basemat includes the structures above the basemat and their effect on the distribution of loads on the basemat. The combined global FE model of the R/B, PCCV, and containment internal structure, including basemat, is presented on Figures 3.8.5-5 through 3.8.5-10.

The analysis considers normal and extreme environmental loads and containment pressure loads. The normal loads include dead loads and live loads. Extreme environmental loads include the SSE.

Dead loads are applied as inertia loads. Live loads and the SSE loads are applied as concentrated loads on the nodes. The SSE loads are applied as equivalent static loads. For the structural design of the basemat concrete and reinforcing, the three directions of the earthquake loading are combined using the Newmark 100-40-40 method. Note that this method of combination is different from that used for the structural design of the PCCV, containment internal structure, and R/B, which is the SRSS method.

~~Linear analyses are performed for all specified load combinations assuming that the soil springs can take tension.~~ The results of the linear ~~cases~~analyses are then used to select critical load cases for non-linear analyses. The results from these analyses include the forces, shears, and moments in the basemat; the bearing pressures under the basemat; and the area of the basemat that is uplifted. Minimum area of steel reinforcement is calculated from the section forces for the most critical load combinations. MIC-03-03-00066

The required reinforcement steel for the portion of the basemat under the R/B (other than PCCV) is determined by considering the reinforcement envelope for the full non-linear iteration of the most critical load combinations.

3.8.5.4.2.1 Global Three-Dimensional FE Modeling of Basemat

The stress conditions of the basemat for the R/B complex are generated by numerous types of loads from the superstructure. The modeling of the basemat therefore involves evaluating the interaction between the basemat and the superstructures to determine the stress conditions at the interface. The global FE model is analyzed utilizing the FE computer program ANSYS (Reference 3.8-14).

Regarding the R/B, the element divisions in a horizontal direction inside the secondary shield walls of the containment internal structure are made in a rectangular grid pattern and those divisions outside the secondary shield wall are made in a polar pattern.

Peripheral areas of the basemat, outside the thickened mat that supports the PCCV and containment internal structure are divided into a rectangular grid.

The upper portion of tendon gallery is considered with concentrated stresses created by the connection with the PCCV. This region is divided into multiple layers of elements in the radial direction to accommodate the differing concrete strengths in this area as shown schematically in Figure 3.8.5-4.

The basemat below the PCCV and the lower portion of containment internal structure are simulated with solid elements (ANSYS SOLID45 elements). The elements below the PCCV are divided into ten layers, and elements in peripheral areas are divided into four layers.

The FE modeling of the PS/Bs is addressed in Subsection 3.8.4.4.

3.8.5.4.3 Boundary Conditions of Basemat

The basemat subgrade is included in the detailed static FE models used for structural design by meshing a sufficiently large volume of soil/rock below and around the basemat. The stiffness of the backfill around the below-grade walls is not considered in the model. ~~To increase computational efficiency, the subgrade part of the FE model is condensed into a super-element.~~ The properties of the subgrade layers used in the FE model of the subgrade are established based on several profiles selected from the generic layered soil profiles described in Technical Report MUAP-10001 (Reference 3.7-47) to cover the entire range of soil/rock conditions at representative nuclear power plant sites within the central and eastern US.

DCD_03.07.02-35

MIC-03-03-00066

3.8.5.4.4 Analyses of Settlement

The potential for foundation subsidence, or differential displacement, is designed for a maximum 2 in. based on enveloping properties of subsurface materials. This is a conservative allowance that may not be applicable at all plant sites. Subsidence and differential displacement may therefore be reduced to less than 2 in. if justified by the COL Applicant based on site-specific soil properties.

Soil conditions for which settlement during construction is considered are identified in Chapter 2, Subsection 2.5.4. To evaluate the potential for settlement, soil conditions applicable to the US-APWR are considered to determine the enveloping design cases. Based on this assessment, soft soil sites with alternating sand and clay layers maximizes early stage settlement and impact of dewatering, while soft soil sites with clay maximize settlement in the long term. In any situation, conditions outside the boundaries of acceptable soils are removed and replaced using compatible structural fill.

The basemat FE model is analyzed for various phases of construction, including the determination of displacement. The design is completed in accordance with ASME Section III, Division 2 (Reference 3.8-2) and ACI 349-06 (Reference 3.8-8) using applicable construction load combinations and factors provided in Table 3.8.4-3. Based on these analyses, the basemat is detailed and constructed to minimize any potential differential settlement during construction. MIC-03-03-00066

Technical Report MUAP-11007 (Reference 3.8-58) describes the methodology used to perform R/B complex, PS/B, A/B structure sliding and overturning stability studies for various soil conditions, including the effects of dead load and buoyancy. Dynamic FE and lumped mass stick models of the structures are used in these studies (see Appendix 3H for lumped mass stick model methodology.) SSE ground motion is applied in three orhogonal directions simultaneously. Soil bearing pressure demands at the toe of the basemat are also calculated. DCD_03.07.02-35 MIC-03-03-00066

R/B complex stability is analyzed by post-processing SSI time-history responses obtained from ACS SASSI analyses and by calculating the factor of safety against possible modes of failure. Ground water level is one foot below finished plant grade and the minimum design safety factors against sliding and overturning are 1.1. Soil bearing pressures are calculated for the combined effects of dead weight, buoyancy, and SSE earthquake loads at each corner of the respective basemat foundations. The basemats are considerd rigid. Soil pressure variation is thus assumed to remain linear. Uplift possibility is investigate at each time step by determining reduction in contact area due to time varying axial force and biaxial moments.

Result of Sliding and Overturning Stability Studies - Results of sliding and overturning analyses determine a factor of safety against sliding and overturning that is >1.1 to ensure stability of the seismic Category I structures.

3.8.5.5.1 Overturning Acceptance Criteria

The factor of safety against overturning is identified as the ratio of the moment resisting overturning (M_r) divided by the overturning moment (M_o). For SSE load cases, M_o is the maximum moment from the time history analyses of the applicable structure's lumped mass stick model in accordance with Section 3.7. Therefore,

$$FS_o = [\, M_r / M_o \,]\text{, not less than } FS_{ot} \text{ as determined from Table 3.8.5-1.}$$

where

FS_o = Structure factor of safety against overturning by the maximum design basis wind, tornado, or earthquake load.

M_r = Resisting moment determined as the dead load of the structure, minus the buoyant force created by the design ground water table, multiplied by the distance from the structure edge to the structure center of gravity provided there is no overstress at the structure's edge.

M_o = Overturning moment caused by the maximum design basis wind, tornado, or earthquake load.

3.8.5.5.2 Sliding Acceptance Criteria

The factor of safety against sliding caused by wind or tornado is identified by the ratio:

COL 3.8(22) *The COL Applicant is to establish a site-specific program for monitoring and maintenance of seismic category I structures in accordance with the requirements of NUMARC 93-01 (Reference 3.8-28) and 10 CFR 50.65 (Reference 3.8-29) as detailed in RG 1.160 (Reference 3.8-30). For seismic category I structures, monitoring is to include base settlements and differential displacements.*

COL 3.8(23) *The COL Applicant is to determine if the site-specific zone of maximum frost penetration extends below the depth of the basemats for the standard plant, and to pour fill concrete under any basemat above the frost line so that the bottom of fill concrete is below the maximum frost penetration level.*

COL 3.8(24) *Other non-standard seismic category I buildings and structures of the US-APWR are designed by the COL Applicant based on site-specific subgrade conditions.*

COL 3.8(25) *The site-specific COL are to assure the design criteria listed in Chapter 2, Table 2.0-1, is met or exceeded.*

COL 3.8(26) *Subsidence and differential displacement may therefore be reduced to less than 2 in. if justified by the COL Applicant based on site specific soil properties.*

COL 3.8(27) *The COL Applicant is to specify normal operating thermal loads for site-specific structures, as applicable.*

COL 3.8(28) *The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures.*

COL 3.8(29) *The COL Applicant is to provide design and analysis procedures for the ESWPT, UHSRS, and PSFSVs.*

COL 3.8(30) *When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349-06 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required.* MIC-03-03-00066

3.8.7 References

3.8-1 Combined License Applications for Nuclear Power Plants, RG 1.206, Rev. 0, U.S. Nuclear Regulatory Commission, Washington, DC, June 2007.

3.8-2 Rules for Construction of Nuclear Facility Components, Division 2, Concrete Containments. Section III, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda (hereafter referred to as ASME Code).

3.8-3 Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments. RG 1.136, Rev. 3, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.

3.8-4 Rules for Inservice Inspection of Nuclear Power Plant Components. Section XI, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda.

3.8-5 Inservice Inspection of Ungrouted Tendons in Pre-stressed Concrete Containments. RG 1.35, Rev. 3, U.S. Nuclear Regulatory Commission, Washington, DC, July 1990.

3.8-6 Determining Pre-stressing Forces for Inspection of Pre-stressed Concrete Containments. RG 1.35.1 U.S. Nuclear Regulatory Commission, Washington, DC, July 1990.

3.8-7 Concrete Containment, Design of Structures, Components, Equipment, and Systems, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants. NUREG-0800 SRP 3.8.1, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.

3.8-8 ~~Code Requirements for Nuclear Safety-Related Concrete Structures. ACI 349-01, American Concrete Institute, 2001.~~Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary, American Concrete Institute, 2006. MIC-03-03-00066

3.8-9 Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, ANSI/AISC N690-1994 including Supplement 2 (2004), American National Standards Institute/American Institute of Steel Construction, 1994 & 2004.

3.8-10 Combustible Gas Control for Nuclear Power Reactors, Domestic Licensing of Production and Utilization Facilities, Energy. Title 10 Code of Federal Regulations Part 50.44, U.S. Nuclear Regulatory Commission, Washington, DC, January 1, 2007.

3.8-11 Control of Combustible Gas Concentrations in Containment, Regulatory Guide 1.7, Rev. 3, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.

3.8-12 Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs. SECY-93-087, U.S. Nuclear Regulatory Commission, Washington, DC, April 2, 1993.

3.8-13 Deleted.

3.8-14 ANSYS, Advanced Analysis Techniques Guide, Release 11.0, ANSYS, Inc., 2007.

3.8-15 Johnson, T.E. Testing of Large Pre-stressing Tendon End Anchor Anchorage Regions. International Conference on Experience in the Design, Construction and Operation of Pre-stressed Concrete Pressure Vessels and Containments for Nuclear Reactors, University of York, England, 8-12 September 1975.

3.8-16 Deleted.

3.8-56 Dameron, R. A., Zhang, L., Rashid, Y. R., Vargas, M. S., Pretest Analysis of a 1:4-Scale Prestressed Concrete Containment Vessel Model, NUREG/CR-6685, U. S. Nuclear Regulatory Commission, Washington, D. C., October 2000.

3.8-57 Dameron, R. A., Hansen, B. E., Parker, D. R., Rashid, Y. R., Posttest Analysis of the NUPEC/NRC 1:4-Scale Prestressed Concrete Containment Vessel Model, NUREG/CR-6809, U. S. Nuclear Regulatory Commission, Washington, D. C., February 2003.

3.8-58 Embedment and Ground Water Effects on SSI for US-APWR Standard Plant, MUAP-11007, Rev. 2, Mitsubishi Heavy Industries, LTD., October 2011. MIC-03-03-00066 DCD_03.07.02-35

3.8-59 Varma, A. and Malushte, S. In-plane Shear Stiffness Recommendation, April 2011. DCD_03.07.02-35

3.8-60 Seismic Design Criteria for Structures, Systems, and Components, American Society of Civil Engineers, ASCE 43-05, 2005.

3.8-61 Ozaki, M. et al. Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-plane Shear, Nuclear Engineering and Design, Volume 228, 2004.

3.8-62 Kennedy, Robert P. Relationship Between Effective Linear Stiffness and Secant Stiffness for Pinched In-Plane Shear Behavior or Shear Walls, RPK-110318.1, March 2011.

3.8-63 Research Achievements of SC Structure and Strength Evaluation of US-APWR SC Structure Based on 1/10th Scale Test Results, MUAP-11005-P, Rev. 0, Mitsubishi Heavy Industries, Ltd., January 2011.

3.8-64 Damping Values for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.61, Rev. 1, U.S. Nuclear Regulatory Commission, March 2007.

3.8-65 FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, November 2000.

3.8-66 Evaluation of the Seismic Design Criteria in ASCE/SEI Standard 43-05 for Application to Nuclear Power Plants, NUREG/CR-6926, Brookhaven National Laboratory, March 2007.

3.8-67 In-Plane Behavior of Concrete Filled Steel (CFS) Elements, Presentation, Enclosure 1 to DCP_NRC_00278, Electronic ADAMS, NRC. Item ID 100130037, Accession Number ML 100050190. DCD_03.07.02-35

3.8-68 Containment Internal Structure Design and Validation Methodology, MUAP-11013, Rev. 1, Mitsubishi Heavy Industries, Ltd., August 2011.

3.8-69 A Study on the Structural Performance of SC Thick Walls, Annual Conference of Architectural Institute of Japan (Parts 1-3), 2003.

3.8-70 Containment Internal Structure: Stiffness and Damping for Analysis, MUAP-11018, Rev. 0, Mitsubishi Heavy Industries, Ltd., August 2011.

3.8-71 Containment Internal Structure: Design Criteria for SC Walls, MUAP-11019, Rev. 0, Mitsubishi Heavy Industries, Ltd., September 2011.

3.8-72 Containment Internal Structure: Anchorage and Connection Design and Detailing, MUAP-11020, Rev. 0, Mitsubishi Heavy Industries, Ltd., September 2011.

Table 3.8.3-2 Design Pressures within Containment Internal Structure

<table>
<tr><th>Compartment No.</th><th>Compartment</th><th>Design Pressure
psi</th><th></th></tr>
<tr><td>SG1</td><td>SG Compartment (25’-3” - 36’-5”)</td><td>18</td><td></td></tr>
<tr><td>SG2</td><td>SG Compartment (36’-5” - 46’-6.3”)</td><td>18</td><td></td></tr>
<tr><td>SG3</td><td>SG Compartment (46’-6.3” - 55’-1”)</td><td>~~7~~13</td><td>MIC-03-03-00066</td></tr>
<tr><td>SG4</td><td>SG Compartment (55’-1” - 79’-1”)</td><td>7</td><td></td></tr>
<tr><td>SG5</td><td>SG Compartment (79’-1” - 85’-9”)</td><td>~~7~~8</td><td>MIC-03-03-00066</td></tr>
<tr><td>SG6</td><td>SG Compartment (85’-9” - 95’-1”)</td><td>~~14~~34</td><td>MIC-03-03-00066</td></tr>
<tr><td>SG7</td><td>SG Compartment (95’-1” - 112’-0”)</td><td>10</td><td>MIC-03-03-00066</td></tr>
<tr><td>Pzr1</td><td>Pressurizer Surge Line Compartment
(25’-3” – 58’-5”)</td><td>2</td><td></td></tr>
<tr><td>Pzr 2</td><td>Pressurizer Compartment (58’-5” - 76’-1”)</td><td rowspan="5">14</td><td></td></tr>
<tr><td>Pzr 3</td><td>Pressurizer Compartment (76’-1” - 89’-9”)</td><td></td></tr>
<tr><td>Pzr 4</td><td>Pressurizer Compartment (89’-9” - 116’-8”)</td><td></td></tr>
<tr><td>Pzr 5</td><td>Pressurizer Compartment (116’-8” - 127’-10”)</td><td></td></tr>
<tr><td>Pzr 6</td><td>Pressurizer Compartment (127’-10” - 139’-6”)</td><td></td></tr>
<tr><td>V1</td><td>Gallery</td><td>39</td><td></td></tr>
<tr><td>V2</td><td>RV Annulus Lower</td><td>14</td><td></td></tr>
<tr><td>V3</td><td>NIS Box</td><td rowspan="2">18</td><td></td></tr>
<tr><td>V4</td><td>Incore Instrumentation Chase</td><td></td></tr>
</table>

Table 3.8.3-3 Summary of Containment Internal Structure Models and Analysis Methods

Computer Program and Model	Analysis Method	Purpose	Concrete Stiffness[(1)]	
Three Dimensional ANSYS FE of containment internal structure fixed at elevation ~~1 ft, 11 in.~~3 ft, 7 in.	~~Static~~Dynamic Analysis	To obtain member forces for seismic and mechanical loads	~~Monolithic~~ ~~Case 1~~ Condition A (Operating) Condition B (Accident)	MIC-03-03-00066
Three Dimensional ANSYS FE of containment internal structure ~~built into R/B whole model~~and R/B basemat	Static Analysis	To obtain member forces for thermal load	~~Monolithic~~ ~~Case 1[(2)]~~ Condition A (Operating) Condition B (Accident) ~~(Cracked Case 2)~~	DCD_03.07.02-35

Note~~s~~: DCD_03.07.02-35

1. See Table 3.8.3-4 for ~~stiffness case description~~description of stiffness conditions. MIC-03-03-00066

~~2. The stress analysis is performed based on the monolithic concrete stiffness, but the thermal stress is reduced by the reduction factor α (=0.5) which is described in the Notes of Table 3.8.3-4.~~ DCD_03.07.02-35

DCD_03.07.02-35
MIC-03-03-00066

Table 3.8.3-4 Summary of Containment Internal Structure Stiffness and Damping Values for Seismic Analysis

Structural Category	Description	Loading Condition A ($E_{ss} + T_o$)			Loading Condition B ($E_{ss} + T_a$)		
		Shear Stiffness	**Flexural Stiffness**	**Damping**	**Shear Stiffness**	**Flexural Stiffness**	**Damping**
1	SC Walls, T ≤ 56”	Uncracked $G_cA_c + G_sA_s$	Cracked-Transformed E_cI_{ct}	4%	Fully Cracked $0.5\,(\rho^{-0.42})A_sG_s$	Cracked-Transformed E_cI_{ct}	5%
2	SC Walls with T > 56”	Uncracked G_cA_c	Uncracked E_cI_c	4%	Cracked $0.5G_cA_c$	Cracked $0.5E_cI_c$	7%
3	Primary Shielding	Uncracked G_cA_c	Uncracked E_cI_c	4%	Uncracked G_cA_c	Uncracked E_cI_c	4%
4	Rienf. Conc. Slabs	Uncracked G_cA_c	Uncracked E_cI_c	4%	Uncracked G_cA_c	Cracked $0.5E_cI_c$	7%
5	Massive Reinf. Conc. Sections	Uncracked G_cA_c	Uncracked E_cI_c	4%	Uncracked G_cA_c	Uncracked E_cI_c	4%
6	Steel structure with non-structural concrete fill	No Concrete Stiffness or Damping Applied					

Note: Refer to Subsection 3.8.3.4 for application of damping values to seismic analyses.

Table 3.8.4-3 Load Combinations and Load Factors for Seismic Category I Concrete Structures and SC modules MIC-03-03-00066

LOAD COMBINATIONS AND FACTORS(1),(2)												
ACI 349-06 Load Combination:		1	2	3	4	5(7)	6(6)	7(6), (7)	8(6), (7)	9	10	11
Load Type												
Dead	***D***	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Liquid	***F***	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Live	***L***	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Earth	***H***	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Design pressure	P_d											
Normal pipe reactions	R_o	1.7	1.7	1.7	1.0	1.0				1.3	1.3	1.3
Normal thermal	T_o				1.0	1.0				1.2(5)	1.2(5)	1.2(5)
Wind	***W***			1.7								1.3
OBE	E_{ob}		1.7(3)					1.15(3)			1.3(3)	
SSE	E_{ss}				1.0(4)				1.0(4)			
Tornado	W_t					1.0						
Accident pressure	P_a						1.4(5)	1.15	1.0			
Accident thermal	T_a						1.0	1.0	1.0			
Accident thermal pipe reactions	R_a						1.0	1.0	1.0			
Pipe rupture reactions	Y_r							1.0	1.0			
Jet impingement	Y_j							1.0	1.0			
Pipe Impact	Y_m							1.0	1.0			
Acceptance Criteria(8)		U	U	U	U	U	U	U	U	U	U	U

MIC-03-03-00066

Notes:

1. Design per ~~ACI 349 Strength Design Method~~ACI 349-06 (Reference 3.8-8), Appendix C, for all load combinations MIC-03-03-00066
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 is taken as zero.
3. OBE loading is applicable for site-specific seismic category I SSCs, only if the value of site-specific OBE is set higher than 1/3 of the site-specific SSE.
4. SSE includes all seismic related hydrodynamic loads and percentage of live loads
5. Load factor adjusted in accordance with RG 1.142, Regulatory Position 6 (Reference 3.8-19). MIC-03-03-00066
6. The maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m including an appropriate dynamic load factor are used, unless an appropriate time history analysis is performed to justify otherwise.
7. Satisfy the load combination first without W_t, Y_r, Y_j, and Y_m. When considering concentrated loads, exceed~~e~~ances of local strengths and stresses may be considered in analyses for impactive or impulsive effects in accordance with ~~ACI 349-97, Appendix C~~ACI 349-06 (Reference 3.8-8), Appendix F, except as noted in RG 1.142 Regulatory Positions 10 and 11. MIC-03-03-00066
8. The required strength U shall be equal to or greater than the strength required to resist the factored loads and/or related internal moments and forces, for each of the load combinations shown in this table.

Table 3.8.4-5 Summary of R/B and PS/Bs Models and Analysis Methods

Computer Program and Model	Analysis Method	Purpose	Concrete Stiffness
Three-dimensional ANSYS FE of R/B ~~whole~~complex model	Static Analysis	To obtain member forces including thermal load	Monolithic(1)
Three-dimensional ANSYS FE of PS/B model	Static Analysis	To obtain member forces	Monolithic~~(1)~~

MIC-03-03-00066

Note:

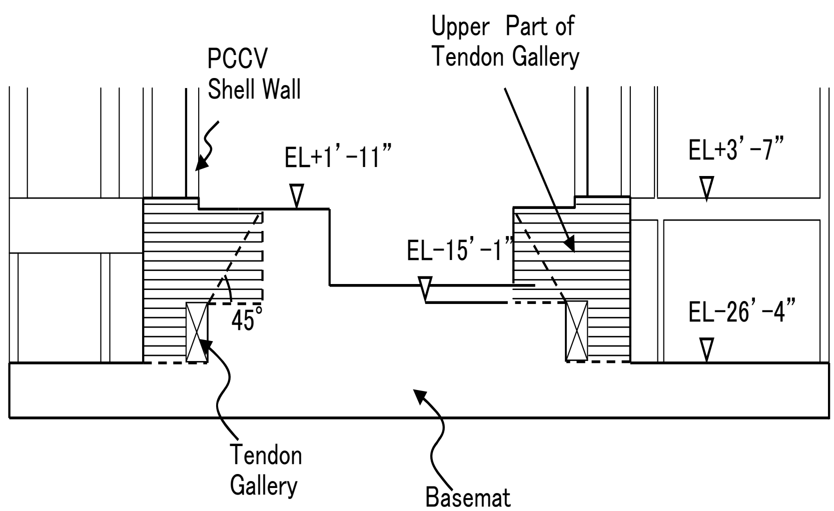
1. The stress analysis is performed based on the monolithic concrete stiffness, but the thermal stress is reduced by the reduction factor α (=0.5).

Table 3.8.5-2 Concrete Properties

Part		Compressive Strength f'c	Modulus of Elasticity Ec(1)	Poisson's Ratio υ	Thermal Expansion Coefficient α	Unit Weight γ
PCCV		7,000 psi	4,769 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
Containment Internal Structure		4,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
R/B		4,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
Basemat	Peripheral	4,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
	Upper part of Tendon Gallery	7,000 psi	4,769 ksi	0.17	0.99×10^{-5}/°C 5.5×10^{-6}/°F	150lb/ft^3

NOTE :

1. 1. Ec=57,000(Fc)$^{1/2}$ psi (ACI 349-06, 8.5.1) MIC-03-03-00066



DCD_03.07.02-35

Security-Related Information – Withheld Under 10 CFR 2.390

Figure 3.8.3-13 Structural Categories Between Elevations 21'-0" and 35'-11"

MIC-03-03-00066

DCD_03.07.02-35

Security-Related Information – Withheld Under 10 CFR 2.390

Figure 3.8.3-14 Structural Categories Between Elevations 37’-9” and 62’-4”

MIC-03-03-00066