3.8 Design of Category I Structures

3.8.1 Concrete Containment

This subsection is not applicable to the AP1000.

3.8.2 Steel Containment

3.8.2.1 Description of the Containment

3.8.2.1.1 General

This subsection describes the structural design of the steel containment vessel and its parts and appurtenances. The steel containment vessel is an integral part of the containment system whose function is described in Section 6.2. It serves both to limit releases in the event of an accident and to provide the safety-related ultimate heat sink.

The containment vessel is an ASME metal containment. The information contained in this subsection is based on the design specification and preliminary design and analyses of the vessel. Final detailed analyses will be documented in the ASME Design Report.

The containment arrangement is indicated in the general arrangement figures in Section 1.2. The portion of the vessel above elevation 132′-3″ is surrounded by the shield building but is exposed to ambient conditions as part of the passive cooling flow path. A flexible watertight and airtight seal is provided at elevation 132′-3″ between the containment vessel and the shield building. The portion of the vessel below elevation 132′-3″ is fully enclosed within the shield building.

Figure 3.8.2-1 shows the containment vessel outline. It is a free-standing, cylindrical steel vessel with ellipsoidal upper and lower heads. [*The containment vessel has the following design characteristics:*

Diameter: 130 feet

Height: 215 feet 4 inches

Design Code: ASME III, Div. 1

Material: SA738, Grade B

Design Pressure: 59 psig

Design Temperature: 300°F

Design External Pressure: 1.7 psid

Lower Personnel Airlock: Elevation 110′-6″ and 107 degrees azimuth

Lower Equipment Hatch: Elevation 112′-6″ and 126 degrees azimuth

Upper Personnel Airlock Elevation 138′-7″ and 107 degrees azimuth

Upper Equipment Hatch Elevation 141′-6″ and 67 degrees azimuth

External Stiffener: Elevation 131′-9″

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Internal Stiffener: Elevation 170′-0″

Bottom Head Tangent Line Elevation 104′-1 1/2″

Upper Head Tangent Line Elevation 244′-2 1/2″

The tangent line is the elevation at which the vessel transitions from the cylinder to the head.

The wall thickness in most of the cylinder is 1.75 inches. The wall thickness of the lowest course of the cylindrical shell is increased to 1.875 inches to provide margin in the event of corrosion in the embedment transition region. The thickness of the heads is 1.625 inches.]* The heads are [*ellipsoidal*]* with a major diameter of 130 feet and a height of 37 feet, 7.5 inches.

The containment vessel includes the shell, hoop stiffeners and crane girder, equipment hatches, personnel airlocks, penetration assemblies, and miscellaneous appurtenances and attachments. The design for external pressure is dependent on the spacing of the hoop stiffeners and crane girder, which are shown on Figure 3.8.2-1. [*The spacing between each pair of ring supports (the bottom flange of the crane girder, the hoop stiffeners, and the concrete floor at elevation 100′-0″) is less than 50 feet, 6 inches. The design of the stiffeners and polar crane girder provides equal or greater radial and rotational stiffness than the design evaluated for the design certification.*]*

The polar crane is designed for handling the reactor vessel head during normal refueling. The crane girder and wheel assemblies are designed to support a special trolley to be installed in the event of steam generator replacement.

The containment vessel supports most of the containment air baffle as described in Subsection 3.8.4. The air baffle is arranged to permit inspection of the exterior surface of the containment vessel. Steel plates are welded to the dome as part of the water distribution system, described in Subsection 6.2.2. The polar crane system is described in Subsection 9.1.5.

3.8.2.1.2 Containment Vessel Support

The bottom head is embedded in concrete, with concrete up to elevation 100′ on the outside and to the maintenance floor at elevation 107′-2″ on the inside. The containment vessel is assumed as an independent, free-standing structure above elevation 100′. The thickness of the lower head is the same as that of the upper head. There is no reduction in shell thickness even though credit could be taken for the concrete encasement of the lower head.

Vertical and lateral loads on the containment vessel and internal structures are transferred to the basemat below the vessel by shear studs, friction, and bearing. The shear studs are not required for design basis loads. They provide additional margin for earthquakes beyond the safe shutdown earthquake.

Seals are provided at the top of the concrete on the inside and outside of the vessel to prevent moisture between the vessel and concrete. A typical cross section design of the seal is presented in Figure 3.8.2-8, sheets 1 and 2.

3.8.2.1.3 Equipment Hatches

Two equipment hatches are provided. One is at the operating floor (elevation 135′-3″) with an inside diameter of 16 feet. The other is at floor elevation 107′-2″ to permit grade-level access into the containment, with an inside diameter of 16 feet. The hatches, shown in Figure 3.8.2-2, consist of a cylindrical sleeve with a pressure seated dished head bolted on the inside of the vessel. The containment internal pressure acts on the convex face of the dished head and the head is in

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

compression. The flanged joint has double O-ring or gum-drop seals with an annular space that may be pressurized for leak testing the seals. Each of the two equipment hatches is provided with an electrically powered hoist and with a set of hardware, tools, equipment and a self-contained power source for moving the hatch from its storage location and installing it in the opening. [*The information in Figure 3.8.2-2 that is considered to be Tier 2* information is the minimum thickness of the hatch cover, the inside diameter of the sleeve, the diameter of the insert plate, the minimum thickness of the insert plate, and the nominal spherical radius of the hatch cover.*]*

3.8.2.1.4 Personnel Airlocks

Two personnel airlocks are provided, one located adjacent to each of the equipment hatches. Figure 3.8.2-3 shows the typical arrangement. Each personnel airlock has about a 10-foot external diameter to accommodate a door opening of width 3 feet 6 inches and height 6 feet 8 inches. The airlocks are long enough to provide a clear distance of 8 feet, which is not impaired by the swing of the doors within the lock. The airlocks extend radially out from the containment vessel through the shield building. They are supported by the containment vessel. [*Area reinforcement for the personnel airlocks is provided by a minimum of 3-3/4-inch-thick insert plates. The surface area of the personnel airlock insert plate, not including the sleeve, is a minimum of 51.9 ft^2.*]*

Each airlock has two double-gasketed, pressure-seated doors in series. The doors are mechanically interlocked to prevent simultaneous opening of both doors and to allow one door to be completely closed before the second door can be opened. The interlock can be bypassed by using special tools and procedures.

3.8.2.1.5 Mechanical Penetrations

The mechanical penetrations consist of the fuel transfer penetration and mechanical piping penetrations and are listed in Table 6.2.3-1. Area is added to the shell by the addition of an insert plate that is thicker than the shell or by increasing the thickness of the nozzle neck or a combination of both. This piping penetration design is then evaluated for external loads on the penetration imposed by the piping system.

Design requirements for the mechanical penetrations are as follows:

- Design and construction of the process piping follow ASME, Section III, Subsection NC. Design and construction of the remaining portions follow ASME Code, Section III, Subsection NE. The boundary of jurisdiction is according to ASME Code, Section III, Subsection NE.
- Penetrations are designed to maintain containment integrity under design basis accident conditions, including pressure, temperature, and radiation.
- Guard pipes are designed for pipe ruptures as described in Subsection 3.6.2.1.1.4.
- Bellows are stainless steel or nickel alloy and are designed to accommodate axial and lateral displacements between the piping and the containment vessel. These displacements include thermal growth of the main steam and feedwater piping during plant operation, relative seismic movements, and containment accident and testing conditions. Cover plates are provided to protect the bellows from foreign objects during construction and operation. The cover plates for the main steam bellows are removable to permit in-service inspection. Access to permit in-service inspection of the pressure retaining portions of the main feedwater bellows is provided via holes in the weld ends of the bellows assembly.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Figure 3.8.2-4, sheet 1, shows typical details for the main steam penetration. This includes bellows to minimize piping loads applied to the containment vessel and a guardpipe to protect the bellows and to prevent overpressurization of the containment annulus in a postulated pipe rupture event. Details for the main feedwater penetration are shown in Figure 3.8.2-4, Sheet 8. [*The main steam and feedwater penetrations are combined into a common 3-3/4-inch-thick insert plate. This thickness is a minimum value. The main steam penetration has an inside sleeve diameter of 57 inches. The feedwater penetration has an inside sleeve diameter of 38 inches.*]* The insert plates for the main steam and feedwater penetrations are shown in Figure 3.8.2-4, Sheet 7. The insert plate also includes the penetration for the 6-inch-diameter startup feedwater pipe. The insert plate is designed in accordance with NE-3330, "Openings and Reinforcement," of the ASME Code.

Figure 3.8.2-4, sheet 2, shows typical details for the startup feedwater penetration. This includes a guardpipe to prevent overpressurization of the containment annulus in a postulated pipe rupture event. Similar details are used for the steam generator blowdown penetration.

Figure 3.8.2-4, sheet 3, shows typical details for the normal residual heat removal penetration. Similar details are used for other penetrations below elevation 107′-2″ where there is concrete inside the containment vessel. The flued head is integral with the process piping and is welded to the containment sleeve. The welds are accessible for in-service inspection. The containment sleeve is separated from the concrete by compressible material with one exception.

For Vogtle Unit 3, the as-built configuration of penetration RNS-PY-C01 (P19) differs from Figure 3.8.2-4, sheet 3. During construction, a layer of concrete intruded between the compressible material and the containment sleeve, where it remains. This condition does not impair the safety function of the penetration. This configuration is only applicable to penetration RNS-PY-C01 for Vogtle Unit 3.

Figure 3.8.2-4, sheet 4 shows representative details for the other mechanical penetrations. These consist of:

- A sleeve welded to containment with flued head welded to the sleeve (detail A).
- A sleeve welded to containment with the process pipe welded directly to the sleeve and in direct contact with the process fluid (detail B).
- A sleeve welded to containment with process pipe passing through the sleeve (detail C).
- A sleeve welded to a thicker insert place welded to containment with sleeve welded to pipe and in contact with process fluid (detail D).

Flued heads are used for stainless piping greater than 2 inches in nominal diameter and for piping with high operating temperatures.

Note: The type of weld connecting the sleeve to pipe may vary.

The fuel transfer penetration, shown in Figure 3.8.2-4, sheet 5, is provided to transfer fuel between the containment and the fuel handling area of the auxiliary building. The fuel transfer tube is welded to the penetration sleeve. The containment boundary is a double-gasketed blind flange at the refueling canal end. The expansion bellows are not a part of the containment boundary. Rather, they are water seals during refueling operations and accommodate differential movement between the containment vessel, containment internal structures, and the auxiliary building.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.2.1.6 Electrical Penetrations

Figure 3.8.2-4, sheet 6, shows a typical 18-inch-diameter electrical penetration. The penetration assemblies consist of conductor modules (or medium voltage cable modules in a similar 18-inch-diameter penetration) passing through a bulkhead attached to the containment nozzle. Electrical design of these penetrations is described in Subsection 8.3.1.1.6.

Electrical penetrations are designed to maintain containment integrity under design basis accident conditions, including pressure, temperature, and radiation. Double barriers permit testing of each assembly to verify that containment integrity is maintained. Design and testing is according to IEEE Standard 317-83 and IEEE Standard 323-74.

3.8.2.1.7 Instrument Line Penetrations

Instrument line penetrations are designed to maintain containment integrity under design basis accident conditions, including pressure, temperature, and radiation.

Figure 3.8.2-4, sheet 4, detail B, shows typical details for the containment pressure instrumentation penetrations. The penetrations consist of sleeves welded to the containment vessel. Pressure transmitters outside containment are connected to pressure sensors inside containment by sealed, fluid-filled tubing (capillary), which passes through the sleeves. The capillary tubing is welded directly to the sleeve at a tubing coupling, which has a thicker wall and larger diameter than the capillary tubing.

Design and construction of the penetrations are in accordance with ASME Section III. The penetration sleeves, including the welds to the tubing couplings, follow ASME Section III, Subsection NE. Because ASME Section III, Subsection NCA excludes the sealed-tubing instrument configuration from the scope of Section III, the capillary tubing is designed and fabricated in accordance with ASME B31.1.

3.8.2.2 Applicable Codes, Standards, and Specifications

[*The containment vessel is designed*]* and constructed [*according to the 2001 edition of the ASME Code, Section III, Subsection NE, Metal Containment, including the 2002 Addenda. Stability of the containment vessel and appurtenances is evaluated using ASME Code, Case N-284-1, Metal Containment Shell Buckling Design Methods, Class MC, Section III, Division 1, as published in the 2001 Code Cases, 2001 Edition, July 1, 2001.*]*

Structural steel nonpressure parts, such as ladders, walkways, and handrails are designed to the requirements for steel structures defined in Subsection 3.8.4.

Section 1.9 discusses compliance with the Regulatory Guides and the Standard Review Plans.

3.8.2.3 Loads and Load Combinations

Table 3.8.2-1 summarizes the design loads, load combinations and ASME Service Levels. They meet the requirements of the ASME Code, Section III, Subsection NE. The loads and load combinations used in the analysis are considered to be part of the method of evaluation. The containment vessel is designed for the following loads specified during construction, test, normal plant operation and shutdown, and during accident conditions:

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

L Live loads or their related internal moments and forces, including crane loads

P_o Operating pressure loads during normal operating conditions resulting from pressure variations either inside or outside containment

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

R_o Piping and equipment reactions during normal operating conditions, based on the most critical transient or steady-state condition

W Loads generated by the design wind on the portion of the containment vessel above elevation 132′, as described in Subsection 3.3.1.1

E_s Loads generated by the safe shutdown earthquake (SSE) as described in Section 3.7

W_t Loads generated by the design tornado on the portion of the containment vessel above elevation 132′, as described in Subsection 3.3.2

P_t Test pressure

P_d Containment vessel design pressure that exceeds the pressure load generated by the postulated pipebreak accidents and passive cooling function

P_e Containment vessel external pressure

T_a Thermal loads under thermal conditions generated by the postulated break or passive cooling function and including T_o. This includes variations around the shell due to the surrounding buildings and maldistribution of the passive containment cooling system water.

R_a Piping and equipment reactions under thermal conditions generated by the postulated break, as described in Section 3.6, and including R_o

Y_r Loads generated by the reaction on the broken high-energy pipe during the postulated break, as described in Section 3.6

Y_j Jet impingement load on a structure generated by the postulated break, as described in Section 3.6

Y_m Missile impact load on a structure generated by or during the postulated break, as from pipe whipping, as described in Section 3.6

Post-accident flooding load combination is not applicable in the design of the AP1000 containment vessel. The post-loss-of-coolant accident (LOCA) flooding event is enveloped by the other design cases.

The AP1000 addresses the production of large quantities of hydrogen from the oxidation of zirconium and other metals as a result of a postulated severe accident. The AP1000 includes hydrogen igniters inside containment to ensure that hydrogen generated in a severe accident is burned prior to reaching an explosive mixture. The discussion of the generation and burning of hydrogen as a result of a severe accident is included in Section 19.41.

The containment vessel is protected from the direct effects of wind/tornado loads (and associated potential missiles) by virtue of its location inside the shield building. The differential pressure effects of a tornado are also reduced because of the location and are bounded by other pressure loadings for which the containment vessel is designed.

The containment is evaluated for the deterministic severe accident pressure capacity. This evaluation is discussed in Subsection 3.8.2.4.2, “Evaluation of Ultimate Capacity.” According to 10 CFR 50.44, the hydrogen generated pressure loads from 100 percent fuel clad-coolant reaction plus the peak pressure from a hydrogen burn must be less than ASME Service Level C (not including buckling). The Service Level C maximum capacity is 117 psig at 300°F as presented in Subsection 3.8.2.4.2.8 The peak pressure from the 100 percent fuel clad-coolant reaction plus the hydrogen burn (Pg1 + Pg2) is 90.3 psig as reported in Section 41.11 and Table 41-4 of the AP1000 Probabilistic Risk Assessment report. The severe accident conditions are beyond design basis accidents, and the load combinations for these severe accident evaluations are not included in the load combinations and service limits for the containment vessel.

The AP1000 does not have a post-accident inerting system. Therefore, there is no load combination that includes inerting of the containment.

Note that loads associated with flooding of the containment below elevation 107′ are resisted by the concrete structures and not by the containment vessel.

3.8.2.4 Design and Analysis Procedures

The design and analysis procedures for the containment vessel are according to the requirements of the ASME Code, Section III, Subsection NE.

The analyses are summarized in Table 3.8.2-4. The detailed analyses will use a series of general-purpose finite element, axisymmetric shell and special purpose computer codes to conduct such analyses. Code development, verification, validation, configuration control, and error reporting and resolution are according to the Quality Assurance requirements of Chapter 17.

3.8.2.4.1 Analyses for Design Conditions

3.8.2.4.1.1 Axisymmetric Shell Analyses

The containment vessel is modelled as an axisymmetric shell and analyzed using the ANSYS computer program. A model used for static analyses is shown in Figure 3.8.2-6.

Dynamic analyses of the axisymmetric model, which is similar to that shown in Figure 3.8.2-6, are performed to obtain frequencies and mode shapes. These are used to confirm the adequacy of the containment vessel stick model as described in Subsection 3.7.2.3.2. Stress analyses are performed for each of the following loads:

- Dead load
- Internal pressure
- Seismic
- Polar crane wheel loads
- Wind loads
- Thermal loads

The seismic analysis performed envelopes all soil conditions. The global seismic loads are applied as equivalent static accelerations using the maximum accelerations shown in Table 3.8.2-5. These accelerations are the maximum accelerations from the nuclear island stick model on hard rock. The global member forces from the equivalent static case exceed those from the soil cases for soil conditions described in Appendix 3G. Based on these comparisons, the design acceleration values used for the global analyses are appropriate for both the hard rock and the soil sites. The seismic analysis of the nuclear island is discussed in Section 3.7 and Appendix 3G. The torsional moments, which include the effects of the eccentric masses, are increased to account for accidental torsion and are evaluated in a separate calculation.

The results of these load cases are factored and combined in accordance with the load combinations identified in Table 3.8.2-1. These results are used to evaluate the general shell away from local penetrations and attachments, that is, for areas of the shell represented by the axisymmetric geometry. The results for the polar crane wheel loads are also used to establish local shell stiffnesses for inclusion in the containment vessel stick model described in Subsection 3.7.2.3. The results of the analyses and evaluations are included in the containment vessel design report.

Design of the containment shell is primarily controlled by the internal pressure of 59 psig. The meridional and circumferential stresses for the internal pressure case are shown in Figure 3.8.2-5. The most highly stressed regions for this load case are the portions of the shell away from the hoop stiffeners and the knuckle region of the top head. In these regions the stress intensity is close to the allowable for the design condition.

Table 3.8.2-1 includes a design load combination to address external pressure. For the design external pressure, a conservatively large magnitude of 1.7 psi differential pressure is used. Design external pressure is defined as a value greater than the external pressure at which the vacuum relief system will open and mitigate the external pressure. This is a part of the containment air filtration system (see Subsection 9.4.7). Upon actuation, the external pressure transient is immediately controlled and the external pressure is relieved. This design external pressure is combined with a coincident -40°F outside air temperature, which corresponds to a -18.5°F metal temperature for the portions of the containment vessel shell not insulated from ambient conditions. The portions of the containment vessel shell that are below the external stiffener are insulated from the cold outside air conditions and result in a metal temperature of 70°F.

A bounding case was analyzed to provide an indication of the margin to acceptance criteria associated with the minimum allowable service metal temperature for the AP1000 containment vessel. Various types of transients were considered to evaluate the minimum service metal temperature, including inadvertent fan cooler cases, inadvertent passive containment cooling system (PCS) actuation, and loss of ac power. The evaluation considered variations in initial conditions for parameters, including humidity, internal temperature, external temperature, and wind speed. These evaluations demonstrate that the -18.5°F service metal temperature is adequate.

Design external pressure is used in load combinations that include thermal loads and are used to evaluate Service Level A and D stress limits. These external pressure conditions are included in the loading combinations in Table 3.8.2-1.

Operating pressures range from -0.2 psig to 1.0 psig, which are then combined with an ambient temperature for the containment vessel. Design internal pressure is 59 psig combined with a containment vessel metal temperature of 300°F to be evaluated in the ASME service limits as well as the design conditions.

A load combination that combines design wind plus internal design pressure is not included in Table 3.8.2-1 because the wind loads are small (within the normal operating range for containment pressure) and because the combination of the design wind and accident pressure is a lower probability than either the design wind or the accident pressure acting alone.

Major loads that induce compressive stresses in the containment vessel are internal and external pressure and crane and seismic loads. Each of these loads and the evaluation of the compressive stresses are discussed below.

- Internal pressure causes compressive stresses in the knuckle region of the top head and in the equipment hatch covers. The evaluation methods are similar to those discussed in Subsection 3.8.2.4.2 for the ultimate capacity.

- Evaluation of external pressure loads is performed in accordance with ASME Code, Section III, Subsection NE, Paragraph NE-3133.

- Crane wheel loads due to crane dead load, live load, and seismic loads result in local compressive stresses in the vicinity of the crane girder. These are evaluated in accordance with ASME Code, Case N-284.

- Overall seismic loads result in axial compression and tangential shear stresses at the base of the cylindrical portion. These are evaluated in accordance with ASME Code, Case N-284.

The bottom head is embedded in the concrete base at elevation 100 feet. This leads to circumferential compressive stresses at the discontinuity under thermal loading associated with the design basis accident. The containment vessel design includes a Service Level A combination in which the vessel above elevation 107′-2″ is specified at the design temperature of 300°F and the portion of the embedded vessel (and concrete) below elevation 100 feet is specified at a temperature of 70°F. The temperature profile for the vessel is linear between these elevations. Containment shell buckling close to the base is evaluated against the criteria of ASME Code, Case N-284.

Revision 1 of Code Case N-284 is used for the evaluation of the containment shell and equipment hatches.

3.8.2.4.1.2 Local Analyses

The penetrations and penetration reinforcements are designed in accordance with the rules of ASME III, Subsection NE. The design of the large penetrations for the two equipment hatches and the two airlocks use the results of finite element analyses which consider the effect of the penetration and its dynamic response (Reference 53).

The personnel airlocks and equipment hatches are modeled in a 3-D shell finite element model of the containment. A 3D shell, finite element model of the containment vessel was developed in ANSYS to consider the effect of the penetrations and their quasi-static response due to a seismic event. The large masses and local stiffness of the personnel locks and equipment hatches are discretely modeled. The polar crane wheel loads are incorporated by appropriate loadings (dead load and seismic loadings). The bottom of the model is fixed at elevation 100′ where the containment vessel is embedded in concrete. This means that rotations and displacements are conservatively fixed at this location.

Static analyses are performed using the finite element model shown in Figure 3.8.2-7 for internal pressure, dead load (including the polar crane in the parked position), thermal loads and seismic loads. The global seismic loads are applied as equivalent static accelerations using the maximum accelerations shown in Table 3.8.2-5. The amplified local responses are included separately for each of the four penetrations. Local seismic axial and rotational accelerations about both horizontal and vertical axes are applied based on the maximum amplified response determined from a time history analysis on a less refined dynamic model with seismic time histories at elevation 100′.

Stresses are evaluated against the stress intensity criteria of ASME Section III, Subsection NE for the load combinations described in Table 3.8.2-1. Stability is evaluated against ASME Code Case N-284. Local stresses in the regions adjacent to the major penetrations are evaluated in accordance with paragraph 1700 of the code case. Stability is not evaluated in the reinforced penetration neck and insert plate which are substantially stiffer than the adjacent shell.

3.8.2.4.2 Evaluation of Ultimate Capacity

The capacity of the containment vessel has been calculated for internal pressure loads for use in the probabilistic risk assessment analyses and severe accident evaluations. These analyses include the evaluation of the peak pressure from the hydrogen-generated pressure loads from 100-percent fuel cladding metal-water reaction plus the hydrogen burn. Each element of the containment vessel boundary was evaluated to estimate the maximum pressure at an ambient temperature of 100°F corresponding to the following stress and buckling criteria:

- Deterministic severe accident pressure capacity corresponding to ASME Service Level C limits on stress intensity, ASME paragraph NE-3222, and ASME Code Case N-284 for buckling of the equipment hatch covers, and 60 percent of critical buckling for the top head. The deterministic severe accident pressure capacity corresponds to the approach in SECY 93-087, to maintain a reliable leak-tight barrier approximately 24 hours following the onset of core damage under the more likely severe accident challenges. This approach was approved by the Nuclear Regulatory Commission as outlined in the Staff Requirements Memorandum on SECY-93-087 - Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light Water Reactor (ALWR) Designs, Dated July 21, 1993.

- Best estimate capacity corresponding to gross membrane yield at the ASME-specified minimum yield stress (SA738, Grade B, yield stress = 60 ksi, ultimate stress = 85 ksi), and critical buckling for the equipment hatch covers and top head.

The results are shown in Table 3.8.2-2. The analyses at a temperature of 100°F are described in the following paragraphs for each element. The critical regions identified in this table are then examined further for their response at higher temperatures. This results in the best-estimate capacity based on the ASME-specified minimum yield properties. The evaluation considered the containment boundary elements including:

- Cylindrical shell
- Top and bottom heads
- Equipment hatches and covers
- Personnel airlocks
- Mechanical and electrical penetrations

The evaluation identified the most likely failure mode to be that associated with gross yield of the cylindrical shell. Loss of containment function would be expected to occur because the large post-yield deflections would lead to local failures at penetrations, bellows, or other local discontinuities.

3.8.2.4.2.1 Tensile Stress Evaluation of Shell

Results of the axisymmetric analyses of the cylinder and top head described in Subsection 3.8.2.4.1 for dead load and internal pressure were evaluated to determine the pressure at which stresses reach yield at an ambient temperature of 100°F. The analyses assume the shell is fixed at elevation 100′, where the bottom head is embedded in concrete. The steel bottom head is identical to the top head and has a pressure capability greater than the top head due to the additional strength of the embedment concrete.

The allowable stress intensity under Service Level C loads is equal to yield. This corresponds to an internal pressure of 135 psig. The critical section is the cylinder, where the general primary membrane stress intensity is greatest.

The best-estimate yield analysis uses the von Mises criterion to establish yield rather than the more conservative ASME stress intensity approach. This increases the yield stress by about 15 percent for the cylinder, where the longitudinal stress is equal to one-half of the hoop stress resulting in first yield at an internal pressure of 155 psig. At this pressure, hoop stresses in the cylinder reach yield. The radial deflection is about 1.6 inches. As pressure increases further, large deflections occur. For a material such as SA738, where the yield plateau extends from a strain of 0.2 percent to 0.6 percent, deflections would increase to 4.8 inches at yield without a substantial increase in pressure. Strain hardening would then permit a further increase in pressure with large radial deflections, as described in Subsection 3.8.2.4.2.6.

3.8.2.4.2.2 Buckling Evaluation of Top Head

The top head has a radius-to-height ratio of 1.728. This is not as shallow as most ellipsoidal or torispherical heads, which typically have a radius-to-height ratio of 2. The ratio was specifically selected to minimize the local stresses and buckling in the knuckle region due to internal pressure. As the ratio decreases, the magnitude of compressive stresses in the knuckle region decreases; for a radius-to-height ratio of 1.4 or smaller, there are no compressive stresses and therefore there is no potential for buckling.

Theoretical Buckling Capacity

The top head was analyzed using the BOSOR-5 computer code (Reference 1). This code permits consideration of both large displacements and nonlinear material properties. It calculates shell stresses and checks stability at each load step. The analysis included a portion of the cylinder with a thickness of 1.625 inches. In this analysis, yield of the cylinder started at a pressure of 144 psig using elastic – perfectly plastic material properties, a yield stress of 60 ksi, and the von Mises yield criterion. Yield of the top of the crown started at an internal pressure of 146 psig. Yield of the knuckle region started at 152 psig. A theoretical plastic buckling pressure of 174 psig was determined. At this pressure, the maximum effective prebuckling strain was 0.23 percent in the knuckle region where buckling occurred and 2.5 percent at the crown. The maximum deflection at the crown was 15.9 inches. A similar analysis was performed using nonlinear material properties considering the effects of residual stresses; buckling did not occur in this analysis, and failure would occur once strains at the crown reach ultimate. The failure mode was found to be an axisymmetric plastic collapse resulting from excessive vertical displacements at the crown. The maximum displacement was 43 inches at 195 psig.

Predicted Pressure Capacity

The actual buckling capacity may be lower than the theoretical buckling capacity because of effects not included in the analysis such as imperfections and residual stresses. This is considered by the use of capacity reduction factors that are based upon a correlation of theory and experiment. The capacity reduction factor for the top head was evaluated based on comparisons of BOSOR-5 analyses against test results of ellipsoidal and torispherical heads. This evaluation is described below and concludes that no reduction in capacity need be considered; that is, a capacity reduction factor of 1.0 is appropriate.

The knuckle region of ellipsoidal and torispherical heads is subjected to meridional tension and circumferential compression. The meridional tension tends to stabilize the knuckle region and reduces its sensitivity to imperfection. The radius-to-height ratio of 1.728 of the AP1000 head results in a larger ratio of meridional tension to circumferential compression than on shallower heads, further reducing the sensitivity to imperfection.

Welding Research Council Bulletin 267 (Reference 22) shows a comparison of BOSOR-5 predictions of buckling against the results of 20 tests of small head models. These results are summarized in Table 4 of the reference and show ratios (capacity reduction factors) of actual buckling to the

BOSOR-5 prediction with an average of 1.2. Only one of the 20 cases shows a capacity reduction factor less than 1.0.

Table 3.8.2-3 shows the key parameters, test results, and BOSOR-5 predictions for two large, fabricated 2:1 torispherical heads tested and reported in NUREG/CR-4926 (Reference 23). The theoretical plastic buckling pressure predicted by BOSOR-5 represents initial buckling based on actual material properties. The initial buckling did not cause failure for either of the tests, and test pressure continued to increase until rupture occurred in the spherical cap. The collapse pressures were three to four times the initial buckling pressures.

- **Test Head 1** – The test result of 58 psig is 79 percent of the predicted theoretical plastic buckling pressure of 74 psig. Many of the buckles occurred directly on the meridional weld seams of the knuckle. The knuckle welds were noticeably flatter than the corresponding welds of the Test 2 head. The as-built configuration extended inside the theoretical shape at some of the meridional weld seams and was most pronounced at the location of the first observed buckle. Model 1 exceeded the tolerances for formed heads specified for containment vessels in NE-4222.2 of ASME, Section III, Subsection NE.

- **Test Head 2** – The test result of 106 psi is 100 percent of the BOSOR-5 predicted theoretical plastic buckling pressure. For test head 2, the welds had no noticeable flat spots and there was a smooth transition between the sphere and knuckle sections. Test head 2 was well within the Code allowable deviations.

The low-capacity reduction factor of 0.79 for test head 1 is attributed to excessive imperfections associated with the fabrication of relatively thin plate (0.196 inch). These imperfections were visible and were outside the tolerances permitted by the ASME Code. The results of test head 1 are therefore not considered applicable to the AP1000. The results of test head 2 and of the small-scale models described in the Welding Research Council Bulletin support the application of a capacity reduction factor of 1.0.

The capacity of the AP1000 head was also investigated using an approach similar to that permitted in ASME Code, Case N284. This code case provides alternate rules for certain containment vessel geometries such as cylindrical shells. The theoretical elastic buckling pressure was calculated to be 536 psi using the linear elastic computer code, BOSOR-4 (Reference 24). A reduction factor (defined as the product of the capacity reduction factor and the plastic reduction factor) was established as 0.385 based on the lower bound curve of test results of 20 ellipsoidal and 28 torispherical test specimens, which also include the two large fabricated heads previously discussed. This resulted in a predicted buckling capacity of 206 psig.

The preceding paragraphs addressed incipient buckling. It is concluded that buckling would not occur prior to reaching the pressure of 174 psig predicted in the BOSOR-5 analyses. Tests indicate that pressure can be significantly increased prior to rupture after the formation of the initial buckles. Failure would occur when local strains reach ultimate either close to a local buckle in the knuckle or at the center of the crown. The best estimate capacity of the head is taken as the theoretical plastic buckling pressure of 174 psig predicted in the BOSOR-5 analyses.

The deterministic severe accident pressure capacity is taken as 60 percent of critical buckling. This is consistent with the safety factor for Service Level C in ASME Code, Case N-284 and results in a containment head capacity of 104 psig.

3.8.2.4.2.3 Equipment Hatches

SECY 93-087 permits evaluation of certain severe accident scenarios against ASME Service Level C limits. The equipment hatch covers were evaluated for buckling against ASME paragraph NE-3222 and according to ASME Code, Case N-284. Use of ASME Code, Case N-284 for this application was confirmed to be appropriate by ASME. The containment internal pressure acts on the convex face of the dished head and the hatch covers are in compression under containment internal pressure loads. The critical buckling capacity is based on classical buckling capacities reduced by capacity reduction factors to account for the effects of imperfections and plasticity. These capacity reduction factors are based on test data and are generally lower-bound values for the tolerances specified in the ASME Code.

The critical buckling pressure is 211 psig for the 16-foot-diameter hatch at an ambient temperature of 100°F. For the Service Level C limits in accordance with paragraph NE 3222, a safety factor of 2.50 is specified, resulting in capabilities of 84 psig (16-foot-diameter). For the Service Level C limits in accordance with Code Case N284, a safety factor of 1.67 is specified, resulting in capabilities of 126 psig (16-foot-diameter).

Typical gaskets have been tested for severe accident conditions as described in NUREG/CR-5096 (Reference 25). The gaskets for the AP1000 will be similar to those tested with material such as Presray EPDM E 603. For such gaskets the onset of leakage occurred at a temperature of about 600°F.

3.8.2.4.2.4 Personnel Airlocks

The capacity of the personnel airlocks was determined by comparing the airlock design to that tested and reported in NUREG/CR-5118 (Reference 3). Critical parameters are the same, so the results of the test apply directly. In the tests the inner door and end bulkhead of the airlock withstood a maximum pressure of 300 psig at 400°F. The capacity of the airlock is therefore at least 300 psig at ambient temperature. The maximum pressure corresponding to Service Level C is conservatively estimated by reducing this capacity in the ratio of the minimum specified material yield to ultimate.

3.8.2.4.2.5 Mechanical and Electrical Penetrations

Subsections 3.8.2.1.3 through 3.8.2.1.7 describe the containment penetrations. Penetration reinforcement is designed following the area replacement method of the ASME Code. The insert plates and sleeves permit development of the hoop tensile yield stresses predicted as the limiting capacity in Subsection 3.8.2.4.1. Capacities of the equipment hatch covers are discussed in Subsection 3.8.2.4.2.3 and of the personnel airlocks in Subsection 3.8.2.4.2.4.

Mechanical penetrations welded directly to the containment vessel are generally piping systems with design pressures greater than that of the containment vessel. Thicknesses of the flued head or end plate are established based on piping support loads or stiffness requirements. The capacities of these penetrations are greater than the capacity of the containment vessel cylinder.

Mechanical penetrations for the large-diameter high-energy lines, such as the main steam and feedwater piping, include expansion bellows. The piping and flued head have large pressure capability. The response of expansion bellows to severe pressure and deformations is described in NUREG/CR-5561 (Reference 4). The bellows can withstand large pressure loading but may tear once the containment vessel deflection becomes large. Testing reported in NUREG/CR-6154 (Reference 26) has shown that the bellows remain leaktight even when subjected to large deflections sufficient to fully compress the bellows. Such large deflections do not occur as long as the containment vessel remains elastic. As described in Subsection 3.8.2.4.2.6, the radial deflection of the shell increases substantially once the containment cylinder yields. The resulting deflections are assumed to cause loss of containment function. The containment penetration bellows are designed for a pressure of 90 psig at design temperature within Service Level C limits, concurrent with the relative displacements imposed on the bellows when the containment vessel is pressurized to these magnitudes.

Electrical penetrations have a pressure boundary consisting of the sleeve and an end plate containing a series of modules. The electrical pressure boundary is designed and built to the requirements of the ASME Code, Section III, Class MC, Subsection NE. The pressure capacity of these elements is large and is greater than the capacity of the containment vessel cylinder at temperatures up to the containment design temperature. Electrical penetration assemblies are also designed to satisfy ASME Service Level C stress limits under a pressure of 90 psig at design temperature. Tests at pressures and temperatures representative of severe accident conditions are described in NUREG/CR-5334 (Reference 5), where typical nuclear industry penetrations were irradiated, aged, then tested. One design was tested to 135 psia at 700°F. Other electrical penetration assemblies were tested to 75 psia at 400°F and 155 psia at 361°F. These tests showed that the electrical penetration assemblies withstand severe accident conditions. The electrical penetration assemblies are qualified for the containment design basis event conditions as described in Appendix 3D. The assemblies are similar to one of those tested by Sandia as reported in NUREG/CR-5334 (Reference 5). The ultimate pressure capacity of the electrical penetration assemblies is primarily determined by the temperature. The maximum temperature of the containment vessel below the operating deck during a severe accident is below the temperature at which the assemblies from the three suppliers in the Sandia tests were tested.

3.8.2.4.2.6 Material Properties

The containment vessel is designed using SA738, Grade B material. This has a specified minimum yield of 60 ksi and ultimate of 85 ksi. Test data for materials having similar chemical properties were reviewed. In a sample of 122 tests for thicknesses equaling or exceeding 1.50 inches and less than 1.75 inches, the actual yield had a mean value of 69.1 ksi with a standard deviation of 3.3 ksi. Thus, the actual yield is expected to be about 15 percent higher than the minimum yield. Membrane yield of the cylinder is predicted to occur at an internal pressure of 178 psig.

A stress-strain curve for material with chemistry similar to SA738, Grade B, indicated constant yield stress of 81.3 ksi from a strain of 0.002 to 0.006 followed by strain-hardening up to a maximum stress of 94.5 ksi at a strain of 0.079. The first portion of the strain-hardening is nearly linear, with a stress of 90 ksi at a strain of 4 percent. This strain occurs at a stress 10 percent above yield. Thus, a pressure load 10 percent higher than that corresponding to yield of the shell would result in 4 percent strain and a 31-inch radial deflection of the containment cylinder. Such a deflection is expected to cause major distress for penetrations, the air flow path, and local areas where other structures are close to the containment vessel. Loss of function is therefore assumed for the containment once gross yield of the containment cylinder occurs.

3.8.2.4.2.7 Effect of Temperature

The evaluations described in the preceding subsections are based on an ambient temperature of 100°F. Nonmetallic items, such as gaskets, are qualified to function at the design temperature. The capacity of steel elements is reduced in proportion to the reduction due to temperature in yield stress, ultimate stress, or elastic modulus. The cylinder is governed by yield stress, and elastic buckling of the hatch covers is governed by the elastic modulus. The reduction in capacity is estimated using the tables given for material properties in the ASME Code. At 400°F, the yield stress is reduced by 17 percent and the pressure capacity corresponding to gross yield is reduced from 155 to 129 psig.

3.8.2.4.2.8 Summary of Containment Pressure Capacity

The ultimate pressure capacity for containment function is expected to be associated with leakage caused by excessive radial deflection of the containment cylindrical shell. This radial deflection causes distress to the mechanical penetrations, and leakage would be expected at the expansion bellows for the main steam and feedwater piping. There is high confidence that this failure would not occur before stresses in the shell reach the minimum specified material yield. This is calculated to occur at a pressure of 155 psig at ambient temperature and 129 psig at 400°F. Failure would be more likely to occur at a pressure about 15 percent higher based on expected actual material properties.

The deterministic severe accident pressure that can be accommodated according to the ASME Service Level C stress intensity limits and using a factor of safety of 1.67 for buckling of the top head is determined by the capacity of the 16-foot-diameter equipment hatch cover and the ellipsoidal head. The maximum capacity of the hatch cover, calculated according to ASME paragraph NE-3222, Service Level C, is 84 psig at an ambient temperature of 100°F and 81 psig at 300°F. When calculated in accordance with ASME Code, Case N-284, Service Level C, the maximum capacity is 126 psig at an ambient temperature of 100°F and 121 psig at 300°F. The maximum capacity of the ellipsoidal head is 104 psig at 100°F and 91 psig at 300°F.

The maximum pressure that can be accommodated according to the ASME Service Level C stress intensity limits, excluding evaluation of instability, is determined by yield of the cylinder and is 135 psig at an ambient temperature of 100°F and 117 psig at 300°F. This limit is used in the evaluations required by 10 CFR 50.44.

3.8.2.5 Structural Criteria

The containment vessel is designed, fabricated, installed, and tested according to the ASME Code, Section III, Subsection NE, and will receive a code stamp.

Stress intensity limits are according to ASME Code, Section III, Paragraph NE-3221 and Table NE-3221-1. [*Critical buckling stresses are checked according to the provisions of ASME Code, Section III, Paragraph NE-3222, or ASME Code Case N-284.*]*

3.8.2.6 Materials, Quality Control, and Special Construction Techniques

Materials for the containment vessel, including the equipment hatches, personnel locks, penetrations, attachments, and appurtenances meet the requirements of NE-2000 of the ASME Code. [*The basic containment material is SA-738, Grade B, plate. The procurement specification for the SA-738, Grade B, plate includes SA-20 supplemental requirements S1, Vacuum Treatment and S20, Maximum Carbon Equivalent for Weldability.*]* This material has been selected to satisfy the lowest service metal temperature requirement of -18.5°F. This temperature is established by analysis for the portion of the vessel exposed to the environment when the minimum ambient air temperature is -40°F. Impact test requirements are as specified in NE-2000.

[*The material of construction for the insert plates and fabricated nozzle necks of penetrations is SA-738 Grade B. The material of construction for forged nozzle neck forgings is SA-350, LF2, Class 1 for penetrations greater than 2 inches nominal diameter and less than 24 inches inside diameter, and for the containment air filtration system penetration nozzle necks. The maximum carbon equivalent for the SA-350, LF2, Class 1 used for penetrations is 0.52 percent. A vacuum refining process is required for SA-350, LF2, Class 1. The SA-350, LF2, Class 1 material is used in portions of the containment vessel that are below the external stiffener.*]* These portions of the containment vessel are insulated from the cold outside air temperature. Insulation is provided around the upper equipment hatch and personnel airlock, including the insert plates, in order to insulate the penetrations from the ambient air temperature in the upper annulus.

The containment vessel is coated with an inorganic zinc coating, except for those portions fully embedded in concrete. The inside of the vessel below the operating floor also has an epoxy top coat. Below elevation 100′ the vessel is fully embedded in concrete with the exception of the few penetrations at low elevations (see Figure 3.8.2-4, sheet 3, for typical details). Embedding the steel vessel in concrete protects the steel from corrosion.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

The AP1000 configuration is shown in the general arrangement figures in Section 1.2 and in Figure 3.8.2-1. The exterior of the vessel is embedded at elevation 100′ and concrete is placed against the inside of the vessel up to the maintenance floor at elevation 107′-2″. Above this elevation the inside and outside of the containment vessel are accessible for inspection of the coating. The vessel is coated with an inorganic zinc primer to a level just below the concrete.

Seals are provided at the surface of the concrete inside and outside the vessel so that moisture is not trapped next to the steel vessel just below the top of the concrete. The seal on the inside accommodates radial growth of the vessel due to pressurization and heatup.

The plate thickness for the first course (elevation 104′-1.5″ to minimum 109′-0″) of the cylinder is 1.875 inches, which is 1/8-inch thicker than the rest of the vessel. This provides margin in the event there would be any corrosion in the transition region despite the coatings and seals described previously. Equivalent margin is available for the 1.625-inch-thick bottom head in the transition region (elevation 100′ to 104′-1.5″). The plate thickness for the head is a constant thickness and is established by the stresses in the knuckle. As a result, the pressure stresses in the transition zone are well below the allowable stress, providing margin in the event of corrosion in this region.

The quality control program involving welding procedures, erection tolerances, and nondestructive examination of shop- and field-fabricated welds conforms with Subsections NE-4000 and NE-5000 of the ASME Code. The containment vessel is designed to permit its construction using large subassemblies. These subassemblies consist of the two heads and three or more ring sections. These are assembled in an area near the final location, using plates fabricated in a shop facility.

3.8.2.7 Testing and In-Service Inspection Requirements

Testing of the containment vessel and the pipe assemblies forming the pressure boundary within the containment vessel will be according to the provisions of NE-6000 and NC-6000, respectively.

Subsection 6.2.5 describes leak-rate testing of the containment system including the containment vessel.

In-service inspection of the containment vessel will be performed. See Section 6.6 for information on inservice inspection for the containment vessel and penetrations.

3.8.3 Concrete and Steel Internal Structures of Steel Containment

3.8.3.1 Description of the Containment Internal Structures

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

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

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

Walls and floors are concrete filled steel plate structural modules. The walls are supported on the mass concrete containment internal structures basemat with the steel surface plate extending down to the concrete floor on each side of the wall. The steel surface plates of the structural modules provide reinforcement in the concrete. The structural modules are anchored to the base concrete by mechanical connections welded to the steel plate as shown in Figure 3.8.3-8, Sheet 2. Figure 3.8.3-1 shows the location of the structural modules. Figures 3.8.3-2 and 3.8.3-15 show the typical structural configuration of the wall modules. Key structural elements of the module design are identified as Tier 2* information in the text and figures of this section. See DCD Introduction, Section 3.5 for a discussion of Tier 2* information. [*The information in Figure 3.8.3-2 that is Tier 2* is the minimum size of the angles and channels used to fabricate the modules. The information in Figure 3.8.3-15 that is Tier 2* is the design spacing between the face plates for the 4-foot-thick refueling canal wall in the containment internal structures and the design spacing between the trusses used to fabricate the modules in locations away from openings or penetrations in the wall. Local variation in the design of the trusses and spacing of the trusses and shear studs may be required to address internal obstructions and accessibility for fabrication and inspection. The obstructions include features such as leak chases; internal structures such as reinforcements, embedments, and backup structures; and internal conduit and piping.*]* See Subsection 3.8.3.5.3.6 for criteria for design of the shear stud size and spacing.

A representative floor module is shown in Figure 3.8.3-3 and also in Figure 3.8.3-16 combined with the liner module. These structural modules are structural elements built up with welded steel structural shapes and plates. The details shown in Figure 3.8.3-3 are a representative design for a specific portion of the operating deck floor in containment. The size, material, and configuration of the structural shapes and plates, the amount and arrangement of the reinforcement, and the type and size of the welds may vary in the final design of floor modules in this and other locations. Concrete is used where required for shielding, but reinforcing steel is not normally used.

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

3.8.3.1.1 Reactor Coolant Loop Supports

3.8.3.1.1.1 Reactor Vessel Support System

The reactor vessel is supported by four supports located under the cold legs, which are spaced 90 degrees apart in the primary shield wall. The supports are designed to provide for radial thermal growth of the reactor coolant system, including the reactor vessel, but they prevent the vessel from lateral and torsional movement. The ends of the supports are bolted to steel embedments in the concrete wall surrounding the reactor vessel. Each support connects to the embedments with eight 2-3/8-inch-diameter dowel studs. In addition, the center portion of the support is attached to the concrete with eight threaded anchor bolts embedded in the primary shield wall concrete. The reactor vessel supports loads are carried by the dowel studs to embedded steel structures and the anchor bolts to the concrete. Figure 3.8.3-4 shows the reactor vessel supports.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.3.1.1.2 Steam Generator Support System

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

The lower steam generator horizontal support is located at the top of the vertical column. It consists of a tension/compression link bar and extension bar oriented approximately perpendicular to the hot leg. The link bar and extension bar is pinned at both the wall bracket and the vertical column to permit movement of the generator during plant heatup and cooldown.

The upper steam generator horizontal support in the direction of the hot leg is located on the upper shell just above the transition cone. It consists of two large hydraulic snubbers oriented parallel with the hot leg centerline. One snubber is mounted on each side of the generator on top of the steam generator compartment wall. The hydraulic snubbers are valved to permit steam generator movement for thermal transition conditions, and to "lock-up" and act as rigid struts under dynamic loads.

The upper steam generator horizontal support in the direction normal to the hot leg is located on the lower shell just below the transition cone. It consists of two rigid tension/compression struts oriented perpendicular to the hot leg. The two rigid struts are mounted on the steam generator compartment wall at the elevation of the operating deck. The steam generator loads are transferred to the struts and snubbers through trunnions on the generator shell. Figure 3.8.3-5 shows the steam generator supports.

The steam generator supports are anchored using anchor bolts or steel weldments embedded in the concrete, designed in accordance with Appendix B of ACI 349. The lower portion of the column pedestal, embedded in the concrete, as shown on sheet 1 of Figure 3.8.3-5, transfers the vertical load into the reinforced concrete basemat. The lower and intermediate horizontal supports are located so that the loads are transferred into the plane of the adjacent floor. The upper supports are located so that the loads are transferred into the plane of the steam generator compartment walls.

3.8.3.1.1.3 Reactor Coolant Pump Support System

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

3.8.3.1.1.4 Pressurizer Support System

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

3.8.3.1.2 Containment Internal Structures Basemat

The containment internal structures basemat is the reinforced concrete structure filling the bottom head of the containment vessel. It extends from the bottom of the containment vessel head at elevation 66′-6″ up to the bottom of the structural modules that start between elevations 71′-6″ and 103′-0″. The basemat includes rooms as shown on Figure 1.2-5. The primary shield wall and reactor cavity extend from elevation 71′-6″ to elevation 107′-2″. They provide support for the reactor vessel and portions of the secondary shield walls and refueling cavity walls. The general arrangement drawings in Section 1.2 show the location and configuration of the primary shield wall and reactor cavity. The walls of the primary shield, the steam generator compartment and the CVS room are structural modules as shown in Figure 3.8.3-1. The rest of the basemat is reinforced concrete.

3.8.3.1.3 Structural Wall Modules

Structural wall modules are used for the primary shield wall around the reactor vessel, the wall between the vertical access and the CVS room, secondary shield walls around the steam generators and pressurizer, for the east side of the in-containment refueling water storage tank, and for the refueling cavity. The general arrangement drawings in Section 1.2 show the location and configuration. Locations of the structural modules are shown in Figure 3.8.3-1. Isometric views of the structural modules are shown in Figure 3.8.3-14. The figure shows the use of modules, overall configuration, and general design approach for incorporation of structural modules into the structural design. Alternatives to the design details shown in Figure 3.8.3-14, including the size, location, and type of structural shapes attached to the faceplates and embedded in the concrete, may be used. The embedded lower portions of the modules in the concrete, attachments, and penetrations may also be added or differ from that shown in the figure. The secondary shield walls are a series of walls that, together with the refueling cavity wall, enclose the steam generators. Each of the two secondary shield wall compartments provides support and houses a steam generator and reactor coolant loop piping. The in-containment refueling water storage tank is approximately 30 feet high. The floor elevation of this tank is 103′-0″. The tank extends up to about elevation 133′-3″, directly below the operating deck. On the west side, along the containment vessel wall, the tank wall consists of a stainless steel plate stiffened with structural steel sections in the vertical direction and angles in the horizontal direction. Structural steel modules, filled with concrete and forming, in part, the refueling cavity, steam generator compartment, and pressurizer compartment walls, compose the east wall. The refueling cavity has two floor elevations. The area around the reactor vessel flange is at elevation 107′-2″. The lower level is at elevation 98′-1″. The upper and lower reactor internals storage is at the lower elevation, as is the fuel transfer tube. The center line of the fuel transfer tube is at elevation 100′-5.75″.

Structural wall modules consist of steel faceplates connected by steel trusses. The primary purpose of the trusses is to stiffen and hold together the faceplates during handling, erection, and concrete placement. The nominal spacing of the trusses is 30 inches. The nominal thickness of the steel faceplates is 0.5 inch. [*In the CA01 and CA05 structural wall modules, the faceplate thickness is greater than the nominal thickness where required to support local demand. For areas subject to these high out-of-plane loads, the faceplate thickness may be increased up to a thickness of 1.5 inches. To support loading associated with the steam generator lateral supports, the faceplate thickness may be increased up to a thickness of 3.0 inches.*]* At corner locations, the trusses are replaced with diaphragms. These diaphragms have similar purpose as the trusses and are used to fabricate the modules in the corners. The detail design of the trusses and the design of the diaphragms shown in Figure 3.8.3-8, Sheet 1 are representative. Details such as the location, type, and size of welds and length and end configuration of channels used to assemble the internal trusses may vary in the design implemented from that shown in the figure. The size, shape, and location of welds, structural shapes, and other elements used to fabricate and stiffen the diaphragms and connect them to the faceplates and the location, size, and shape of the access opening may vary from that shown in the figure. Steel plates, structural shapes, reinforcement bars, or tie bars are installed between the structural wall module faceplates and embedded in the concrete as additional structural elements that support out-of-plane loads and structural integrity. This reinforcement is designed to the applicable requirements of ACI 349 and AISC N690. Each module has different characteristics that require the detailed design to be specific for that module and loading and

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

interferences within that module.Shear studs are welded to the inside faces of the steel faceplates. Face plates are welded to adjacent plates with full penetration welds so that the weld is at least as strong as the plate. The full penetration welds are identified in Figure 3.8.3-8, Sheet 1 with the weld symbol that includes the notation CJP for complete joint penetration. Plates on each face of the wall module extend down to the elevation of the adjacent floor. Where the floors in the rooms on each side of the wall module are at different elevations, one of the plates at the bottom of the module extends further down than the other. This single-sided configuration is designated on Figure 3.8.3-1 as “CA Structure Module with Single Surface Plate” for lower elevations in containment. A typical example with the single-sided configuration at the bottom of the module is shown in Figure 3.8.3-8. Some modules or portions of modules do not have the single-sided configuration at the bottom of the module. The module functions as a wall above the upper floor level (elevation 103′-0″ in Figure 3.8.3-8). The single plate below this elevation is designed to transfer the reactions at the base of the wall into the basemat. This plate also acts as face reinforcement for the basemat. Basemat reinforcement dowels are provided at the bottom of the single plate as shown in Figure 3.8.3-8.

[*Shear studs, structural shapes, or horizontally oriented reinforcement bars mechanically connected or welded to the module faceplates may be used to anchor the CA01 and CA05 structural modules to the base concrete. Shear lugs designed in accordance with ACI 349 may also be used to transfer loads at the connection. Where horizontal bars are attached perpendicular to the faceplate, they function like shear studs to provide for faceplate development. Where additional reinforcement is required for detailing base concrete at module walls with offsets in faceplate elevation, the reinforcement is developed in accordance with ACI 349. Where horizontal bars are attached to the edge of a faceplate at an elevation offset, the bars are designed for a combination of applicable demand and developed in accordance with ACI 349.*]*

[*The information in Figure 3.8.3-8, Sheet 1 that is considered to be Tier 2* information is the design spacing of the faceplates, trusses, channels in the trusses and the minimum size and design spacing of the headed studs for the modular wall in the containment internal structure in locations away from openings or penetrations in the walls. Local variation in the design of the trusses and spacing of the trusses and shear studs may be required to address internal obstructions and accessibility for fabrication and inspection. As shown in Figure 3.8.3-1, Sheets 2 and 4, some CA01 and CA05 wall thicknesses are different than the wall thicknesses listed in Figure 3.8.3-8. Wall thickness increase due to local faceplate thickness increase is below the level of detail depicted in Figure 3.8.3-1. The module wall thicknesses are determined based on radiation protection considerations and are designed to the applicable requirements of AISC N690 and ACI 349. The use of full penetration welds to connect the faceplates of the modules is also considered to be Tier 2* information. The information in Figure 3.8.3-8, Sheet 2 that is considered to be Tier 2* information is the use of mechanical connectors and the development length requirement for the mechanical connectors.*]* The detail design of the mechanical connectors is governed by AISC N690 and ACI 349, and the representative design shown is not considered to be Tier 2* information. The material, size, and configuration of plates, size and type of welds, method of attaching the connectors to the modules, and size of deformed bars anchoring the embedments into the concrete may be different for each module and may vary from that shown in the figure provided that the embedment design satisfies the AISC N690 and ACI 349 codes. Sheet 3 of Figure 3.8.3-8 shows a wall of the IRWST that is a steel structure anchored in concrete and is not a concrete filled module. [*The information in Figure 3.8.3-8, Sheet 3 that is considered to be Tier 2* information is the plate thickness for the IRWST wall, the use, spacing, and size of angles and tees to stiffen the wall, the number, size, and use of studs provided to anchor the module, and the number, size, vertical spacing, and development length of the deformed bars provided to connect the module to the mass concrete. The design implemented in fabrication and construction drawings and instructions will have the design shown, an equal design, or a better design for the key structural elements.*]*

The structural wall modules are anchored to the concrete base by reinforcing steel dowels or other types of connections mechanically connected or welded to the modules and embedded in the

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

reinforced concrete below or adjacent to the module. [*The reinforcing steel used to anchor the modules to the concrete may be oriented horizontally or vertically and has a development that satisfies the requirements of ACI 349.*]* After erection, concrete is placed between the faceplates. Typical design details of the structural modules are shown in Figures 3.8.3-2 and 3.8.3-8. Shear-friction reinforcement may be included in construction joints at the transition between the base concrete and concrete in the modules and is not part of the mechanical connection between the module and base concrete described above. The shear-friction reinforcement at the construction joints is spaced and sized in accordance with ACI 349 requirements and is not connected to the module faceplates.

Representative design details of the connections of floor modules to structural wall modules are shown in Figure 3.8.3-17. These connections connect the floor modules that make up the maintenance floor at elevation 107'-2" and the operating deck at elevation 135'-3". Similar connection designs are used within structural modules to connect floors to walls. The design details for the floor modules, including the size and spacing of the reinforcement, the use of shear studs, and the sizes of the structural shapes, are provided for background information and vary based on loading conditions and geometry. The seat angles, beam seats, shear plates, and other design elements supporting the floor modules and connecting the floors to the wall modules are sized to satisfy AISC N690 criteria and requirements. The clip angles and shear plates shown in Figure 3.8.3-17 and other design elements are used as required at beam locations for the connection. The module faceplates in the area of the connection may be thicker than 0.5 inch, up to 1.5-inch thick, in some locations. The backup structure shown in the module wall is a representative design, and the details for the design implemented include variation of the size and thickness of the plates used in the backup structure. The designs of the backup structures satisfy AISC N690 criteria and requirements. As an alternative, locally thicker faceplates or other design elements embedded in the wall and connected to the faceplate may be used to provide sufficient strength in the connection. Where the reinforcement is included as part of the connection design, the size, spacing, and length of the reinforcement satisfies ACI 349 requirements. The standard hooks in the wall module and the reinforcement bar connectors on the faceplates satisfy ACI 349 requirements.

3.8.3.1.4 Structural Floor Modules

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

3.8.3.1.5 Internal Steel Framing

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.3.2 Applicable Codes, Standards, and Specifications

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

[• *American Concrete Institute (ACI), Code Requirements for Nuclear Safety Related Concrete Structures, ACI 349-01*]* (refer to Subsection 3.8.4.5 for supplemental requirements)

- American Concrete Institute (ACI), ACI Detailing Manual
- American Concrete Institute (ACI), Standard Specifications for Tolerances for Concrete Construction and Materials, ACI-117
- American Concrete Institute (ACI), Guide to Formwork for Concrete, ACI-347

[• *American Institute of Steel Construction (AISC), Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690-1994*]* (refer to Subsection 3.8.4.5 for supplemental requirements)

[• *American Welding Society (AWS), Structural Welding Code - Steel, AWS D1.1-2000*

AWS D1.1-2000 is an acceptable code for use in lieu of AWS D1.1-1992 for the design, qualification, fabrication, and inspection for AISC N690 applications.

The use of AWS D1.1-2000 to replace and supplement provisions in AISC N690-1994 includes the provisions noted below:

– *The provisions in Section 2.3.4.1 of AWS D1.1-2000 are used to size complete joint penetration groove welds for tubular members.*

– *ASTM A992 is an alternative to ASTM A36 steel for structural steel shapes. Where ASTM A992 steel is used, the provisions for matching weld metal identified in AWS D1.1-2000, Section 3.3 and Table 3.1 are used to supplement the information in AISC N690 Section Q1.4.4.*

– *Where directionality is applied to increase allowable weld stress for fillet welds identified in AISC N690, Section Q1.5.3 and Table Q1.5.3, the provisions in Sections 2.14.4 and 2.14.5 of AWS D1.1-2000 shall be used.*

For fillet weld groups concentrically loaded in the plane of the axis of the weld and consisting of elements oriented both longitudinally and transversely to the direction of applied load, the deformation capability of the elements shall be considered. As a supplement to the provisions in AWS D1.1-2000, the strength of the weld group is limited to the larger of the two following evaluations:

1. *The combined strength of the individual weld elements using 1.0 times the strength of the longitudinally oriented element and 1.0 times the strength of the transversely oriented element*

2. *The combined strength of the individual weld elements using 0.825 times the strength of the longitudinally oriented element and 1.5 times the strength of the transversely oriented element*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

The provisions for directionality are not applied to fillet welds of tubular member T, Y, and K connections described in Table 2.5 of AWS D1.1-2000.

The provisions in AWS D1.1-2000 for the application of directionality to linear fillet welds are supplemented by extending the provisions to weld configurations not specified in AWS D1.1-2000 as noted below:

– *Where directionality is considered for concentric out-of-plane loading for linear fillet weld groups, including lines and rectangular configurations, the directionality provisions in AWS D1.1-2000 are applied.*

– *Where directionality is considered for concentric out-of-plane loading for circular-shaped fillet weld configurations, the directionality provisions in AWS D1.1-2000 are applied. The use of directionality provisions in AWS D1.1-2000 for circular-shaped fillet weld configurations is limited to tension loading concentrically applied out-of-plane to the axis of the weld.*]*

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

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

3.8.3.3 Loads and Load Combinations

The loads and load combinations used in the analysis are considered to be part of the method of evaluation. The loads and load combinations for the containment internal structures are the same as for other Category I structures described in Subsection 3.8.4.3 and the associated tables, except for the following modifications:

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

3.8.3.3.1 Passive Core Cooling System Loads

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

The passive core cooling system and the automatic depressurization system (ADS) are described in Section 6.3. The automatic depressurization system is in part composed of two spargers that are submerged in the in-containment refueling water storage tank. The spargers provide a controlled distribution of steam flow to prevent imposing excessive dynamic loads on the tank structures. Capped vent pipes are installed in the roof of the tank on the side near the containment wall.

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These caps prevent debris from entering the tank from the containment operating deck, but they open under slight pressurization of the in-containment refueling water storage tank. This provides a path to vent steam released by the spargers. An overflow is provided from the in-containment refueling water storage tank to the refueling cavity to accommodate volume and mass increases during automatic depressurization system operation. Two sets of loads representing bounding operational or inadvertent transients are considered in the design of the in-containment refueling water storage tank.

- ADS_1 – This automatic depressurization system load is associated with blowdown of the primary system through the spargers when the water in the in-containment refueling water storage tank is cold and the tank is at ambient pressure. Dynamic loads on the in-containment refueling water storage tank due to automatic depressurization system operation are determined using the results from the automatic depressurization system hydraulic test as described in Subsection 3.8.3.4.2. The hydrodynamic analyses described in Subsection 3.8.3.4.2 show that member forces in the walls of the in-containment refueling water storage tank are bounded by a case with a uniform pressure of 5 psi applied to the walls. The in-containment refueling water storage tank is designed for a uniform pressure of 5 psi applied to the walls. This pressure is taken as both positive and negative due to the oscillatory nature of the hydrodynamic loads. This automatic depressurization system transient is of short duration such that the concrete walls do not heat up significantly. It is combined with ambient thermal conditions. Long-term heating of the tank is bounded by the design for the ADS_2 load.

- ADS_2 – This automatic depressurization system transient considers heatup of the water in the in-containment refueling water storage tank. This may be due to prolonged operation of the passive residual heat removal heat exchanger or due to an automatic depressurization system discharge. For structural design, an extreme transient is defined starting at 50°F since this maximizes the temperature gradient across the concrete-filled structural module walls. Prolonged operation of the passive residual heat removal heat exchanger raises the water temperature from an ambient temperature of 50°F to saturation in about 4 hours, increasing to about 260°F within about 10 hours. Steaming to the containment atmosphere initiates once the water reaches its saturation temperature. The temperature transient is shown in Figure 3.8.3-7. Blowdown of the primary system through the spargers may occur during this transient and occurs prior to 24 hours after the initiation of the event. Since the flow through the sparger cannot fully condense in the saturated conditions, the pressure increases in the in-containment refueling water storage tank and steam is vented through the in-containment refueling water storage tank roof. The in-containment refueling water storage tank is designed for an equivalent static internal pressure of 5 psi in addition to the hydrostatic pressure occurring at any time up to 24 hours after the initiation of the event.

The ADS_1 and ADS_2 loads are considered as live loads. The dynamic ADS_1 load is combined with the safe shutdown earthquake by the square root sum of the squares (SRSS). ADS_2 is an equivalent static pressure which is included algebraically with other normal loads and then combined with plus/minus SSE loads.

3.8.3.3.2 Concrete Placement Loads

The steel faceplates of the structural wall modules, designed for the hydrostatic pressure of the concrete, act as concrete forms. The concrete placement loads are 1050 pounds per square foot determined in accordance with ACI-347. The bending stress in the faceplate due to this hydrostatic pressure of the concrete is approximately 13 ksi, based on the assumption of a continuous faceplate, or 20 ksi based on the assumption of simple spans. The minimum yield strength of material for the faceplates is 36 ksi. The stress is well below the allowable, since the plate is designed to limit the out-of-plane deflection. After the concrete has gained strength, these stresses remain in the steel;

however, since the average residual stress is zero and since the concrete no longer requires hydrostatic support, the ultimate strength of the composite section is not affected, and the full steel plate is available to carry other loads as described below.

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

3.8.3.4 Analysis Procedures

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

indicate that concrete and steel composites similar to the structural modules have significant advantages over reinforced concrete elements of equivalent thickness and reinforcement ratios:

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

Methods of analysis for the structural modules are similar to the methods used for reinforced concrete. Table 3.8.3-2 summarizes the finite element analyses of the containment internal structures and identifies the purpose of each analysis and the stiffness assumptions for the concrete filled steel modules. For static loads the analyses use the monolithic (uncracked) stiffness of each concrete element. The elastic modulus is taken as 0.80 times the value calculated based on the ACI Code. This reduced elastic modulus considers a small degree of cracking as described in the seismic analyses in Subsection 3.7.2.3. For thermal and dynamic loads the analyses consider the extent of concrete cracking as described in later subsections.

Stiffnesses are established based on analyses of the behavior and review of the test data related to concrete-filled structural modules. The stiffnesses directly affect the member forces resulting from restraint of thermal growth. The in-plane shear stiffness of the module influences the fundamental horizontal natural frequencies of the containment internal structures in the nuclear island seismic analyses described in Subsection 3.7.2. The out-of-plane flexural stiffness of the module influences the local wall frequencies in the seismic and hydrodynamic analyses of the in-containment refueling water storage tank. Member forces are evaluated against the strength of the section calculated as a reinforced concrete section with zero strength assigned to the concrete in tension.

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

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

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

- Case 2 considers the full thickness of the wall as uncracked concrete. This stiffness value is shown for comparison purposes. It is applicable for loads that do not result in significant cracking of the concrete and is the basis for the stiffness of the reinforced concrete walls in the nuclear island seismic analyses (prior to the reduction in concrete stiffness by a factor of 0.8). This stiffness was used in the harmonic analyses of the internal structures described in Subsection 3.8.3.4.2.2.

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

3.8.3.4.1 Seismic Analyses

3.8.3.4.1.1 Finite Element Model

The structural modules are simulated within the finite element model using 3D shell elements. Equivalent shell element thickness and modulus of elasticity of the structural modules are computed as shown below. The shell element properties are computed using the combined gross concrete section and the transformed steel faceplates of the structural modules. This representation models the composite behavior of the steel and concrete. The equivalent modulus of concrete, E_m, is reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5 in Subsection 3.7.6). See Section 3.7 and Appendix 3G for further discussion of the containment internal structures finite element model.

- Axial and Shear Stiffnesses of module:

$$\sum EA = E_c(Lt + 2(n-1)L\,t_s)$$

- Bending Stiffness of module:

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

where:

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

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

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

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

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

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

The seismic analyses of the in-containment refueling water storage tank address the local response of the walls and water and are performed to verify the structural design of the tank. The local analyses performed uses the cracked section stiffness values based on composite behavior with zero stiffness for the concrete in tension (Case 3 of Table 3.8.3-1). The local analyses use the finite element model described in Subsection 3.8.3.4.2.2. Response spectrum analyses are performed using the floor response spectra at the base of the tank.

3.8.3.4.1.3 Damping of Structural Modules

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

3.8.3.4.2 Hydrodynamic Analyses

This subsection describes the hydrodynamic analyses performed for the AP600 which demonstrated that design of the walls of the in-containment refueling water storage tank for 5 psi as described in Subsection 3.8.3.3.1 would bound the loads from the time history transient analysis. The analyses were performed using the AP600 test results. The peak values from these tests are also applicable to the AP1000 since they occur at the beginning of the transient, and the automatic depressurization system and the initial conditions are the same for the two plant designs (Reference 52). The structural configuration of the tank is identical. The minor differences in the height of the steam generator and pressurizer compartment walls and in the mass of the steam generators and pressurizer will have only a minor effect on the significant structural frequencies. Since the time histories applied in the AP600 analyses cover a broad range of frequencies the response of the AP1000 tank boundary will be similar to that of the AP600. The 5 psi pressure design basis for the tank boundary is therefore also applicable to the AP1000.

Hydrodynamic analyses were performed for the AP600 for automatic depressurization system discharge into the in-containment refueling water storage tank. This discharge is designated as ADS_1 in the load description of Subsection 3.8.3.3.1 and results in higher hydrodynamic loading than the ADS discharge into a hot tank in ADS_2. The first three stages of the automatic depressurization system valves discharge into the tank through spargers under water, producing hydrodynamic loads on the tank walls and equipment. Hydrodynamic loads, measured in hydraulic tests of the automatic depressurization system sparger in a test tank, are evaluated using the source load approach (Reference 34). Analyses of the tests define source pressure loads that are then used in analyses of the in-containment refueling water storage tank to give the dynamic responses of the containment internal structures. The basic analysis approach consists of the following steps:

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

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

3.8.3.4.2.1 Sparger Source Term Evaluation

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

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

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

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

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

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

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

- The hydrodynamic loads on the vessel head support columns and ADS sparger piping located in the IRWST are developed from the forcing functions using the methodology documented in Reference 51.

3.8.3.4.2.2 In-Containment Refueling Water Storage Tank Analyses

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

The 3D ANSYS finite element model includes the in-containment refueling water storage tank boundary, the water within the in-containment refueling water storage tank, the adjacent structural walls of the containment internal structures, and the operating floor. The model of the in-containment refueling water storage tank, shown in Figures 3.8.3-10 (sheet 2), 3.8.3-11, and 3.8.3-12, represents the outer steel structures, the inner concrete walls, and the water. The model of the adjacent structural walls and floors is shown in Figure 3.8.3-10 (sheet 1). The flexible steel outer wall is represented using beam and shell elements; isotropic plate elements are used to represent the inner structural module walls. The water is modelled as a compressible fluid to provide an acoustic medium to transmit the source pressure. The model has two bubble boundaries representing the spargers. Pressure loads are applied to the solid element faces adjacent to the air bubbles. The forcing functions at the sparger locations are conservatively assumed to be in phase. Rayleigh damping of 5 percent is used for the concrete-filled structural modules and fluid damping is neglected. All degrees of freedom were retained in the step-by-step direct integration solution procedure for the in-containment refueling water storage tank boundary and the water. Degrees of freedom in the adjacent walls and floor were condensed by Guyon reduction.

Significant structural frequencies of the AP600 containment internal structures were analyzed using the harmonic response option with the ANSYS model of the in-containment refueling water storage tank and containment internal structures. A harmonic unit pressure is applied at the surface of the spherical bubble representing the automatic depressurization system spargers. Material properties for the concrete elements are based on the uncracked gross concrete section (Case 2 of Table 3.8.3-1). The results of these harmonic response analyses show the response deflection as a function of input frequency at nodes in the containment internal structures. The harmonic response analyses show that the largest responses are close to the wetted boundary of the in-containment refueling water storage tank and that the significant frequencies are from 18 to 50 hertz.

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

The analyses of the AP600 in-containment refueling water storage tank give wall pressures, displacements, accelerations, hydrodynamic floor response spectra, and member forces due to the automatic depressurization system discharge pressure forcing functions. Consideration of pressure wave transmission and fluid-structure interaction shows a significant wall pressure attenuation with distance from the sparger region and with increasing wall flexibilities, relative to the measured sparger pressure forcing function. The member stresses are evaluated against the allowable stresses specified in Subsection 3.8.3.5 for seismic Category I structures, considering the hydrodynamic loads as live loads. The analyses show that the member forces in the walls of the in-containment refueling water storage tank are bounded by a case with a uniform pressure of 5 psi applied to the walls.

3.8.3.4.3 Thermal Analyses

The in-containment refueling water storage tank water and containment atmosphere are subject to temperature transients as described in Subsection 3.8.3.3.1. The temperature transients result in a nonlinear temperature distribution within the wall modules. Temperatures within the concrete wall are calculated in a unidimensional heat flow analysis. The average and equivalent linear gradients are applied to a finite element model of the containment internal structures at selected times during the transient. The effect of concrete cracking is considered in the stiffness properties for the concrete elements subjected to the thermal transient. The finite element model is that described in Subsection 3.8.3.4.2.2 except that the model of the water in the IRWST is not needed.

The structural modules are subject to a rapid temperature transient in the event of a loss-of-coolant accident (LOCA) or a main steam line break (MSLB). The structural modules were evaluated for these rapid temperature transients. The evaluation considered both carbon and stainless steel faceplates. The steel plate heats up most rapidly in the LOCA event with temperatures up to 270°F in the first few minutes for an ambient initial temperature of 50°F. The faceplate of the structural module will see differential temperatures of 220°F relative to the concrete. The concrete heats up more slowly and does not see a significant temperature increase during the early part of the transient. There is relative thermal growth of the faceplate, causing shear loads in the shear studs, and embedded angles of the structural steel trusses that are welded to the faceplate. The heatup of the surface plates during the initial portion of the LOCA transient results in cracking of the concrete walls except in regions where there is significant external restraint. The structural module maintains its integrity throughout the rapid thermal transient.

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

3.8.3.5 Design Procedures and Acceptance Criteria

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

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

[*The design and construction of anchors and embedments conform to the procedures and standards of Appendix B to ACI 349-01*]* and are in conformance with the regulatory positions of NRC Regulatory Guide 1.199, Revision 0. [*Alternative requirements to ACI 349, Appendix B apply to the anchoring of headed shear reinforcement for the basemat (see Subsection 3.8.4.4.1). Alternative requirements to ACI 349, Appendix B apply to the anchoring of headed reinforcement above the basemat (see Subsection 3.8.4.4.1).*]*

The secondary shield walls, in-containment refueling water storage tank, refueling cavity, and operating floor above the in-containment refueling water storage tank are designed using structural modules. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. Structural floor modules are designed as composite structures in accordance with AISC-N690. Methods of analysis used are based on accepted principles of structural mechanics and are consistent with the geometry and boundary conditions of the structures.

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

The determination of pressure and temperature loads due to pipe breaks is described in Subsections 3.6.1 and 6.2.1.2. Subcompartments inside containment containing high energy piping are designed for a pressurization load of 5 psi. The pipe tunnel in the CVS room (room 11209, Figure 1.2-6) is designed for a pressurization load of 7.5 psi. These subcompartment design pressures bound the pressurization effects due to postulated breaks in high energy pipe. The design for the effects of postulated pipe breaks is performed as described in Subsection 3.6.2. Determination of pressure loads resulting from actuation of the automatic depressurization system is described in Subsection 3.8.3.3.1.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

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

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

3.8.3.5.1 Reactor Coolant Loop Supports

3.8.3.5.1.1 Reactor Vessel Support System

The reactor vessel supports are described in Subsection 3.8.3.1.1.1. Figure 3.8.3-4 shows the embedments and anchor bolts for the reactor vessel supports. The embedded portions of the reactor vessel supports, which are outside the ASME jurisdictional boundary, are designed by elastic methods of analysis. Note the dowel studs connecting the support to the steel embedment are not embedded in the concrete and are within the ASME jurisdictional boundary. The embedded portions, including the anchor bolts, are analyzed and designed to resist the applicable loads and load combinations given in Subsection 3.8.4.3. The design is according to AISC-N690 and ACI-349. The design of the steel to resist the loads satisfies the AISC-N690 requirements. The design and evaluation to assure that the embedment transfers the loads to the concrete and is retained in the concrete satisfies the requirements of ACI 349.

3.8.3.5.1.2 Steam Generator Support System

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

3.8.3.5.1.3 Reactor Coolant Pump Support System

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

3.8.3.5.1.4 Pressurizer Support System

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

3.8.3.5.2 Containment Internal Structures Basemat

The containment internal structures basemat including the primary shield wall and reactor cavity are designed for dead, live, thermal, pressure, and safe shutdown earthquake loads. The structural modules are designed as described in Subsection 3.8.3.5.3.

The reinforced concrete forming the base of the containment internal structures is designed according to ACI 349.

3.8.3.5.3 Structural Wall Modules

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

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

Figure 3.8.3-8 shows the typical design details of the structural modules, typical configuration of the wall modules, typical anchorages of the wall modules to the reinforced base concrete, and connections between adjacent modules. Variations in the module design details and design elements used are described in the Figure 3.8.3-8 notes and in Subsections 3.8.3.1 and 3.8.3.1.3. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The application of ACI-349 and the supplemental requirements are supported by the behavior studies described in Subsection 3.8.3.4.1. The steel plate modules are anchored to the reinforced concrete basemat by mechanical connections welded to the steel plate. Loads are transferred directly from the faceplates to the base concrete using reinforcing bars, structural shapes, and shear studs connected directly to the modules with mechanical connectors or welds. The reinforcing steel used to anchor the modules to the concrete has a development that satisfies the requirements of ACI 349. The design of the surface plate, base plate, and vertical stiffeners is checked by finite element analysis. The design of critical sections is described in Subsection 3.8.3.5.8.

3.8.3.5.3.1 Design for Axial Loads and Bending

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

3.8.3.5.3.2 Design for In-Plane Shear

Design for in-plane shear is in accordance with the requirements of ACI-349, Chapters 11 and 14. The steel faceplates are treated as reinforcing steel, contributing as provided in Section 11.10 of ACI-349.

3.8.3.5.3.3 Design for Out-of-Plane Shear

Design for out-of-plane shear is in accordance with the requirements of ACI-349, Chapter 11.

3.8.3.5.3.4 Evaluation for Thermal Loads

The effect of thermal loads on the structural wall modules, with and without concrete fill, is evaluated by using the working stress design method for load combination 3 of Tables 3.8.4-1 and 3.8.4-2. This evaluation is in addition to the evaluation using the working stress design method of AISC N690 or the strength design method of ACI-349 for the load combinations without the thermal load.

Acceptance for the load combination with normal thermal loads, which includes the thermal transients described in Subsection 3.8.3.3.1, is that the 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 Service Level A loads given in ASME Code, Section III, Subsection NE, Paragraphs NE-3213.13 and 3221.4.

3.8.3.5.3.5 Design of Trusses

The trusses provide a structural framework for the modules, maintain the separation between the faceplates, support the modules during transportation and erection, and act as "form ties" between the faceplates when concrete is being placed. After the concrete has cured, the trusses act with shear studs to provide interfacial shear transfer between the faceplates and concrete. The trusses also provide structural integrity and permit the development of the nominal out-of-plane shear strength of the concrete without shear reinforcement. In areas of a module wall where the hooked dowels from a floor slab do not overlap the hooked dowels of an opposing floor slab on the opposite face of the wall, the horizontal channel member of the trusses may be credited as a tension tie, by confinement of the faceplates, preventing separation of the concrete. The welded connection of the truss to the faceplate is sized to fully develop the specified minimum yield strength of the channel member. As described in Subsection 3.8.3.1.3, additional structural capacity is provided between the module faceplates in some areas to support localized loads. The trusses, connecting steel plates, structural shapes, reinforcement bars, and tie bars are designed according to the applicable requirements of AISC N690 and ACI 349.

3.8.3.5.3.6 Design of Shear Studs

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

The design of the shear stud size and spacing includes consideration of shear transfer and faceplate buckling. Shear transfer is evaluated in accordance with AISC N690 Sections Q1.11.1 and Q1.11.4. Faceplate buckling is evaluated in accordance with AISC-N690 Section Q1.5.1.3. Conformance with these requirements precludes the possible failure modes for structural module construction for design basis loads.

3.8.3.5.4 Structural Floor Modules

Figure 3.8.3-3 shows the design details for a representative example of the floor modules. The details shown in Figure 3.8.3-3 are the representative design for a specific portion of the operating deck floor in containment. The size, material, and configuration of the structural shapes and plates, the amount and arrangement of the reinforcement, the location, size, and spacing of the shear studs welded on the top of the floor bottom plate, and the type and size of the welds may vary in the final design of floor modules in this and other locations. The operating floor is designed for dead, live, thermal, safe shutdown earthquake, and pressure due to automatic depressurization system operation or due to postulated pipe break loads. The operating floor region above the in-containment refueling water storage tank is a series of structural modules. The remaining floor is designed as a composite structure of concrete slab and steel beams in accordance with AISC-N690.

For vertical downward loads, the floor modules are designed as a composite section, according to the requirements of Section Q1.11 of AISC-N690. Composite action of the steel section and concrete fill is assumed based on meeting the intent of Section Q1.11.1 for beams totally encased in concrete.

Although the bottom plate of the floor, which is welded to the web and acts as the bottom flange, is not encased within concrete, the design configuration of the floor module provides complete concrete confinement to prevent spalling. It also provides a better natural bonding than the code-required configuration. When used, the shear studs welded on the top of the floor bottom plate supplement the natural bottom plate-to-concrete bond capability. The shear stud size and spacing is not directly credited in the composite section design. The capacity of the studs, whenever required for localized effects, is determined in accordance with ACI 349.

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

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

- Concrete provides restraint against buckling of steel plates. The buckling unbraced length of the steel plate, therefore, is assumed to equal the span length between the fully embedded steel plates and shapes.
- Although the floor modules forming the top (ceiling) of the in-containment refueling water storage tank are not in contact with water, stainless steel plates are used for the tank boundary.
- The floor modules are designed as simply supported beams.

3.8.3.5.4.1 Design for Vertical Downward Loads

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

3.8.3.5.4.2 Design for Vertical Upward Loads

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

3.8.3.5.4.3 Design for In-Plane Loads

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

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

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

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

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

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

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

Therefore, the condition at yield becomes:

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

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

Normal condition:

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

Severe condition:

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

Extreme/abnormal condition:

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

Thermal stresses in the faceplates result from restraint of growth during the thermal transients described in Subsection 3.8.3.3.1. Evaluation for thermal stresses is the same as discussed in Subsection 3.8.3.5.3.4 for the wall modules.

3.8.3.5.5 Internal Steel Framing

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

3.8.3.5.6 Steel Form Modules

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

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

3.8.3.5.7 Design Summary Report

A design summary report is prepared for containment internal structures documenting that the structures meet the acceptance criteria specified in Subsection 3.8.3.5.

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.

- The structural design meets the acceptance criteria specified in Section 3.8
- The seismic floor response spectra meet the acceptance criteria specified in Subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgment to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report.

3.8.3.5.8 Design Summary of Critical Sections

3.8.3.5.8.1 Structural Wall Modules

[*This subsection summarizes the design of the following critical sections:*

- *Southwest wall of the refueling cavity (4′ 0″ thick)*
- *South wall of west steam generator compartment (2′ 6″ thick)*
- *North east wall of in-containment refueling water storage tank (2′ 6″ thick)*

The thicknesses and locations of these walls which are part of the boundary of the in-containment refueling water storage tank are shown in Figure 3.8.3-18. The structural configuration and typical details are shown in Figures 3.8.3-1, 3.8.3-2, 3.8.3-8, 3.8.3-14 and 3.8.3-15. Plate thickness, structural shape size, and reinforcement provided may be increased locally. The design implemented in fabrication and construction drawings and instructions may have alternative structural shapes or reinforcement arrangements if they provide equal or better load capacity. As described in Subsection 3.8.3.1.3, steel plates, structural shapes, reinforcement bars, or tie bars are used in other locations between the structural module faceplates to address local out-of-plane loads.]* The structural analyses are described in Subsection 3.8.3.4 and summarized in Table 3.8.3-2. The design procedures are described in Subsection 3.8.3.5.3.

[*The details shown in Figure 3.8.3-17 are representative of connections between floors in containment and walls constructed using steel plate concrete composite construction.*]* The design details for the floor module are provided for background information and vary based on loading

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

conditions and geometry. The seat angles, beam seats, clip angles, shear plates, shear studs, and reinforcement bars shown in Figure 3.8.3-17 and other design elements are used as required for the connection. A key feature of the connection design is that the design elements provide a direct load path from the floor into the wall. Additional information on the connection design is provided in Subsection 3.8.3.1.3. Details of the connection design, including plate thickness, structural shape type and size, use of specific design elements, and reinforcement arrangement, vary based on local loads. The design implemented in fabrication and construction drawings and instructions may have alternative structural shapes or reinforcement arrangements if they provide sufficient load capacity to satisfy AISC N690 and ACI 349 criteria and requirements.

[*The three walls extend from the floor of the in-containment refueling water storage tank at elevation 103′ 0″ to the operating floor at elevation 135′ 3″. The south west wall is also a boundary of the refueling cavity and has stainless steel plate on both faces. The other walls have stainless steel on one face and carbon steel on the other.*]* For each wall design information is summarized in Tables 3.8.3-3, 3.8.3-4, 3.8.3-5, and 3.8.3-6. [*Results are shown at the middle of the wall (mid span at mid height), at the base of the wall at its mid point (mid span at base) and at the base of the wall at the end experiencing greater demand (corner at base). The first part of each table shows the member forces due to individual loading. The lower part of the table shows governing load combinations. The steel plate thickness required to resist mechanical loads is shown at the bottom of the table as well as the thickness provided. The maximum principal stress for the load combination including thermal is also tabulated. If this value exceeds the yield stress at temperature a supplemental evaluation is performed*]* as described in Subsection 3.8.3.5.3.4; [*for these cases the maximum stress intensity range is shown together with the allowable stress intensity range which is twice the yield stress at temperature.*]*

3.8.3.5.8.2 In-Containment Refueling Water Storage Tank Steel Wall

[*The in-containment refueling water storage tank steel wall is the circular boundary of the in-containment refueling water storage tank. The structural configuration and typical details are shown in sheet 3 of Figure 3.8.3-8.*]* The structural analyses are described in Subsection 3.8.3.4 and summarized in Table 3.8.3-2. The design procedures are described in Subsection 3.8.3.5.3. [*The steel wall extends from the floor of the in-containment refueling water storage tank at elevation 103′ 0″ to the operating floor at elevation 135′ 3″. The wall is a 5/8″ thick stainless steel plate. It has internal vertical stainless steel T-section columns spaced 4′-8″ apart and external hoop carbon steel (L-section) angles spaced 18″ to 24″ apart. The wall is fixed to the adjacent modules and floor except for the top of columns which are free to slide radially and to rotate around the hoop direction.*

The wall is evaluated as vertical and horizontal beams. The vertical beams comprise the T-section columns plus the effective width of the plate. The horizontal beams comprise the L-section angles plus the effective width of the plate. Table 3.8.3-7 shows the ratio of the design stresses to the allowable stresses. When thermal effects result in stresses above yield, the evaluation is in accordance with the supplemental criteria]* as described in Subsection 3.8.3.5.3.4.

3.8.3.5.8.3 Column Supporting Operating Floor

[*This subsection summarizes the design of the most heavily loaded column in the containment internal structures. The column extends from elevation 107′-2″ to the underside of the operating floor at elevation 135′-3″. In addition to supporting the operating floor, it also supports a steel grating floor at elevation 118′-0″.*

The load combinations in Table 3.8.4-1 were used to assess the adequacy of the column. For mechanical load combinations, the maximum interaction factor due to biaxial bending and axial load is 0.59. For load combinations with thermal loads, the maximum interaction factor is 0.94. Since the interaction factors are less than 1, the column is adequate for all the applied loads.]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

Subsection 3.8.4.6 describes the materials and quality control program used in the construction of the containment internal structures.

[*The structural steel modules are constructed using carbon steel plates and shapes (ASTM A36, ASTM A992, or steel with equal or better material properties*). *Duplex 2101 (American Society for Testing and Materials A240, designation S32101) stainless steel plates or steel with equal or better material properties*]* are used on the surfaces of the modules in contact with water during normal operation or refueling. [*The material of construction for studs attached to the module plates used to transfer loads into the concrete is A-108 or steel with equal or better material properties*. *Bars used to anchor the modules in the concrete are deformed bars according to Reference 19, Grade 60, and Reference 20.*]*

The structural wall and floor modules are fabricated and erected in accordance with AISC-N690. Loads during fabrication and erection due to handling and shipping are considered as normal loads as described in Subsection 3.8.4.3.1.1. Packaging, shipping, receiving, storage and handling of structural modules are in accordance with NQA-1, Subpart 2.2 (formerly ANSI/ASME N45.2.2 as specified in AISC N690).

3.8.3.6.1 Fabrication, Erection, and Construction of Structural Modules

Modular construction techniques are used extensively in the containment internal structures (Figure 3.8.3-1). Subassemblies, sized for commercial rail shipment, are assembled offsite and transported to the site. Onsite fabrication consists of combining the subassemblies in structural modules, which are then installed in the plant. A typical modular construction technique is described in the following paragraphs for Module CA01, which is the main structural module in the containment internal structures.

The CA01 module is a multicompartmented structure which, in its final form, comprises the central walls of the containment internal structures. The vertical walls of the module house the refueling cavity, the reactor vessel compartment, and the two steam generator compartments. The module (Figure 3.8.3-14) is in the form of a "T" and is approximately 88 feet long, 95 feet wide and 86 feet high. The module is assembled from about 40 prefabricated wall sections called structural submodules (Figure 3.8.3-15). The submodules are designed for railroad transport from the fabricator's shop to the plant site with sizes up to 12 feet by 12 feet by 80 feet long, weighing up to 80 tons. A typical submodule weighs between 9 and 11 tons. The submodules are assembled outside the nuclear island with full penetration welds between the faceplates of adjacent subunits.

The completed CA01 module is lifted to its final location within the containment vessel by the heavy lift construction crane. Following placement of the CA01 module within the containment building, the hollow wall structures are filled with concrete, forming a portion of the structural walls of the containment internal structures.

Tolerances for fabrication, assembly and erection of the structural modules conform to the requirements of section 4 of ACI-117, applicable sections of AWS D1.1, and sections Q1.23 and Q1.25 of AISC-N690. In walls around the reactor vessel cavity, where the concrete is placed between portions of unconnected modules or between a module and a left-in-place form, the tolerance for the wall thickness may be increased over those in ACI 117. These walls have been evaluated against ACI 349-01 reinforcement design requirements and demonstrated sufficient margin to accommodate the increased tolerance. Tolerances for shear stud spacing requirements are identified on Figure 3.8.3-8, Sheet 1 and conform to AWS D1.1, Paragraph 7.4.5.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.3.6.2 Nondestructive Examination

Nondestructive examination of the submodules and module is performed according to AISC-N690 and AWS D1.1. Welds are visually examined for 100 percent of their length. Full penetration welds are inspected by ultrasonic or radiographic examination for 10 percent of their length, or for 100 percent of the length of 1 weld in 10. Partial penetration welds are inspected by magnetic particle or liquid penetrant examination for 10 percent of their length, or for 100 percent of the length of 1 weld in 10.

3.8.3.6.3 Concrete Placement

After installation of the CA01 module in the containment, the hollow walls are filled with concrete. The concrete is placed through multiple delivery trunks located along the top of the wall or through windows in the module walls or pumping ports built into the module wall. It is placed in incremental layers with the placement rate based on the pressure of the wet concrete and its setting time. During concrete placement, workers and inspectors have access to the inside of the modules. The arrangement of the module internal trusses provides communication to aid in the free flow of concrete and movement of personnel.

3.8.3.7 In-Service Testing and Inspection Requirements

The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

3.8.3.8 Construction Inspection

Construction inspection is conducted to verify the concrete wall thickness and the surface plate thickness. The location for measurement of the structural wall modules is at the locations of the trusses used to provide the structural framework for the modules. Inspections will be measured at applicable sections excluding designed openings or penetrations. Inspections will confirm that each section provides the minimum required steel and concrete thicknesses as shown in Table 3.8.3-3. The minimum required steel and concrete thicknesses represent the minimum values to meet the design basis loads. Table 3.8.3-3 also indicates the steel plate thickness provided which may exceed the minimum required value for the following reasons:

- Structural margin
- Ease of construction
- Construction loads
- Use of standard thicknesses

[*As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.*]*

3.8.4 Other Category I Structures

The other seismic Category I structures are the shield building and the auxiliary building. New fuel and spent fuel racks are described in Section 9.1.

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.4.1 Description of the Structures

3.8.4.1.1 Shield Building

The shield building is the structure and annulus area that surrounds the containment building. It shares a common basemat with the containment building and the auxiliary building. The shield building uses concrete-filled steel plate construction (SC) as well as reinforced concrete (RC) structure. The figures in Section 1.2 show the layout of the shield building and its interface with the other buildings of the nuclear island.

A non-linear analyses performed on the nuclear island finite element model is used to validate the stiffness used in the evaluation of concrete-filled steel modules. This analysis used benchmarked layered shell elements for the reinforced concrete/steel concrete composite (RC/SC) connection, which transferred all stresses to the steel in the connections once a maximum yield stress in concrete was reached. These elements were included in the comparison made between linear and non-linear models. Results show the 80-percent stiffness model response spectra enveloped the non-linear model and provide a conservative approach in terms of response spectra and maximum stresses obtained in the shield building wall.

Figure 3.8.4-5 shows the following significant features and the principal systems and components of the shield building:

- Shield building cylindrical structure
- Shield building roof structure
- RC/SC connections
- Air inlets and tension ring
- Knuckle region (connection to exterior wall of PCS tank)
- Compression ring (connection to interior wall of PCS tank)
- Passive containment cooling system water storage (PCS) tank

The overall configuration of the shield building is established from functional requirements related to radiation shielding, missile barrier, passive containment cooling, tornado, and seismic event protection. These functional requirements led to establishing the design based on two primary design codes used for nuclear plant structures: 1) ACI 349 for reinforced concrete design, and 2) ANSI/AISC N690 for structural steel design.

The shield building SC walls are anchored to the RC basemat and shield building RC wall by mechanical connections. [*These RC-to-SC connections are shown in Figures 1, 2, 3, and 4 of APP-GW-GLR-602 (Reference 57).*]* These RC-to-SC connections are also used in the other regions of the shield building, including:

- Auxiliary building RC roof connection to the shield building SC wall
- Auxiliary building RC wall connection to shield building SC wall
- Tension ring connection to the shield building RC roof

The connections provide for the direct transfer of forces from the RC reinforcing steel to the SC liner plates. A typical auxiliary building RC roof connection to the shield building SC wall is shown in [*Figure 7 of APP-GW-GLR-602 (Reference 57).*]*

The cylindrical shield wall has an outside radius of 72.5 feet and a thickness of 36 inches. The cylindrical wall section that is a few feet below the auxiliary building roof line is a reinforced concrete structure. The section that is not protected by the auxiliary building is a steel concrete composite structure; see Figure 3H.5-16. The overall thickness of 36 inches is the same as the RC wall below. The concrete for the SC portion is standard concrete with compressive strength of 6000 psi. The SC

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portion is constructed with steel surface plates, which act as concrete reinforcement. The 0.75-inch tie bars are welded to the steel faceplates to develop composite behavior of the steel faceplates and concrete. The shear studs are welded to the inside surface of the steel plate. The tie bar spacing is reduced in the higher stress regions. A typical SC wall panel is shown in Figure 3H.5-13 and [*Figure 5 of APP-GW-GLR-602 (Reference 57).*]*

The tension ring is located at the interface of the shield building steel concrete composite air inlet structures and the shield building reinforced concrete roof. The top of the tension ring interfaces with the RC roof slab. The tension ring supports the roof girders that are located under the RC roof slab. The bottom of the tension ring is attached to the air inlets structure. The bottom of the air inlets structure is attached to the top of the cylindrical SC wall of the shield building. The connection of the tension ring to the roof is a mechanical connection design and is described above. The tension ring and air inlets are shown in Figure 3H.5-14.

The primary function of the tension ring is to resist the thrust from the shield building roof. The air inlets structure is located directly below the tension ring and includes the air openings that provide for natural circulation of cooling air. Though its steel plates are connected to the concrete infill by studs and tie bars, the tension ring is conservatively designed as a hollow steel box girder. The concrete infill is credited only for out-of-plane shear transfer and for stability of the steel plates. The tension ring is designed to have high stiffness and to remain elastic under required load combinations.

The air inlets structure is a 4.5-foot-thick SC structure with through-wall openings for air flow. The air inlet openings consist of circular pipes at a downward inclination of 38 degrees from the vertical. Steel plates on each face, aligned with the inner and outer flanges of the tension ring, serve as primary reinforcement. The concrete infill is connected to the steel plates with tie bars and studs. The top of the air inlets structure is welded to the underside of the tension ring. The bottom of the air inlets structure is welded to the SC wall.

The shield building conical roof steel structure consists of 32 radial beams. Between each pair of radial beams there are circumferential beams. A steel plate is welded to the top flanges of each beam and forms a surface on which the concrete is placed. The steel structure forms a conical shell that spans the area from the compression ring to the tension ring.

The outside diameter of the PCS tank (passive containment cooling water storage tank) intersects with the shield building roof at the knuckle region. Outside of the PCS tank, the concrete roof slab thickness is 3 feet and at the bottom of the PCS tank the concrete thickness is 2 feet. The wall from the PCS tank applies a load to the roof slab, and also provides stiffness and increases the strength of the roof in that region.

The inside diameter of the PCS tank intersects with the roof slab at the compression ring. The compression ring provides the compression support for the conical roof dome. It consists of a composite structure having a curved steel beam section, which supports the concrete roof directly above it. The inside wall of the PCS tank is located above the concrete roof. Studs are placed on the top flange of the steel girder to allow the steel and concrete sections to act as a composite unit. The curved girder is designed to provide support for the steel structure during construction and during the initial placement of the concrete roof before the concrete has hardened sufficiently.

The PCS tank sits on top of the shield building roof. It is supported by and acts integrally with the conical roof. The inside surface has a liner that functions to provide leak protection, but is not required to provide structural strength to the structure. Leak chase channels are provided over the liner welds. The top elevation of the water inside the tank for the PCS has sufficient freeboard to preclude impact on the roof during the SSE.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Appendix 3H provides additional information regarding the shield building design, testing, and analysis that demonstrates the robust behavior of the design with respect to ductility, effects of creep, reserve strength of seismic margins, buckling, and shrinkage.

3.8.4.1.2 Auxiliary Building

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

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

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

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

Structural modules are used for part of the south side of the auxiliary building. These structural modules are structural elements built up with welded steel structural shapes and plates. Concrete is used where required for shielding, but reinforcing steel is not normally used. These modules include the spent fuel pool, fuel transfer canal, and cask loading and cask washdown pits.

The configuration of the auxiliary building structural modules is similar to the structural modules described in Subsection 3.8.3.1 for the containment internal structures. As in the containment internal structures, the structural wall modules consist of steel faceplates connected by steel trusses with shear studs welded to the inside faces of the steel faceplates. Figure 3.8.4-4 shows the location of the structural modules in the auxiliary building. The structural modules extend from elevation 66′-6″ to elevation 135′-3″.

The auxiliary building modules are different in configuration and design details compared to the containment internal structures. The following examples are provided. The thickness of the structural wall modules differs from the figures for the containment internal structures modules and ranges from 2′-6″ to 5′-0″ for the key structural walls. The auxiliary building modules also includes smaller structural walls that act as labyrinth walls or barrier access walls with thicknesses less than 2′-6″. Local variation in the design of the trusses and spacing of the trusses and shear studs may be required to address internal obstructions and accessibility for fabrication and inspection. The minimum design thickness of the faceplates is 0.5 inch. Portions of the faceplates are thicker than the nominal design thickness to provide strength for localized loads due to attachments and connections. The auxiliary building modules are placed on the nuclear island basemat and do not have an offset of elevation of the bottom of the module faceplates through the thickness of the wall that is shown in Figure 3.8.3-8 for a portion of a module inside containment. The detail design of the internal connections within the modules including those at the corners, and for design elements and structures within the modules such as the diaphragms associated with the corner design, is different than depicted for the containment internal structures. The design of the mechanical connection of the module to the basemat varies for details such as size of deformed bars, thickness and configuration of plates, design of welds, and use of bolted connections in lieu of welds.

The ceiling of the main control room (floor at elevation 135′-3″), and the instrumentation and control rooms (floor at elevation 117′-6″) are designed as finned floor modules (Figure 3H.5-9). A finned floor consists of a 24-inch-thick concrete slab poured over a stiffened steel plate ceiling. The fins are rectangular plates welded perpendicular to the plate. Shear studs are welded on the other side of the steel plate, and the steel and concrete act as a composite section. Figure 3H.5-9 shows the finned floor above the main control room and the connection of the floor to the wall on column line 11. The finned floors above the instrumentation and control rooms are similar, but differ with some design details, such as the size and spacing of reinforcement and the connecting dowels used in the connection of the floor to the wall. Penetrations and other interferences in the floors and adjacent walls may cause localized variances with the design details shown in the figure. The fins are exposed to the environment of the room, and enhance the heat-absorbing capacity of the ceiling (see Design Control Document (DCD) Subsection 6.4.2.2). Several shop-fabricated steel panels, placed side by side, are used to construct the stiffened plate ceiling in a modularized fashion. The number of panels used is determined by the size of the room and fabricator capabilities. The stiffened plate is designed to withstand construction loads prior to concrete hardening. Additional bottom layer reinforcing steel is provided in the finned floors at elevation 117'-6" where needed to maintain the structural integrity of the fire barrier during a fire event due to potential deterioration of mechanical properties of exposed steel fin plates during the fire.

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

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

3.8.4.1.3 Containment Air Baffle

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

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

- A series of panels attached to the containment vessel cylindrical wall and the knuckle region of the dome
- A flexible seal closing the gap between the wall and the panels fixed to the containment vessel, designed to accommodate the differential movements between the containment vessel and shield building
- Flow guides attached at the bottom of the air baffle to minimize pressure drop

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

- The baffle and its supports are configured to minimize pressure losses as air flows through the system
- The baffle and its supports have a design objective of 60 years
- The baffle and its supports are configured to permit visual inspection and maintenance of the air baffle as well as the containment vessel. Periodic visual inspections are primarily to inspect the condition of the coatings
- The baffle is designed to maintain its function during postulated design basis accidents
- The baffle is designed to maintain its function under specified external events including earthquakes, hurricanes and tornadoes

The general arrangement of the containment air baffle is shown in sheet 1 of Figure 3.8.4-1. The course layout, plate/insert plate geometry, weld seams for the containment, hatches, airlocks, and the number and configuration of the containment air baffle panels are provided for illustrative purposes only. The lowest elevation of the air baffle is approximately 142′-0". The upper air baffle panels are on the head. The air baffle is supported by U-shaped support attachments welded to the containment vessel. The attachments can be installed in the subassembly area and, therefore, should not interfere with the containment vessel erection welds. The only penetrations through the containment vessel above the operating deck at elevation 135′-3″ are the main equipment hatch and personnel airlock. Air baffle panels and flow guides around the equipment hatch and personnel airlock are designed to accomodate the hatch and airlock insulated enclosures.

Panels are attached to the containment vessel above the cylindrical portion. The panels are angled to follow the curvature of the knuckle region of the containment vessel head, forming a conical baffle that provides a transitional flow region into the upper shield building. A flexible seal is provided between this upper row of panels and the air baffle that is attached directly to the shield building roof as shown in sheet 4 of Figure 3.8.4-1. This flexible seal is attached to the top of the upper row of panels. The flexible seal is set at ambient conditions to permit relative movements from minus 2 inches to plus 3 inches radially and minus 1 inch to plus 4 inches vertically. This accommodates the differential movement between the containment vessel and the shield building, based on the absolute sum of the containment pressure and temperature deflections and of the seismic deflections, such that the integrity of the air baffle is maintained.

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

3.8.4.1.4 Seismic Category I Cable Tray Supports

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

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

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

3.8.4.2 Applicable Codes, Standards, and Specifications

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

[• *American Concrete Institute (ACI), Code Requirements for Nuclear Safety Related Concrete Structures, ACI 349-01*]* (refer to Subsection 3.8.4.5 for supplemental requirements)

- [*American Concrete Institute (ACI), Building Code Requirements for Structural Concrete, ACI 318-11, Section 12.6*]* (refer to subsection 3.8.4.4.1 for application of ACI 318-11 Section 12.6)

- American Concrete Institute (ACI), ACI Detailing Manual

- American Concrete Institute (ACI), Self-Consolidating Concrete, ACI-237R

[• *American Institute of Steel Construction (AISC), Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities, AISC-N690-1994*]* (refer to Subsection 3.8.4.5 for supplemental requirements)

[• *American Welding Society (AWS), Structural Welding Code - Steel, AWS D1.1-2000*

AWS D1.1-2000 is an acceptable code for use in lieu of AWS D1.1-1992 for the design, qualification, fabrication, and inspection for AISC N690 applications.

The use of AWS D1.1-2000 to replace and supplement provisions in AISC N690-1994 includes the provisions noted below:

– *The provisions in Section 2.3.4.1 of AWS D1.1-2000 are used to size complete joint penetration groove welds for tubular members.*

– *ASTM A992 is an alternative to ASTM A36 steel for structural steel shapes. Where ASTM A992 steel is used, the provisions for matching weld metal identified in AWS D1.1-2000, Section 3.3 and Table 3.1 are used to supplement the information in AISC N690 Section Q1.4.4.*

– *Where directionality is applied to increase allowable weld stress for fillet welds identified in AISC N690, Section Q1.5.3 and Table Q1.5.3, the provisions in Sections 2.14.4 and 2.14.5 of AWS D1.1-2000 shall be used.*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

For fillet weld groups concentrically loaded in the plane of the axis of the weld and consisting of elements oriented both longitudinally and transversely to the direction of applied load, the deformation capability of the elements shall be considered. As a supplement to the provisions in AWS D1.1-2000, the strength of the weld group is limited to the larger of the two following evaluations:

1. *The combined strength of the individual weld elements using 1.0 times the strength of the longitudinally oriented element and 1.0 times the strength of the transversely oriented element*

2. *The combined strength of the individual weld elements using 0.825 times the strength of the longitudinally oriented element and 1.5 times the strength of the transversely oriented element*

The provisions for directionality are not applied to fillet welds of tubular member T, Y, and K connections described in Table 2.5 of AWS D1.1-2000.

The provisions in AWS D1.1-2000 for the application of directionality to linear fillet welds are supplemented by extending the provisions to weld configurations not specified in AWS D1.1-2000 as noted below:

– *Where directionality is considered for concentric out-of-plane loading for linear fillet weld groups, including lines and rectangular configurations, the directionality provisions in AWS D1.1-2000 are applied.*

– *Where directionality is considered for concentric out-of-plane loading for circular-shaped fillet weld configurations, the directionality provisions in AWS D1.1-2000 are applied. The use of directionality provisions in AWS D1.1-2000 for circular-shaped fillet weld configurations is limited to tension loading concentrically applied out-of-plane to the axis of the weld.*]*

- American Iron and Steel Institute (AISI), Specification for the Design of Cold Formed Steel Structural Members, Parts 1 and 2, 1996 Edition and 2000 Supplement

Section 1.9 describes conformance with the Regulatory Guides.

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

3.8.4.3 Loads and Load Combinations

The loads and load combinations used in the analysis are considered to be part of the method of evaluation.

3.8.4.3.1 Loads

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

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3.8.4.3.1.1 Normal Loads

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

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

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

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

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

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

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

3.8.4.3.1.2 Severe Environmental Loads

The severe environmental load is the following:

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

3.8.4.3.1.3 Extreme Environmental Loads

Extreme environmental loads are the following:

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

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

N = Loads generated by the probable maximum precipitation (provided previously in Table 2.0-201).

The application of the 48-hour PMWP and the 100-year return period ground-level snowpack in the roof design of safety-related structures is addressed in Subsection 2.3.1.3.4.

3.8.4.3.1.4 Abnormal Loads

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

P_a = Pressure load within or across a compartment generated by the postulated break. The main steam isolation valve (MSIV) compartments are designed for a 6.5 psi pressurization load. The steam generator blowdown valve compartment is designed for a 6 psi pressurization load. The subcompartment design pressure bounds the pressurization effects due to postulated breaks in high energy pipe. Determination of subcompartment pressure loads is discussed in Subsection 6.2.1.2.

Ta = Thermal loads under thermal conditions generated by the postulated break and including To. Determination of subcompartment temperatures is discussed in Subsection 6.2.1.3.

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

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

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

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

3.8.4.3.1.5 Dynamic Effects of Abnormal Loads

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

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

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

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

3.8.4.3.2 Load Combinations

3.8.4.3.2.1 Steel Structures

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

3.8.4.3.2.2 Concrete Structures

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

3.8.4.3.2.3 Live Load for Seismic Design

Floor live loads, based on requirements during plant construction and maintenance activities, are specified varying from 50 to 250 pounds per square foot (with the exception of the containment operating deck which is designed for 800 pounds per square foot specified for plant maintenance condition).

For the local design of members, such as the floors and beams, seismic loads include the response due to masses equal to 25 percent of the specified floor live loads or 75 percent of the roof snow load, whichever is applicable. These seismic loads are combined with 100 percent of these specified live loads, or 75 percent of the roof snow load, whichever is applicable, except in the case of the containment operating deck. For the seismic load combination, the containment operating deck is designed for a live load of 200 pounds per square foot which is appropriate for plant operating condition. The mass of equipment and distributed systems is included in both the dead and seismic loads.

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Seismic Category I Structures

[*The design and analysis procedures for the seismic Category I structures (other than the containment vessel, containment internal structures, and other structures constructed using concrete-filled steel plate construction), including assumptions on boundary conditions and expected behavior under loads, are in accordance with ACI-349 with clarification as provided below for concrete structures, with AISC-N690 for steel structures, and AISI for cold formed steel structures.*]* For the column line N Wall from 2 to 4 from 100'-0" to 135'-3", the tolerance for the wall thickness is increased over those in ACI 349-01. These walls were evaluated against ACI 349-01 reinforcement design requirements and demonstrated sufficient margin to accommodate the increased tolerance. The structural wall modules in the auxiliary building are designed using similar design concepts as the structural modules in the containment internal structures described in Subsection 3.8.3.5.3. The shield building is designed using the procedures and requirements described in Subsection 3.8.4.5.5.

[*The criteria of ACI-349, Chapter 12, are applied in development and splicing of the reinforcing steel. The ductility criteria of ACI-349, Chapter 21, are applied in detailing and anchoring of the reinforcing steel. Provisions in ACI 318-11 Section 12.6*]* (Reference 58) [*for headed and mechanically anchored deformed bars apply to development of headed reinforcement as an alternative to the provisions in ACI 349 Appendix B.*]* The specific application of ACI 318-11 Section 12.6 is used for development of the headed reinforcement to provide ductile behavior. Alternatively, a rebar hook with requirements according to ACI 349 may also be used.

[*The application of Chapter 21 detailing is demonstrated in the reinforcement details of critical sections*]* in Subsection 3.8.5 and Appendix 3H.

[*Sections 21.2 through 21.5 of Chapter 21 of ACI 349 are applicable to frame members resisting earthquake effects. These requirements are considered in detailing structural elements subjected to significant flexure and out-of-plane shear. These elements include the following examples described in Appendix 3H*:]*

- Reinforcement details for the basemat are described in Subsection 3.8.5. [*Shear stirrups have T headed anchors at each end. Provisions in ACI 318-11 Section 12.6*]* (Reference 58) [*for headed and mechanically anchored deformed bars apply to headed shear reinforcement as an alternative to the provisions in ACI 349 Appendix B.*]* The specific application of ACI 318-11 Section 12.6 is used for development of the headed shear reinforcement to provide ductile behavior.

- Reinforcement details for the exterior walls below grade are described in Subsection 3H.5.1.1. [*Shear ties have T headed anchors at each end. Provisions in ACI 318-11 Section 12.6*]* (Reference 58) [*for headed and mechanically anchored deformed bars apply to headed reinforcement as an alternative to the provisions in ACI 349 Appendix B.*]* The specific

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application of ACI 318-11 Section 12.6 is used for development of the headed reinforcement to provide ductile behavior. Alternatively, a conventional tie with alternating 90 and 135 degree hooks around the longitudinal reinforcement may be used.

[*Sections 21.2 and 21.6 of Chapter 21 of ACI 349 are applicable to walls, diaphragms, and trusses serving as parts of the earthquake force-resisting systems as well as to diaphragms, struts, ties, chords and collector elements. These requirements are considered in the detailing of reinforcement in the walls and floors of the auxiliary building and in the shield building cylindrical wall and roof.*]*

- Reinforcement for the shear walls and floors are shown in Subsections 3H.5.1 to 3H.5.4. [*Transverse reinforcement terminating at the edges of structural walls or at openings is detailed in accordance with 21.6.6.5 of ACI 349.*]*

The bases of design for the tornado, pipe breaks, and seismic effects are discussed in Sections 3.3, 3.6, and 3.7, respectively. The foundation design is described in Subsection 3.8.5.

The seismic Category I structures are reinforced concrete, concrete-filled steel plate, and structural module shear wall structures consisting of vertical shear/bearing walls and horizontal slabs supported by structural steel framing. Seismic forces are obtained from the response spectrum analysis of the three dimensional finite element models described in Table 3G.1-1. These results are modified to account for accidental torsion as described in Subsection 3.7.2.11. Where the refinement of these finite element models is insufficient for design of the reinforcement, for example in walls with a large number of openings, detailed finite element models are used. Also evaluated and considered in the shear wall and floor slab design are out-of-plane bending and shear loads, such as live load, dead load, seismic, lateral earth pressure, hydrostatic, hydrodynamic, and wind pressure. These out-of-plane bending and shear loads are obtained from the response spectrum analyses supplemented by hand calculations.

The exterior walls of the seismic Category I structures below the grade are designed to resist the worst case lateral earth pressure loads (static and dynamic), soil surcharge loads, and loads due to external flooding as described in Section 3.4. The lateral earth pressure loads are evaluated for two cases:

- Lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98 (Reference 56), Subsection 3.5.3, Figure 3.5-1, “Variation of Normal Dynamic Soil Pressures for the Elastic Solution”

- Lateral earth pressure equal to the passive earth pressure

The shield building roof and the passive containment cooling water storage tank are analyzed using the three-dimensional finite element quadrant model described in Subsection 3G.2.3.1 with the ANSYS computer code. Loads and load combinations are given in Subsection 3.8.4.3 and include construction, dead, live, thermal, wind and seismic loads. The seismic response of the water in the tank is applied as static pressures corresponding to the impulsive and convective response. The results are used in the design of the tension ring, air inlet structure, PCS tank, shield building roof, and radial roof beams. The PCS tank is designed using the maximum accelerations at the applicable elevation resulting from time history dynamic analyses of the nuclear island. The tension ring and air inlet use maximum accelerations that are increased such that the member forces in these regions envelope those from a response spectrum analysis using the refined NI05 model, as described in Appendix 3G.2.2.4.

The liner for the passive containment cooling water storage system tank is analyzed by hand calculation. The design considers construction loads during concrete placement, loads due to handling and shipping, normal loads including thermal, and the safe shutdown earthquake. Buckling

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of the liner is prevented by anchoring the liner using the embedded stiffeners and welded studs. The liner is designed as a seismic Category I steel structure in accordance with AISC N690 with the supplemental requirements given in Subsection 3.8.4.

The structural steel framing is used primarily to support the concrete slabs and roofs. Metal decking, supported by the steel framing, is used as form work for the concrete slabs and roofs. The structural steel framing is designed for vertical loads. Appendix 3H shows typical structural steel framing in the auxiliary building.

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

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

The concrete floors on steel plates, including the floors in the CA20 module, are designed as reinforced concrete slabs in accordance with ACI-349. The steel panels are designed and constructed in accordance with AISC-N690. For positive bending, the steel plate is in tension and the steel plate serves as the bottom reinforcement. For negative bending, compression is resisted by the concrete and steel plate and the tension by top reinforcement in the concrete. This methodology is similar to that described for the control room ceiling in Subsection 3H.5.4.

The design of the connection meets ACI 349 and AISC N690 requirements for shear stud capacity. The anchorage of the reinforcing dowels may vary from that shown in Figure 3H.5-9, Sheet 2. Differences in dowel anchorage details meet ACI 349 requirements for standard hooks and ACI 318-11 requirements for headed reinforcement.

The construction sequence for CA20 floors is as follows:

- Each panel (steel plates, structural tees, and shear connectors) is fabricated in the shop, brought to the floor location, and placed in position. In some cases, panels are preassembled and placed as a module. The bottom plate is supported by brackets connected to the walls.
- The steel plate is used as the formwork for concrete placement. The wet concrete load is carried by the steel plates and the structural tees that act to stiffen the bottom plate.
- During concrete placement, shoring may be provided for floors with longer spans. Local shoring of the steel plates at penetrations and other openings in the floor and supporting wall, or to act as temporary support, may also be provided.

3.8.4.4.2 Seismic Category I Cable Tray Supports

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

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

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.4.4.4 Below Grade Exterior Walls

The design and analysis procedures for seismic Category I exterior walls below grade are described below.

The nuclear island exterior walls below grade are subjected to various loads, including the lateral earth pressure loads. Lateral loads used in design of the nuclear island are based on conservative assumptions (soil profiles with highest lateral loads) for the properties of the soil adjacent to the exterior walls. Lateral loads are calculated for a range of possible soil properties and a conservative set of loads is specified for design.

The plant grade elevation is 100′-0″, the high groundwater level is 98′-0″, and the maximum flood level is at plant elevation 100′-0″.

Load Conditions

Hydrostatic (groundwater) (Live, L)

The design high groundwater level is at elevation 98′-0″, and the probable maximum flood level is at elevation 100′-0″. Both of these loads are treated as live loads. The hydrostatic unit water pressure (P_w) at a depth h (units: feet) below ground level is calculated as:

$$P_w = \gamma_w h \tag{1}$$

Where, γ_w = unit weight of water = 62.4 pcf

At-Rest Earth Pressure (Earth, H)

Static earth pressure is based on “at -rest” conditions, and the coefficient of earth pressure for the at-rest condition (K_o) is determined from the following relationship:

$$P_o = K_o \gamma h \tag{2}$$

Where,

K_o = 1-SIN(φ)

φ = angle of internal friction

h = depth below grade (El. 100′-0″)

$\gamma = \gamma_s$ = Saturated unit weight of granular back fill above water table, or

$\gamma = \gamma_s - \gamma_w$ below water table

Static and Dynamic Surcharge Pressures

The static surcharge pressure (Dead, D) is considered a static pressure load, and therefore, the at-rest coefficient (K_o) is used. The static lateral surcharge pressure is defined for the same soil case as the at-rest pressure as follows:

$$P_{surch} = K_o q \tag{3}$$

Where,

q = static surcharge pressure based on footprint loads of adjacent structures

The dynamic lateral surcharge pressure (Seismic, Es_1) is based on 0.3 times the static surcharge pressure. The static and dynamic lateral surcharge pressures act uniformly along the height of the exterior walls.

Dynamic Earth Pressure (Seismic, Es_2)

The dynamic earth pressure is calculated in accordance with ASCE 4-98 (Reference 56), Subsection 3.5.3, Figure 3.5-1, “Variation of Normal Dynamic Soil Pressures for the Elastic Solution.” The Poisson’s ratio (v) for the soil varies between 0.35 and 0.4. The most conservative dynamic soil pressure distribution, obtained using 0.4 for v, is used. The seismic acceleration levels are the maximum accelerations associated with the seismic response of the nuclear island from elevation 60.5′ to elevation 100′. The north-south seismic excitation acceleration values are associated with Walls 1 and 11. The east-west seismic acceleration values apply to Walls N, Q, and I.

Passive Earth Pressure (Seismic, Es_3)

The nuclear island includes passive pressure on the exterior walls to resist sliding during an SSE. Therefore, the exterior walls below grade shall also be designed for passive earth pressure in the load combinations that include Es. The passive earth pressure is calculated from:

$$P_P = K_P \gamma h \qquad (4)$$

Where,

$K_P = \mathrm{Tan}^2(45^\circ + \varphi / 2)$

φ = angle of internal friction

h = depth below grade (El. 100′-0″)

$\gamma = \gamma_s$ = Saturated unit weight of granular back fill above water table or

$\gamma = \gamma_s - \gamma_w$ below water table

Load Combinations

Lateral loading conditions, identified above, are used for the following load combinations:

- Load Combination 3: (L) + (H) + (D) + (Es_1) + (Es_2)
- Load Combination 7: (L) + (D) + (Es_1) + (Es_3)
- Load Combinations 1 & 2: 1.4 x (D) + 1.7 x ((L) + (H))
- Load Combinations 4, 5 & 6: (L) + (H) + (D)
- Load Combinations 8 & 9: 1.05 x (D) + 1.3 x ((L) + (H))

The design of the exterior below grade walls is designed from the load combinations listed above, which incorporate the effects of full lateral passive earth pressure.

Based on the results for the load combinations, the critical lateral pressure distribution for the design loads along the exterior, below grade walls would result when combining the effects of Load Combinations (LC) 3 and 7 along the nuclear island’s east-west axis. The pressure distribution for LC 3 would control from elevation 100′ (ground surface) to the intersection with the LC 7 pressure distribution. Graphically, this occurs at approximately elevation 91′. The LC 7 plot would control from this intersection to elevation 60.5′ (bottom of the basemat). LC 7 incorporates the effects of full lateral passive earth pressure.

Lateral Soil Pressure with Vertical Seismic Effects

Soil Pressure – Vertical seismic is the at-rest earth pressure for the generic soil model multiplied by 0.3 (seismic acceleration factor). This same factor was used to determine the dynamic lateral surcharge pressure (Seismic, Es_2) noted above. These tabular values where combined with the values from the dynamic soil pressure (along the east-west axis) by the SRSS Method (square root, sum of squares) and plotted against the dynamic soil pressure (along the east-west axis) alone. The contribution of vertical seismic effect on the at-rest earth pressure is negligible to the dynamic soil pressure. Therefore, vertical seismic effect is neglected and the dynamic soil pressure alone was used in determining the load combination.

3D SASSI Analyses

3D SASSI soil-structure interaction (SSI) analyses for soil conditions soft-to-medium (SM) and upper-bound-soft-to-medium (UBSM) soil were used to estimate the dynamic soil pressure on the exterior nuclear island walls. Five locations were used to calculate lateral pressure on the structure, including spring elements at elevation 100′ and 82.5′, and structure elements at elevation 91′, 71′, and 57′. Axial spring forces normal to and along each wall are tabulated and converted to a unit load and corresponding stress at each elevation. Similarly, element stresses normal to the plane of the wall are tabulated for each elevation, and the maximum normal stress was determined.

The 3D SASSI UBSM and SM dynamic soil pressure for the north-south and east-west vertical profiles are compared to the ASCE 4-98 lateral soil pressure distribution curve.

3D ANSYS Analyses

The analyses of the nuclear island for the lateral earth pressure loads are performed using 3D linear elastic ANSYS analysis. The lateral earth pressure loads are applied to the finite element nodes associated with embedded nuclear island structural walls. The analysis performed is linear elastic. Loads and moments at the base of the exterior walls are extracted from the analyses and are added in the design analyses of the basemat.

3.8.4.5 Structural Criteria

[*The analysis and design of concrete conform to ACI-349 as supplemented below and with clarifications provided in Subsection 3.8.4.4.1. The analysis and design of structural steel conform to AISC-N690 as supplemented below and with clarifications provided in Subsection 3.8.4.5.2. The analysis and design of cold-formed steel structures conform to AISI. The margins of structural safety are as specified by those codes.*]*

3.8.4.5.1 Supplemental Requirements for Concrete Structures

[*Supplemental requirements for ACI-349-01 are given in the position on Regulatory Guide 1.142 in Appendix 1A. The structural design meets the supplemental requirements identified in Regulatory Positions 2 through 8, 10 through 13, and 15.*]*

Paragraph 21.6.1 of ACI 349-01 should reference 21.6.6 instead of 21.6.5. Paragraph 21.6.5 in ACI 349-97 was renumbered to 21.6.6 in ACI 349-01, and the reference in 21.6.1 was not updated. The errata for ACI 349-01 are being updated to include this correction. This makes the paragraph consistent with ACI 349-97, which was endorsed by Regulatory Guide 1.142.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

[*Design and construction of fastening to concrete is in accordance with ACI 349-01, Appendix B*]* and are in conformance with the regulatory positions of NRC Regulatory Guide 1.199, Revision 0. [*Alternative requirements to ACI 349, Appendix B apply to the anchoring of headed shear reinforcement for the basemat (see Subsection 3.8.4.4.1). Alternative requirements to ACI 349, Appendix B apply to the anchoring of headed reinforcement above the basement (see Subsection 3.8.4.4.1).*

Weldable coupler connections of reinforcing bar to structural steel shall develop 125% of the specified yield strength of the bar in accordance with ACI 349-01 Section 12.14.3.4. Qualification of the C2/C3J coupler welds is demonstrated as follows:

For reinforcing bar sizes #4, #5, and #6, the coupler connection weld adequacy is demonstrated by calculations in accordance with AISC N690-1994 requirements using a stress limit coefficient (SLC) of 1.6.

For reinforcing bar sizes #7, #8, #9, #10, and #11, coupler connection weld adequacy is demonstrated through testing, as permitted by AISC N690-1994 Section Q1.0.1, and more specifically, Section Q1.22.2. Two sets of testing are performed to demonstrate that the strength of the reinforcing bar is the limiting feature of the coupler reinforcing bar splice and weld system:

(1) Six static tension tests are performed for each reinforcing bar size on samples of the coupler reinforcing bar splice and weld system, retaining the 90%/95% upper confidence limit of the coupler system static tension test results. Three cyclic tests are also performed as described in ACI 349-01 12.14.3.4.1(b) to confirm that there is not significant coupler system degradation under cyclic demand.

(2) Static tension testing of the coupler weld to failure using ten representative sample weld configurations from each of the five reinforcing bar coupler sizes is performed to determine the 90%/95% lower confidence limit weld capacity.

The tests demonstrate that the factor of safety between the weld 90%/95% lower confidence limit and 125% of the specified yield strength of the reinforcing bar exceeds 1.4. The tests also demonstrate that the factor of safety between the weld 90%/95% lower confidence limit and the 90%/95% upper confidence limit of the coupler system static tension test results exceeds 1.0. The analyses and tests establish a minimum fillet reinforcement size for the C2/C3J couplers:

C2/C3J Coupler Size	*#4*	*#5*	*#6*	*#7*	*#8*	*#9*	*#10*	*#11*
Min. Fillet Weld Size (inches)	*1/4*	*1/4*	*1/4*	*1/4*	*1/4*	*5/16*	*1/4*	*3/8*]*

The tests also demonstrate that non-conforming 1/4-inch fillet welds for #9 and #11 C3J couplers identified in module CA20 are acceptable by comparing the 90%/95% lower confidence limit weld strength to 125% of the specified yield strength of the reinforcing bar and to the 90%/95% upper confidence limit of the coupler system static tension test results. The factor of safety exceeds 1.3 in comparison to 125% of the specified yield strength of the reinforcing bar and 1.0 in comparison to the 90%/95% upper confidence limit of the coupler system.

3.8.4.5.2 Supplemental Requirements for Steel Structures

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

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

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

- *In Table Q1.5.7.1, the stress limit coefficients for compression are as follows:*

  1.3 instead of 1.5 in load combinations 2, 5, and 6.

  1.4 instead of 1.6 in load combinations 7, 8, and 9.

  1.6 instead of 1.7 in load combination 11.

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

- *Sections Q1.24 and Q1.25.10 are supplemented as follows:*

  Shop painting is in accordance with Section M of the Manual of Steel Construction, Load and Resistance Factor Design, First Edition. Exposed areas after installation are field painted in accordance with the applicable portion of Chapter M of the Manual of Steel Construction, Load and Resistance Factor Design, First Edition.]* See Subsection 6.1.2.1 for additional description of the protective coatings.

- [*In Sections Q1.26.2.2, Q1.26.2.3, and Q1.26.3 for the non-conforming partial penetration welds associated with reinforcement bar sizes #6 and #9 C3J couplers installed on ASTM A240 stainless steel embedment plates under CA01 that did not undergo non-destructive examination at the time of fabrication, as identified in Amendment Nos. 80 and 79 for VEGP Units 3 and 4, respectively, the quality and strength of the welds is demonstrated through visual examination and through static tension testing of an uninstalled representative population as follows:*

  - *Visual Examination: Coupler welds from the production fabrication population underwent visual examination by the manufacturer. The manufacturer visual examinations provided satisfactory results.*

  - *Static Tension Testing: Weldable coupler connections of reinforcing bar to structural steel shall develop 125% of the specified yield strength of the bar in accordance with ACI 349-01, Section 12.14.3.4. The mechanical connection strength requirement is applied to the weld to demonstrate that the coupler weld is stronger than the reinforcing bar strength requirement, thereby satisfying provisions for design limits outlined in AISC N690-1994. To determine that the population of #6 and #9 C3J coupler welds on ASTM A240 stainless steel embedment plates are adequate in their ability to perform their intended design function, static tension testing of an uninstalled representative population of production welds is evaluated experimentally in two phases.*

    Phase I: Static tension testing has been performed on a total of six #9 sized couplers and a total of fifteen #6 sized couplers. The static tension tests results were evaluated to obtain the 90/95% confidence interval break strength for both coupler sizes. The Phase I test results demonstrate that the 90/95% confidence interval break strength exceeds 125% of the specified yield strength of the reinforcing bar and demonstrate that the rebar or coupler thread is the weak link in the mechanical connection system.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Phase II: Testing was performed to investigate the strength of the coupler weld. A total of 47 samples were tested, 37 #6 sized couplers and 10 #9 sized couplers. The statically-tested samples failed within the coupler body, and demonstrate that the production PJP with fillet weld is stronger than the coupler body. To confirm that the statically-tested sample population is representative of the installed population of coupler welds, the test sample population considered factors such as sample size, weld process, automatic/manual processes, human performance factors, welding procedure specifications, non-destructive examination, fabrication schedule, filler metal, and coupler material.

Safety margin was calculated using the nominal tensile strength and the 90/95% confidence interval test coefficient based on the test samples and is penalized by lower bound failure modes and a finite sample size. The safety margin, or Factor of Safety (FoS), was calculated against both the 125% yield strength of the rebar and against the system strength (i.e., weakest link of the system, rebar or coupler thread). The minimum safety margin with respect to the 125% yield strength of the rebar was calculated to be 2.91 for the #6 sized couplers and 2.26 for the #9 sized couplers. The minimum safety margin with respect to the system strength was calculated to be 2.16 for the #6 sized couplers and 1.68 for the #9 sized couplers.]*

- [*In Section Q1.26.2.2, for the non-conforming partial penetration welds associated with reinforcement bar size #9 C3J couplers installed on carbon steel embedment plates under CA20 at Vogtle Unit 3 and Unit 4 and under CA01 at Vogtle Unit 3 that did not undergo non-destructive examination at the time of fabrication, as identified in Amendment Nos. 86 and 85 for VEGP Units 3 and 4, respectively, the strength and quality of the welds is demonstrated through non-destructive examination and static tension testing of portions of the original production and supplemental, uninstalled populations and through visual examination of the production populations as follows:*

  - *Visual Examination: Coupler welds from the production fabrication populations underwent visual examination by the manufacturers. The manufacturers' visual examinations provided satisfactory results.*

  - *Static Tension Testing: Weldable coupler connections of reinforcing bar to structural steel shall develop 125% of the specified yield strength of the bar in accordance with ACI 349-01, Section 12.14.3.4. The mechanical connection strength requirement is applied to the weld to demonstrate that the coupler weld is stronger than the reinforcing bar strength requirement, thereby satisfying provisions for design limits outlined in AISC N690-1994. To determine that the populations of #9 sized C3J coupler welds on carbon steel embedment plates is adequate in the ability to perform their intended design function, static tension testing of portions of the original production and supplemental, uninstalled populations of welds is evaluated experimentally in two phases.*

    Phase I: Static tension testing has been performed on a total of four #9 sized couplers. The static tension test results were evaluated to obtain the 90/95% confidence interval break strength. The Phase I test results demonstrate that the 90/95% confidence interval break strength exceeds 125% of the specified yield strength of the reinforcing bar and demonstrate that the rebar or thread is the weak link in the mechanical connection system.

    Phase II: Testing was performed to investigate the strength of the coupler weld. A total of 30 #9 sized couplers and 3 #11 sized couplers were tested. The statically-tested samples failed within the coupler body, and demonstrate that the

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

production PJP with fillet weld is stronger than the coupler body. To confirm that the statically-tested sample population is representative of the installed populations of coupler welds, the test sample population considered factors such as sample size, weld process, semi-automatic processes, human performance factors, welding procedure specification, non-destructive examination, filler metal, and coupler material.

Safety margin was calculated using the nominal tensile strength and the 90/95% confidence interval test coefficient based on the test samples and is penalized by lower bound failure modes and a finite sample size. The safety margin, or Factor of Safety (FoS), was calculated against both the 125% yield strength of the rebar and against the system strength (i.e., weakest link of the system, rebar or thread). The minimum safety margin with respect to the 125% yield strength of the rebar was calculated to be 2.08 for the #9 sized couplers. The minimum safety margin with respect to the system strength was calculated to be 1.42 for the #9 sized couplers.

- *Non-destructive examination of portions of the original production and supplemental, uninstalled populations: Magnetic particle examination was performed on 10% of the total populations of welds from the uninstalled original production population and from the uninstalled supplemental population. The magnetic particle examinations provided satisfactory results.*]*

3.8.4.5.3 Design Summary Report

A design summary report is prepared for seismic Category I structures documenting that the structures meet the acceptance criteria specified in Subsection 3.8.4.5.

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 Sections 3.7 and 3.8 provided the following acceptance criteria are met.

- the structural design meets the acceptance criteria specified in Section 3.8
- the seismic floor response spectra meet the acceptance criteria specified in Subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgment to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report.

3.8.4.5.4 Design Summary of Critical Sections

[*The design of representative critical elements of the following structures is described in Appendix 3H.*

- *South wall of auxiliary building (column line 1), elevation 66'-6" to elevation 180'-0" – see subsection 3H.5.1.1 and Figures 3H.5-2 and 3H.5-3*

- *Interior wall of auxiliary building (column line 7.3), elevation 66'-6" to elevation 160'-6" – see subsection 3H.5.1.2 and Figure 3H.5-4*

- *West wall of main control room in auxiliary building (column line L), elevation 117'-6" to elevation 153'-0" – see subsection 3H.5.1.3 and Figure 3H.5-12*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

- *North wall of MSIV east compartment (column line 11 between lines L and M), elevation 117'-6" to elevation 153'-0" – see subsection 3H.5.1.4 and Figure 3H.5-5*
- *Roof slab at elevation 180'-0" adjacent to shield building cylinder – see subsection 3H.5.2.1 and Figure 3H.5-7*
- *Floor slab on metal decking at elevation 135'-3" – see subsection 3H.5.2.2 and Figure 3H.5-6*
- *2'-0" slab in auxiliary building (operations work area (tagging room) ceiling) at elevation 135'-3" – see subsection 3H.5.3.1 and Figure 3H.5-8 (Note: The 'Tagging Room' has been renamed as "Operations Work Area." However, to avoid changing the associated design and analysis documents, this room is referred to as the 'Tagging Room.')*
- *Finned floor in the main control room at elevation 135'-3" – see subsection 3H.5.4 and Figure 3H.5-9*
- *Shield building roof, exterior wall of the PCS water storage tank – see subsection 3H.5.6.3 and Figure 3H.5-11*
- *Shield building roof, interior wall of the PCS water storage tank – see subsection 3H.5.6.2 and Figure 3H.5-11*
- *Shield building roof, tension ring, and air inlet structure – see subsections 3H.5.6 and 3H.5.6.1*
- *Divider wall between the spent fuel pool and the fuel transfer canal – see subsection 3H.5.5.1 and Figure 3H.5-10*
- *Shield building SC cylinder – see subsection 3H.5.7.1, Figure 3H.5-16, and Figures 5 and 6 of APP-GW-GLR-602 (Reference 57)*
- *Shield building SC to RC connection – see Subsection 3H.5.7.2, Figure 3H.5-16, and Figures 1, 2, 3, and 4 of APP-GW-GLR-602 (Reference 57)*]*

3.8.4.5.5 Shield Building Structural Wall Modules

[*The shield building concrete-filled steel module walls are designed for loads such as dead, live, thermal, wind, and safe shutdown earthquake loads as identified in Table 3.8.4-2. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI 349, and supplemented with additional requirements discussed in subsection 3.8.3.5.3 and below. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The steel plate modules are anchored to the reinforced concrete by mechanical connections welded to the steel plate.*]*

Figure 5 in Reference 57 shows the typical design details of the shield building concrete-filled steel module walls. The design of the shield building critical sections is described in Appendix 3H.

3.8.4.5.5.1 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. The reinforcement for in-plane tension and compression combined with out-of-plane moments is calculated using a rectangular concrete stress distribution, in accordance with Sections 10.2 and 10.3 of the ACI 349 code.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

3.8.4.5.5.2 Design for In-Plane Shear

Design for in-plane shear is in accordance with the requirements of ACI 349, Chapters 11 and 14. The steel faceplates are treated as reinforcing steel, contributing as provided in Section 11.10 of ACI 349.

3.8.4.5.5.3 Design for Out-of-Plane Shear

Design for out-of-plane shear is in accordance with the requirements of ACI 349, Chapter 11.

3.8.4.5.5.4 Evaluation for Thermal Loads

Thermal loads are combined with other loads according to the load combinations defined in Table 3.8.4-2. The effect of concrete cracking is considered in the stiffness properties for the concrete elements subjected to the thermal loads. Thermal forces and moments are calculated by multiplying linear elastic thermal stress analysis results by a stiffness reduction ratio α. The stiffness reduction ratio α is calculated as the ratio of the cracked stiffness to the elastic stiffness of the section to be evaluated.

3.8.4.5.5.5 Design of Shear Studs and Tie Bars

[*The shear stud connectors and tie bars are sized to carry the horizontal shear at the junction of the steel faceplates and the concrete infill. The shear stud and tie bar design is such that the shear stud and tie bar strength is sufficient to develop the full yield strength of the steel plate in approximately 3 x the thickness of the wall, or about 9 feet.*

The tie bars provide a structural framework for the modules, maintain the separation between the faceplates, support the modules during transportation and erection, and act as "form ties" between the faceplates when concrete is being placed. The tie bars 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 area and spacing of the tie bars satisfy the requirement for minimum shear reinforcement for beams given in the ACI codes. The connection between the tie bars and the steel faceplates is designed to develop 125% of the specified yield strength of the tie bar.]*

3.8.4.5.5.6 Design of Connections

The shield building steel plate modules are connected to standard reinforced concrete using a connection design satisfying the requirements outlined below. The locations of these connections include the following:

1. At the base of the shield building wall into the concrete below

2. Between SC and RC portions of the shield building wall

3. Where RC floors, roofs, and walls connect to the SC modules.

The steel plate modules are connected to the reinforced concrete by reinforcement or deformed bars directly connected to the modules. These connections are sized using a strength design approach, and the mechanical connection portion of the connection satisfies the requirements of AISC N690. In the area of the connections, the faceplates are thicker (up to 1.0-inch nominal thickness), as necessary, to address loads from the connections. The connection design satisfies the following requirements:

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

- The connections provide for the direct transfer of forces from the RC reinforcing steel to the SC faceplates. Lap splice type connections between dowels extending out of the RC into the SC adjacent to the headed shear studs are not used to connect the modules to the base concrete.

- Loads are transferred directly from the faceplates to the reinforced concrete using reinforcing bars, mechanical connection, and welds.

- The mechanical connections develop 125 percent of specified yield of the connected reinforcement bar.

- Loads that are out of plane of the wall, such as from roofs or walls, are carried through the full thickness of the shield building walls.

Figures 1, 2, 3, and 4 in APP-GW-GLR-602 (Reference 57) show the typical design details for the connection of the wall modules to the reinforced concrete. Figure 5 in Reference 57 shows the typical design details of the shield building concrete-filled steel module walls.

Figure 3H.5-7 and Figure 7 in Reference 57 show typical auxiliary building reinforced concrete roof connections to the shield building SC wall. The details of the connections between the auxiliary building roof and the shield building wall vary because of loads on the connection and the orientation of the wall to the roof reinforcement arrangement as outlined in the notes on Figure 3H.5-7 and in Section 4 of Reference 57. Figure 3H.5-7 also shows typical connections between the auxiliary building walls and the shield building SC wall modules. The connection elements used and the design details vary because of the orientation of the auxiliary building wall to the shield building wall. These connection design details satisfy the requirements of AISC N690 and the requirements identified above.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

This subsection contains information relating to the materials, quality control program, and special construction techniques used in the construction of the other seismic Category I structures, as well as the containment internal structures. An edition of the referenced specifications applicable after the start of construction or procurement activities will be used.

3.8.4.6.1 Materials

3.8.4.6.1.1 Concrete

[*The compressive strength of concrete used in the seismic Category I structures and containment internal structures is f_c' = 4000 psi except as noted in the following. For the nuclear island basemat (the nominal 6 ft. thick foundation described in Subsection 3.8.5.1) the compressive strength of concrete is f_c' = 5000 psi. For the SC composite portion of the shield building structure including the connection region below the SC/RC interface and the shield building roof, the compressive strength of concrete is f_c' = 6000 psi.*]* The test age of concrete containing pozzolan is up to 56 days. The test age of concrete without pozzolan is up to 28 days. Concrete is mixed, batched, and placed according to Reference 6, Reference 7, and ACI-349.

Portland cement conforms to Reference 8, Type II. Certified copies of mill test reports showing that the chemical composition and physical properties conform to the specification are obtained for each cement delivery.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Aggregates conform to Reference 9. The fineness modulus of fine aggregate (sand) is not less than 2.3, nor more than 3.1, per Reference 9. In at least four of five successive test samples, such modulus is not allowed to vary more than 0.20 from the moving average established by the last five tests. Coarse aggregates may be rejected if the loss from the Los Angeles abrasion test, Reference 10, using Grading A or Reference 11, exceeds 40 percent by weight at 500 revolutions. Acceptance of source and aggregates is based on the tests specified in Table 3.8.4-3 as applicable based on type of aggregate and ACI-349 requirements.

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

The concrete contains an air entraining admixture when required by ACI-349 and may contain pozzolans and a water-reducing admixture. A viscosity modifying admixture (VMA) may be used in self-consolidating concrete (SCC) for adjusting the viscosity and improving its stability as outlined in ACI 237R, “Self-Consolidating Concrete.” Admixtures, except pozzolan, are stored in liquid solution.

Admixture types A, B, C, and F are used as noted herein to produce quality concrete. The admixtures shall be proven compatible with all constituents and other admixtures in mix prior to use in production of concrete. Furthermore, all admixtures shall be measured, introduced into the mix, and used in strict accordance to the manufacturer’s recommendations.

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

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

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

Air entraining admixture conforms to Reference 16.

Water-reducing admixtures conform to Reference 17 and are of types A or F. A type A admixture may be used to increase the slump for conventional mixes (may use type F, optional). A type F admixture shall be used for SCC admixtures.

Retarding admixtures conform to Reference 17 and are of type B. Type B admixtures may be used to retard set times for conventional and SCC mixes (i.e., for mass concrete).

Accelerating admixtures conform to Reference 17 and are type C. Type C admixtures may be used to accelerate the set time for conventional and SCC mixes (i.e., for thin members placed in cold weather).

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

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

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

Proportioning of the concrete mix is in accordance with Reference 18 and Option B of Reference 6.

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

Forms for concrete are designed as recommended in ACI 347.

3.8.4.6.1.2 Reinforcing Steel

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

In areas where reinforcing steel splices are necessary and lap splices are not practical, mechanical connections (e.g., threaded splices, swaged sleeves or cadwelds) are used.

[*Headed reinforcement meeting the requirements of ASTM A970 (Reference 49) is used where mechanical anchorage is required,*]* such as for shear reinforcement in the nuclear island basemat and in the exterior walls below grade.

As stated in Subsection 3.4.1.1.1, seismic Category I structures that are located below grade elevation are protected against flooding by a waterproofing system. This, in conjunction with the ACI 349 Code requirements for concrete cover for exposure to earth or weather for the reinforcing steel, provides sufficient protection for the reinforcing steel. Therefore, the use of coated reinforcing steel is not planned.

3.8.4.6.1.3 Structural Steel

Basic materials used in the structural and miscellaneous steel construction conform to the ASTM standards listed in Table 3.8.4-6. The table includes the major structural steel materials of construction. Materials used in limited applications and minor amounts may not be included. [*Additional information on structural and reinforcing steel for the shield building is provided in Table 1 of APP-GW-GLR-602 (Reference 57).*]*

3.8.4.6.1.4 Masonry Walls

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

3.8.4.6.2 Quality Control

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

3.8.4.6.3 Special Construction Techniques

Construction techniques for the structural modules are the same as special construction techniques for the containment internal structures discussed previously in Subsection 3.8.3.6.1. Design-specific tolerances for alignment of the faceplates at welds connecting the shield building structural wall modules permit up to 1/4" offset in the alignment of the faceplates. The separation between the faying surfaces of the shield building wall modules butt joints landing on a backing bar is allowed up to 1/4". These design-specific tolerances modify plate alignment tolerances and faying surface separation requirements specified in AWS D1.1 (subsections 5.22.1.1 and 5.22.3) and are used in

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

conjunction with a weld configuration that is designed for the larger alignment offset and the larger backing bar gap. The shield building wall design using the modified weld joint is in conformance with applicable requirements of ACI 349 and AISC N690.

Where out-of-plane shear reinforcement is provided within the radial/circumferential reinforcement pattern for the main steam line anchor in auxiliary building Wall 11 (Figure 3H.5-5), tolerance on shear reinforcement is specified to maintain the maximum spacing requirements of ACI 349-01, and the minimum spacing requirements of ACI 318-11 for headed reinforcement.

3.8.4.7 Testing and In-Service Inspection Requirements

Structures supporting the passive containment cooling water storage tank on the shield building roof will be examined before and after first filling of the tank.

- The boundaries of the passive containment cooling water storage tank and the shield building roof above the tension ring at the intersection of the shield building roof and the shield building cylinder will be inspected visually for excessive concrete cracking before and after first filling of the tank. Any significant concrete cracking will be documented and evaluated in accordance with ACI 349.3R-96 (Reference 50). The structure around the passive containment cooling water storage tank will be inspected for water leaking out of the tank through the concrete.

- The vertical elevation of the passive containment cooling water storage tank relative to the top of the shield building cylindrical wall at the tension ring will be measured before and after first filling. The change in relative elevation will be compared against the predicted deflection.

- A test will be performed to measure the leakage from the passive containment cooling water storage tank based on measuring the water flow out of the leak chase collection system.

- A report will be prepared summarizing the test and evaluating the results.

During the operation of the plant, the condition of these structures should be monitored by the Combined License holder to provide reasonable confidence that the structures are capable of fulfilling their intended functions. The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

3.8.4.8 Construction Inspection

Construction inspection is conducted to verify the concrete wall thickness and quantity of concrete reinforcement. The construction inspection includes concrete wall thickness and reinforcement expressed in units of in^2/ft (linear length) equivalent when compared to standard reinforcement bar sections. Inspections will be measured at applicable sections excluding designed openings or penetrations. Inspections will confirm that each applicable section provides the minimum required reinforcement, steel plate thickness, and concrete thickness. The minimum required reinforcement, steel plate thickness, and concrete thickness represent the minimum values to meet the design basis loads. Appendix 3H also indicates the reinforcement provided which may exceed the minimum required reinforcement for the following reasons:

- Structural margin
- Ease of construction
- Use of standardized reinforcement sizes and spacing

A shield building construction mockup program will be used to build full-scale replicas of areas of the shield building that present critical construction areas identified as challenging areas of construction. These mockups provide an opportunity to apply and evaluate alternate and innovative construction and inspection methods and procedures. The fabrication, assembly, and erection of a full-scale mockup provides the accurate physical representation needed to evaluate true working conditions, physical configuration, accessibility, and quality control issues that may be encountered in construction. Construction practices review and examination of the mockups will be used to confirm the adequacy of construction means, methods, and procedures. If defects are found, the procedures will be revised and the mockup repeated until the required result is obtained. The major tasks that will be performed on each mockup include the following: field performance testing of the quality of concrete mixes, methods of concrete placement, inspections and surveillance, and post-placement activities.

In addition to these important process control tasks performed on the mockups, an inspection program will be undertaken on the AP1000 construction site mockups that uses the enhanced shield building design. Both visual inspection and non-destructive examination (NDE) will be performed for assessing defects that may impact structural integrity, such as crack distress, deterioriation caused by honeycomb, voids, and delaminations. The NDE inspection of the mockups will be performed at the first construction site of an AP1000 plant with an enhanced shield building design, and mockups applicability will be evaluated for the subsequent AP1000 site. This is to demonstrate the construction quality process control for concrete placement, and develop and document insights and requirements for corrective action, if required, to be used in the construction inspection program for all AP1000 plants.

3.8.5 Foundations

3.8.5.1 Description of the Foundations

The nuclear island structures, consisting of the containment building, shield building, and auxiliary building are founded on a common 6-foot-thick, cast-in-place, reinforced concrete basemat foundation. The top of the foundation is at elevation 66′-6″.

The depth of overburden and depth of embedment are given in Subsection 2.5.4.5.

A description of the safety-related backfill, which supports Category I structures, is given in Subsection 2.5.4.5.

[*The turbine building and annex building are structurally separated from the nuclear island structures by a 2-inch gap at and below the grade. A 3-inch minimum gap is provided above grade.*]* This provides space to prevent interaction between the nuclear island structures and the adjacent seismic Category II structures during a seismic event. The maximum relative seismic displacement between the roof of the nuclear island and the turbine and annex buildings is less than 2 inches. This results in a clearance (gap) between buildings greater than 1 inch during a seismic event. Therefore, there are no interactions between the nuclear island and adjacent seismic Category II buildings during a seismic event. The radwaste building[1] is separated from the nuclear island by a 2-inch gap at and below grade and a 4-inch gap above grade. Figure 3.8.5-1 shows the foundations for the nuclear island structures and the adjoining structures.

Resistance to sliding of the concrete basemat foundation is provided by passive soil pressure and soil friction. This provides the required factor of safety against lateral movement under the most stringent loading conditions.

1. It should be noted that an evaluation of the radwaste building was made to consider its impact on the nuclear island or collapse in the safe shutdown earthquake; it was concluded that it would not impair the integrity of the nuclear island (see Subsection 3.7.2.8.2).

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

For ease of construction, the foundation is built on a mud mat. The mud mat is lean, nonstructural concrete and rests upon the load-bearing soil. Waterproofing requirements are described in Subsection 3.4.1.1.1.

The scope of the LWA foundation work includes: placing the mud mats, water proofing membrane, concrete forms, drains and other items necessary to prepare the Nuclear Island base slab for the first concrete pour.

After backfill beneath the NI (Nuclear Island) has been placed and compacted to roughly the required elevation for the first mud mat, the construction of the retaining wall will begin. The retaining wall will be a vertical mechanically-stabilized earth (MSE) wall with smooth-faced concrete panels. This wall will function as both a retaining wall as the backfill outside the NI volume is brought up to plant grade and as the exterior concrete form for the outer walls of the NI. Subsection 2.5.4.5, Excavation and Backfill, provides additional information on the backfill and MSE wall.

The construction of the MSE wall begins with installation of a concrete footer. The top surface of the MSE wall footer will be installed below the bottom elevation of the first mud mat. The size and reinforcement for the concrete footer will be as required by the designer of the MSE wall. The MSE wall footer is a relatively thin concrete structure that provides a stable, level surface for construction of the MSE wall. It provides no structural support for the mud mats or the NI itself.

The first course of the MSE wall will be placed on top of the footer at the surveyed locations required to outline the NI footprint. Inspections will be performed as required to assure that the outer dimensions of the NI are properly set.

Backfill around the outer sides of the MSE wall will commence as required by the designer of the MSE wall, with the standard large compaction equipment being used away from the wall, and smaller equipment providing the required compaction at the edges of the wall. During backfill placement and compaction, the backfill surface will be sloped away from the NI to drain surface water away from the NI excavation volume. Additional courses of the MSE wall will be added until final plant grade is reached.

In parallel with the construction of the MSE wall, work within the NI footprint will continue. Temporary features to provide removal of surface water within the confined area of the NI will be installed as required. These features may include plastic sheeting, temporary sumps and pumps. In addition, the surface may be sloped to provide adequate drainage.

After the first few courses of the MSE wall have been placed, the backfill within the NI volume will be prepared for placement of the mud mat. Temporary drainage features will be removed, and material will be removed or added as required to establish the final elevation for the mud mat placement.

The first mud mat will consist of a 6-inch layer of non-reinforced concrete placed uniformly within the confines of the MSE wall. No additional formwork will be required. When this lower mud mat slab has reached the specified strength, a layer of waterproof membrane will be applied to the entire top of the slab, and extended vertically up the face of the MSE wall surface. The top portion of the mud mat slab (also a 6-inch layer of non-reinforced concrete) will then be placed, sandwiching the waterproof membrane. Additional detail concerning the waterproof membrane is provided in following Subsection 3.8.5.1.1. Figure 2.5-390 provides an illustration of the location of the waterproof membrane. Rebar and foundation embedments are not incorporated in either of these mud mats; therefore installation of such elements will not puncture the waterproofing membrane.

3.8.5.1.1 Waterproof Membrane

For Vogtle Electric Generating Plant (VEGP) Units 3 and 4 an alternate waterproofing system is presented as a departure from the DCD design. The alternate waterproofing system is an elastomeric “spray-on” waterproof membrane. The membrane is applied as a high-viscosity liquid that cures after exposure to air. This material may be applied by brush, roller or airless spray equipment.

Prior to procurement of the membrane material, a qualification program will be developed to demonstrate that the selected material will meet the waterproofing and friction requirements. This qualification program will address, as a minimum, the following:

- chemical properties of the membrane material,
- physical properties of the membrane material,
- surface finish requirements for the lower mudmat, and
- installation procedures necessary to achieve the required properties and coefficients of friction.

The qualification program will include testing to demonstrate that the ITAAC design commitment in Table 3.8-201 for friction coefficient has been met. Testing methods will simulate field conditions to demonstrate that a minimum 0.7 coefficient of friction is achieved by the mudmat waterproof membrane structural interface. A technical report will be provided for the ITAAC to document the basis for determining that the material will meet the required friction factor. Application procedures will be developed based on the results of qualification testing to assure that the conditions and assumptions of the qualification tests are maintained during product application.

Based upon the qualification program requirements, it is concluded that the installed waterproof membrane will provide a level of protection from external flooding and meet the coefficient of friction at the waterproof membrane-mudmat interface that is consistent with that of the existing DCD design.

The elastomeric waterproof membrane will be applied to the entire surface of the lower mudmat and the inner face of the MSE wall. Final thickness of the membrane will be specified based on the physical properties of the selected material but is expected to be on the order of 80 to 120 mils. The membrane may be applied in multiple coats to achieve the required thickness.

The surface of both the mudmat and the MSE wall will be prepared in accordance with procedures that are consistent with the surface preparation requirements determined during the material qualification testing program. At the transition between the lower mudmat and the MSE wall, a small transition (chamfer or fillet) between the mudmat and wall will be provided to allow a smooth transition for the membrane.

The surface of the MSE wall will be prepared as necessary to assure that the waterproof coating application can bridge the small gaps and corners of the MSE blocks. This preparation process will likely include attaching a geo-textile material to the wall prior to application of the membrane material. It should be noted that the cured membrane has a degree of flexibility which allows it to accommodate thermal expansion and other minor movements between substrate members.

The application procedures will address all aspects of the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repair procedures.

The final mudmat will be placed on top of the waterproof membrane. Procedures will address inspection and testing as required to assure that the membrane surface will meet the required coefficient of friction.

The gap between the MSE wall and the NI wall is sealed to prevent water intrusion.

3.8.5.2 Applicable Codes, Standards, and Specifications

The applicable codes, standards, and specifications are described in Subsection 3.8.4.2.

3.8.5.3 Loads and Load Combinations

Loads and load combinations are described in Subsection 3.8.4.3. The loads and load combinations used in the analysis are considered to be part of the method of evaluation. As described in Subsection 3.8.2.1.2, the bottom head of the steel containment vessel is the same as the upper head and is capable of resisting the containment internal pressure without benefit of the nuclear island basemat. However, containment pressure loads affect the nuclear island basemat since the concrete is stiffer than the steel head. The containment design pressure is included in the design of the nuclear island basemat as an accident pressure in load combinations 5, 6, and 7 of Table 3.8.4-2. In addition to the load combinations described in Subsection 3.8.4.3, the nuclear island is checked for resistance against sliding and overturning due to the safe shutdown earthquake, winds and tornados, and against flotation due to floods and groundwater according to the load combinations presented in Table 3.8.5-1.

The effect of the groundwater level has been studied extensively. For the AP1000, Westinghouse has performed a time history analysis using a saturated and unsaturated soft-medium soil profile (Poisson's ratio = 0.35) and compared the floor response spectra of the two analyses. Generic SSI analyses for the AP1000 assume the water table to be at grade level with saturated soil properties supporting the nuclear island. The unsaturated soil profile was produced where the water table was assumed to be well below the nuclear island.

The results of this analysis concluded that the depth of the water table used for SSI analyses has a negligible effect on the floor response spectra at the key nodes. Since the floor response spectra differences between the two models are negligible, no additional analyses are required to compare member forces or deformations.

3.8.5.4 Design and Analysis Procedures

The seismic Category I structures are concrete, shear-wall structures consisting of vertical shear/bearing walls and horizontal floor 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 walls then transmit the loads to the basemat. The walls also provide stiffness to the basemat and distribute the foundation loads between them.

The design of the basemat consists primarily of applying the design loads to the structures, calculating shears and moments in the basemat, and determining the required reinforcement. For a site with hard rock below the underside of the basemat vertical loads are transmitted directly through the basemat into the rock. Horizontal loads due to seismic are distributed on the underside of the basemat resulting primarily in small membrane forces in the mat. The 6-foot-thick basemat is designed for the upward hydrostatic pressure due to groundwater reduced by the downward deadweight of the mat.

[*Seismic loads for the evaluation of the basemat of the Nuclear Island are developed from the results of the global seismic analyses on hard rock. They are specified as equivalent static accelerations.*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

The equivalent static accelerations used in the non-linear design analyses of the nuclear island basemat are evaluated for the revised design with the enhanced shield building by comparing total base reactions and bearing pressures in a linear analysis using these equivalent static accelerations to those from a dynamic analysis of the nuclear island 3D finite element model. A time history fixed base analysis of the model is performed using time history inputs that envelope the basemat response given by the 3D SASSI analyses at the corners and centers of the basemat for all the specified generic soil cases.]*

The basemat reactions for the equivalent static analyses compare well against those of the "all soils" time history. [*Bearing pressures are calculated from the basemat reactions assuming a rigid basemat for dead live and seismic loads. Seismic loads are considered using the 1.0, 0.4, 0.4 combination method.*]* The bearing pressures resulting from the equivalent static accelerations are similar to those due to the "all soils" time history analysis demonstrating the adequacy of the equivalent static accelerations applied in the basemat analyses.

3.8.5.4.1 Analyses for Loads during Operation

The analyses of the basemat use the three-dimensional ANSYS finite element models of the auxiliary building and containment internal structures, which are described in Subsections 3G.2.1.1 and 3G.2.1.2 and shown in Figures 3G.2-1 and 3G.2-2. The model considers the interaction of the basemat with the overlying structures and with the soil. Provisions are made in the model for two possible uplifts. One is the uplift of the containment internal structures from the lower basemat. The other is the uplift of the basemat from the soil.

The three-dimensional finite element model of the basemat includes the structures above the basemat and their effect on the distribution of loads on the basemat. The finite element model of the basemat is shown on sheet 1 of Figure 3.8.5-2.

The subgrade is modeled with one vertical spring and two horizontal springs at each node of the basemat. The vertical springs act in compression only. The horizontal springs are active when the vertical spring is closed and inactive when the vertical spring lifts off. The analyses of the basemat accounted for the range of soil sites described in Section 2.5. Horizontal bearing reactions on the side walls below grade are conservatively neglected.

The nuclear island basemat below the containment vessel, and the containment internal structures basemat above the containment vessel, are simulated with solid tetrahedral elements. Nodes on the two basemats are connected with spring elements normal to the theoretical surface of the containment vessel.

Normal and extreme environmental loads and containment pressure loads are considered in the analysis. The normal loads include dead loads and live loads. Extreme environmental loads include the safe shutdown earthquake.

Dead loads are applied as inertia loads. Live loads and the safe shutdown earthquake loads are applied as concentrated loads on the nodes. The safe shutdown earthquake loads are applied as equivalent static loads using the assumption that while maximum response from one direction occurs, the responses from the other two directions are 40 percent of the maximum. Combinations of the three directions of the safe shutdown earthquake are considered.

Linear analyses are performed for all specified load combinations assuming that the soil springs can take tension. Critical load cases are then selected for non-linear analyses with basemat liftoff based on the results of the linear cases. The results from the analysis include the forces, shears, and moments in the basemat; the bearing pressures under the basemat; and the area of the basemat that

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is uplifted. Reinforcing steel areas are calculated from the member forces for each load combination case.

The required reinforcing steel for the portion of the basemat under the auxiliary building and under the shield building is determined by considering the reinforcement envelope for the full non-linear iteration of the most critical load combination cases. Additional reinforcement is provided in the design of the 6′ mat for soil sites such that the basemat can resist loads 20 percent greater than the demand calculated by the equivalent static acceleration analyses on uniform soil springs. This increase accommodates potential site specific lateral variability of the soil investigated separately in a series of parametric studies. Figure 3.8.5-3 shows the basemat reinforcement.

3.8.5.4.2 Analyses of Settlement During Construction

Construction loads are evaluated in the design of the nuclear island basemat. This evaluation is performed for soil sites meeting the site interface requirements of Subsection 2.5.4 at which settlement is predicted to be maximum. In the expected basemat construction sequence, concrete for the mat is placed in a single placement. Concrete may be placed below the sumps and elevator pits, if determined to enhance constructability, prior to the remaining single pour of the nuclear island basemat. The placement includes the first 6 feet of the thicker basemat below the containment vessel and shield building, but excludes the central zone directly below the bottom of the containment vessel. Construction continues with a portion of the shield building foundation and containment internal structure and the walls of the auxiliary building. The critical location for shear and moment in the basemat is around the perimeter of the shield building. Once the shield building and auxiliary building walls are completed to elevation 82′-6″, the load path changes and loads are resisted by the basemat stiffened by the shear walls.

The analyses account for the construction sequence, the associated time varying load and stiffness of the nuclear island structures, and the resulting settlement time history. To maximize the potential settlement, the analyses consider a 360 feet deep soft soil site with soil properties consistent with the soft soil case described in Subsection 2A.2. Two soil profiles are analyzed to represent limiting foundation conditions, and address both cohesive and cohesionless soils and combinations thereof:

- A soft soil site with alternating layers of sand and clay. The assumptions in this profile maximize the settlement in the early stages of construction and maximize the impact of dewatering.

- A soft soil site with clay. The assumptions maximize the settlement during the later stages of construction and during plant operation.

The analyses focus on the response of the basemat in the early stages of construction when it could be susceptible to differential loading and deformations. As subsequent construction incorporates concrete shear walls associated with the auxiliary building and the shield building, the structural system significantly strengthens, minimizing the impact of differential settlement. The displacements, and the moments and shear forces induced in the basemat are calculated at various stages in the construction sequence. These member forces are evaluated in accordance with ACI 349 using the load factors given in Table 3.8.4-2. Three construction sequences are examined to demonstrate construction flexibility within broad limits.

- A base construction sequence which assumes no unscheduled delays. The site is dewatered and excavated. Concrete for the basemat is placed. Concrete for the exterior walls below grade is placed after the basemat is in place. Exterior and interior walls of the auxiliary building are placed in 16 to 18-foot lifts.

- A delayed shield building case which assumes a delay in the placement of concrete in the shield building while construction continues in the auxiliary building. This bounding case maximizes tension stresses on the top of the basemat. The delayed shield building case assumes that no additional concrete is placed in the shield building after the pedestal for the containment vessel head is constructed. The analysis incorporates construction in the auxiliary building to elevation 117′-6″ and filling the CA20 module with concrete to elevation 135′-3″, and thereafter assumes that construction is suspended.

- A delayed auxiliary building case which assumes a delay in the construction of the auxiliary building while concrete placement for the shield building continues. This bounding case maximizes tension stresses in the bottom of the basemat. There are two construction sequence limitations from this case:

  – The first construction sequence limitation assumes a delayed auxiliary building case in which construction can continue in the shield building to elevation 139′-6″. Shield building construction may proceed up to 139′-6″ with the following requirements and exceptions:

    - Auxiliary building walls must be completed up to an elevation of 82′-6″. Completion of auxiliary building 82′-6″ floors is not required. Completion of module CA20 is not required provided that the steel portions are constructed.

    - Wall N is an exception to the completion of the auxiliary building walls to elevation 82′-6″. This wall does not have to be constructed in order for the shield building to proceed up to 139′-6″.

    - Construction of module CA01 is limited to steel only.

    - No concrete is placed in the SC shield building walls above elevation 131′-6″.

    - Construction in the RC portion of the shield building between AZ 181.98° and AZ 340.00° may proceed to an elevation of 141′-6″ provided that no concrete is placed above elevation 100′-0″.

    - Shield building construction may proceed to an elevation of 149′-6″ between AZ 181.98° and AZ 183.01° to permit installation of the RC/SC connection panel. No concrete is placed in this panel.

    - Containment vessel rings 1 and 2 may be placed.

    - Modules CA02, CA03, CA04, and CA05 may be fully constructed.

  – The second construction sequence limitation assumes the auxiliary building is complete to 82′-6″ excluding floors, and that the concrete has been placed in module CA20 walls to elevation 128′-0″ except for the north end of the east and west walls of the spent fuel pool, which has been placed to elevation 87′-3″. Construction in the shield building may proceed up to elevation 159′-6″, with the following requirements and exceptions:

    - Shield building concrete elevation must remain at or below elevation 149′-6″.

    - Wall N is an exception to the completion of the auxiliary building walls to elevation 82′-6″. This wall does not have to be constructed in order for the shield building concrete to continue to 149′-6″.

    - The containment vessel ring 3 and top head are not placed.

- Module CA01 may be fully constructed.

– Resumption in construction of the shield building above elevation 159′-6″ can proceed once the auxiliary building is completed to elevation 100′-0″, including all walls and 100′-0″ elevation floors.

For the base construction sequence, the largest basemat moments and shears occur at the interface with the shield building before the connections between the auxiliary building and the shield building are credited. Once the shield building and auxiliary building walls are completed to elevation 82′-6″, the load path for successive loads changes and the loads are resisted by the basemat stiffened by the shear walls. Dewatering is discontinued once construction reaches grade, resulting in the rebound of the subsurface.

Of the three construction scenarios analyzed, the delayed auxiliary building case results in the largest demand for the bottom reinforcement in the basemat. The delayed shield building results in the largest demand for the top reinforcement in the basemat. The analyses of the three construction sequences demonstrate the following:

- The design of the basemat and superstructure accommodates the construction-induced stresses considering the construction sequence and the effects of the settlement time history.

- The design of the basemat can accommodate delays in the shield building so long as the auxiliary building construction is suspended at elevation 117′-0″. Resumption in construction of the auxiliary building can proceed once the shield building is advanced to elevation 100′-0″.

- The design of the basemat can accommodate delays in the auxiliary building so long as the shield building construction is suspended at the elevation limits as previously discussed in the delayed auxiliary building case.

- After the structure is in place and cured to elevation 100′-0″, the basemat and structure act as an integral 40 foot deep structure and the loading due to construction above this elevation is not expected to cause significant additional flexural demand with respect to the basemat and the shield building concrete below the containment vessel. Accordingly, there is no need for placing constraints on the construction sequence above elevation 100′-0″.

The site conditions considered in the evaluation provide reasonable bounds on construction induced stresses in the basemat. Accordingly, the basemat design is adequate for practically all soil sites and it can tolerate major variations in the construction sequence without causing excessive deformations, moments and shears due to settlement over the plant life.

The analyses of alternate construction scenarios considered the softest material properties satisfying the shear wave velocity limit of 1000 feet per second. These analyses show that member forces in the basemat are acceptable subject to the limits shown below on the relative level of construction of the buildings. Construction of the AP1000 will satisfy the limits shown below, or a site-specific analysis of settlement and member forces will be completed. These limits do not apply to AP1000 units with a soil profile where the shear wave velocity exceeds 7500 feet per second.

Prior to completion of the shield building at elevation 82′-6″:

- Concrete may not be placed above elevation 117′-6″ in the auxiliary building, except in the CA20 structural module, where it may be placed to elevation 135′-3″.

Member forces in the basemat considering settlement during construction differ from those obtained from the analyses on uniform elastic soil springs described in Subsection 3.8.5.4.1.

Although the bearing pressures at the end of construction are similar in the two analyses, the resulting member forces differ due to the progressive changes in structural configuration during construction. The design using the results of the analyses of Subsection 3.8.5.4.1 provides sufficient structural strength to resist the specified loads including bearing reactions on the underside of the basemat. The member forces in these analyses are those due to primary externally applied loads and do not consider secondary stresses and strains locked in during early stages of construction. A confirmatory evaluation was performed to demonstrate that the member forces due to design basis loads, including locked-in forces due to construction settlement, remain within the capacity of the section. The evaluation was performed for critical locations which were selected as locations where the effect of locked in member forces were judged to be most significant.

The governing scenario is the case with a delay in the auxiliary building construction for the soft soil site with alternating layers of sand and clay. The delay is postulated to occur just prior to the stage where the auxiliary building walls are constructed. Member forces at the end of construction are calculated considering the effects of settlement during construction. The difference in these member forces from those calculated for dead load in the analyses on soil springs are added as additional dead loads in the critical safe shutdown earthquake load combination.

The member forces for the load combination of dead load plus safe shutdown earthquake, including the member forces locked-in during various stages of plant construction, are within the design capacity for the five critical locations. The evaluation demonstrates that the member forces including locked-in forces calculated by elastic analyses remain within the capacity of the section.

3.8.5.4.3 Design Summary Report

A design summary report is prepared for the basemat documenting that the structures meet the acceptance criteria specified in Subsection 3.8.5.5.

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 Sections 3.7 and 3.8 provided the following acceptance criteria are met.

- The structural design meets the acceptance criteria specified in Section 3.8
- The seismic floor response spectra meet the acceptance criteria specified in Subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgment to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report.

3.8.5.4.4 Design Summary of Critical Sections

The basemat is designed to meet the acceptance criteria specified in Subsection 3.8.4.5. Two critical portions of the basemat are identified below together with a summary of their design. The boundaries are defined by the walls and column lines which are shown in Figure 3.7.2-12 (sheet 1 of 12). Table 3.8.5-3 shows the reinforcement required and the reinforcement provided for the critical sections.

[*Basemat between column lines 9.1 and 11 and column lines K and L*

This portion of the basemat is designed as a two way slab with the shorter directions spanning a distance of 23′6″ between the walls on column lines K and L. The slab is continuous with the adjacent slabs to the east and west. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at the walls. The basemat is designed for the member forces from the analyses]* described in Subsection 3.8.5.4.1. [*The top and bottom reinforcement in the east west direction of span are equal. The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement are shown in Figure 3H.5-3.*

Basemat between column lines 1 and 2 and column lines K-2 and N

This portion of the basemat is designed as a two way slab with the shorter direction spanning a distance of 22′0″ between the walls on column lines 1 and 2. The slab is continuous with the adjacent slabs to the north and with the exterior wall to the south. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at wall 2. The basemat is designed for the member forces from the analyses on uniform soil springs]* described in Subsection 3.8.5.4.1. [*The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement are shown in Figure 3H.5-3.*]*

Figure 3.8.5-3 shows minimum design reinforcement for the concrete in the areas defined. This location information does not define the exact length or position of the reinforcement bars. The detailed reinforcement design, including requirements for detailing and concrete cover, is in conformance with ACI-349 requirements for foundations. The reinforcement arrangement implemented may have a greater amount of reinforcement than that shown in the figure. The size and spacing of the reinforcement bars may vary from that shown as long as the reinforcement area provided is equal to or greater than the area for the size and spacing identified in the figure. Incorporation of embedments, drains and other piping, support structures, and other obstructions may result in local variations from the reinforcement arrangement shown. 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 Sections 3.7 and 3.8 provided the following acceptance criteria are met.

- The structural design meets the acceptance criteria specified in Section 3.8.
- The amplitude of the seismic floor response spectra does not exceed the design basis floor response spectra by more than 10 percent.

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgment to performance of a revised analysis and design.

3.8.5.5 Structural Criteria

The analysis and design of the foundation for the nuclear island structures are according to ACI-349 with margins of structural safety as specified within it. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are described in Section 2.5. The minimum required factors of safety against sliding, overturning, and flotation for the nuclear island structures are given in Table 3.8.5-1.

[*The design and construction of anchors and embedments conform to the procedures and standards of Appendix B to ACI 349-01*]* and are in conformance with the regulatory positions of NRC

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Regulatory Guide 1.199, Revision 0. [*Alternative requirements to ACI 349, Appendix B apply to the anchoring of headed shear reinforcement for the basemat (see Subsection 3.8.4.4.1).*]*

[*The basemat below the auxiliary building is designed for shear in accordance with the following supplemental provisions which are based on requirements for continuous deep flexural members in ACI 349-01. As permitted by paragraph 11.5.5.1 of ACI 349-01, shear reinforcement is not provided when the factored shear force, V_u, is less than one half of the shear strength provided by the concrete, φV_c.*

- *The design for shear is based on 11.1 through 11.5 of ACI 349-01 except that the critical section measured from the face of the support is taken at a distance of 0.15 l_n.*
- *Shear strength, V_n, is not taken greater than $8\sqrt{f_c'}\, b_w d$ when l_n/d is less than 2.*
  - *When l_n/d is between 2 and 5, $V_n = 2/3\ (10 + l_n/d)\ \sqrt{f_c'}\, b_w d$*
- *Minimum vertical shear reinforcement is provided in each bay. The area of vertical shear reinforcement, A_v, is not less than $0.0015\ b_w s$*
  - *Spacing of shear reinforcement, s, based on provisions in Paragraph 11.5.4.1 does not exceed d/2, nor 24 in.*
- *Shear reinforcement required at the critical section is used throughout the span.*

The terms φ, d, l_n, V_c, A_v, b_w, s, and f_c' are defined in ACI 349.]*

3.8.5.5.1 Nuclear Island Maximum Bearing Pressures

The foundation will be demonstrated to be capable of withstanding the bearing demand from the nuclear island as described in Subsection 2.5.4.10.1.

3.8.5.5.2 Flotation

The factor of safety against flotation of the nuclear island is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{D}{(F \text{ or } B)}$$

where:

F.S.	=	factor of safety against flotation
D	=	total weight of structures and foundation
F	=	buoyant force due to the design basis flood
B	=	buoyant force due to high ground water table

3.8.5.5.3 Sliding

The factor of safety against sliding of the nuclear island during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

$$F.S. = \frac{F_S}{F_H}$$

where:

F.S. = factor of safety against sliding from tornado or design wind

F_S = shearing or sliding resistance at bottom of basemat

F_H = maximum lateral force due to active soil pressure, including surcharge, and tornado or design wind load

The factor of safety against sliding of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{F_S}{F_D}$$

where:

F.S. = factor of safety against sliding from a safe shutdown earthquake

F_S = shearing or sliding resistance at bottom of basemat

F_D = seismic force from safe shutdown earthquake

The sliding resistance is based on the friction force developed between the basemat and the foundation with a static coefficient of friction of 0.55. The governing friction value in the soil below the mudmat has an angle of internal friction of 35°. The effect of buoyancy due to the water table is included in calculating the sliding resistance. Passive soil pressure resistance is not included in the equations above because passive pressure is not considered for sliding stability. Since there is no passive pressure considered, active and overburden soil pressures are also not considered.

3.8.5.5.4 Overturning

The factor of safety against overturning of the nuclear island during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{M_R}{M_O}$$

where:

F.S. = factor of safety against overturning from tornado or design wind

M_R = resisting moment

M_O = overturning moment of tornado or design wind

The factor of safety against overturning of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is evaluated using the time history analysis assuming overturning about the edge of the nuclear island at the bottom of the basemat. The factor of safety is defined as follows:

$$F.S. = (M_R)/(M_O + M_{AO})$$

where:

F.S. = factor of safety against overturning from a safe shutdown earthquake

M_R = nuclear island's resisting moment against overturning

M_O = maximum safe shutdown earthquake induced overturning moment acting on the nuclear island, applied as a static moment

M_{AO} = Moment due to lateral forces caused by active and overburden pressures

The resisting moment is equal to the nuclear island dead weight, minus buoyant force from ground water table, multiplied by the distance from the edge of the nuclear island to its center of gravity. The overturning moment is the maximum moment about the same edge from the time history analyses of the nuclear island NI20 model described in Subsection 3.7.2 and Appendix 3G.2. Resistance moment due to passive pressure is not included in the equation above because passive pressure is not considered for overturning stability.

3.8.5.5.5 Seismic Stability Analysis

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I, and the west side of the shield building at each time step of the seismic time history. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock (HR) case to the UBSM soil case, and the SM soil case. These factors are used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and, therefore, are not considered.

A non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 3.8.5.5.6 and Appendix 3G.2.2.5). This 2D non-linear model is for the east-west direction. There is no need to consider the north-south direction since the nuclear island deflections calculated to maintain a factor of safety of 1.1 are largest in the east-west direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10 percent to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum displacement at the base of the nuclear island basemat (elevation 60.5′) using a coefficient of friction of 0.55 is 0.12 inch without buoyant force consideration, and 0.19 inch with buoyant force considered. This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the nuclear island is stable against sliding.

The minimum seismic stability factors of safety values are reported in Table 3.8.5-2.

3.8.5.5.6 Effect of Nuclear Island Basemat Uplift on Seismic Response

The effects of basemat uplift were evaluated using an east-west lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs (see Appendix 3G.2.2.5). Floor response spectra from safe shutdown earthquake time history analyses, which included basemat uplift, were compared to those from analyses that did not include uplift. The comparisons showed that the effect of basemat uplift on the floor response spectra is not significant.

The design analyses of the nuclear island basemat include consideration of sliding and liftoff between the containment internal structures and the containment vessel and of sliding between the containment vessel and the nuclear island basemat. Analyses of stability demonstrate that there was no uplift or sliding at the interface of the containment internal structures and the containment vessel.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The materials and quality control program used in the construction of the nuclear island structures foundation are described in Subsection 3.8.4.6.

There are no special construction techniques used in the construction of the nuclear island structures foundation. Subsection 2.5.4.2 describes information related to the excavation, backfill, and mudmat.

3.8.5.7 In-Service Testing and Inspection Requirements

The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

The need for foundation settlement monitoring is site-specific as discussed in Subsection 2.5.4.3.

3.8.5.8 Construction Inspection

Construction inspection is conducted to verify the concrete wall thickness and quantity of concrete reinforcement. The construction inspection includes concrete wall thickness and reinforcement expressed in units of in^2/ft (linear length) equivalent when compared to standard reinforcement bar sections. Inspections will be measured at applicable sections excluding designed openings or penetrations. Inspections will confirm that each section provides the minimum required reinforcement and concrete thickness as shown in Table 3.8.5-3. The minimum required reinforcement and concrete thickness represent the required minimum values to meet the design basis loads. Table 3.8.5-3 also indicates the reinforcement provided which may exceed the required minimum reinforcement for the following reasons:

- Structural margin
- Ease of construction
- Use of standardized reinforcement sizes and spacing

3.8.6 Combined License Information

3.8.6.1 Containment Vessel Design Adjacent to Large Penetrations

The final design of containment vessel elements (reinforcement) adjacent to concentrated masses (penetrations) is addressed in APP-GW-GLR-005 (Reference 53), and the applicable changes are incorporated into the UFSAR.

3.8.6.2 Passive Containment Cooling System Water Storage Tank Examination

Not used.

3.8.6.3 As-Built Summary Report

Not used.

3.8.6.4 In-Service Inspection of Containment Vessel

Not used.

3.8.6.5 Structures Inspection Program

The structures inspection program to address maintenance requirements for the seismic Category I and seismic Category II structures is addressed in Subsections 3.8.3.7, 3.8.4.7, 3.8.5.7, and 17.6.

3.8.6.6 Construction Procedures Program

Construction and inspection procedures for concrete filled steel plate modules address activities before and after concrete placement, use of construction mock-ups, and inspection of modules before and after concrete placement as discussed in Subsection 3.8.4.8. The procedures will be made available to NRC inspectors prior to use.

3.8.7 References

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*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.2-1
Load Combinations and Service Limits for Containment Vessel

Load Description		Load Combination and Service Limit										
		Con	**Test**	**Des.**	**Des.**	**A**	**A**	**A**	**C**	**D**	**C**	**D**
Dead	D	x	x	x	x	x	x	x	x	x	x	x
Live	L	x	x	x	x	x	x	x	x	x	x	x
Wind(5)	W	x						x				
Safe shutdown earthquake	E_s								x	x		x
Tornado	W_t										x	
Test pressure	P_t		x									
Test temperature	T_t		x									
Operating pressure	P_o							x			x	
Design pressure	P_d			x			x		x			x
Design external pressure	P_e				x	x				x		
Normal reaction	R_o				x	x		x		x	x	
Normal thermal(4)	T_o				x	x		(3)		x	(3)	
Accident thermal reactions	R_a			x			x		x			x
Accident thermal	T_a			x			x		x			x
Accident pipe reactions	Y_r											x
Jet impingement	Y_j											x
Pipe impact	Y_m											x

Notes:
1. Service limit levels are per ASME-NE.
2. Where any load reduces the effects of other loads, that load is to be taken as zero, unless it can be demonstrated that the load is always present or occurs simultaneously with the other loads.
3. Temperature of vessel is 70°F.
4. Temperature distribution for normal operation in cold weather.
5. Wind load for the construction load combination is based on a 70 mph wind. Wind load for the Service Level A load combination is analyzed as a reduction in external pressure.

Table 3.8.2-2
Containment Vessel Pressure Capabilities

Containment Element		Pressure Capability				
		Deterministic Severe Accident Capacity(1)			**Maximum Pressure Capability(2)**	
Temperature		100°F	300°F	400°F	100°F	400°F
Cylinder		135 psig	117 psig	112 psig	155 psig	129 psig
Ellipsoidal Head		104 psig	91 psig	87 psig	174 psig	144 psig
16-foot equipment hatch	F.S. = 1.67	126 psig	121 psig	118 psig	210 psig	198 psig
	F.S. = 2.50	84 psig	81 psig	79 psig		
Personnel airlocks(3)		>163 psig	>163 psig	>163 psig	>300 psig	>300 psig

Notes:

1. The buckling capacity of the ellipsoidal head is taken as 60 percent of the critical buckling pressure calculated by the BOSOR-5 nonlinear analyses; the buckling capacity at higher temperatures is calculated by reducing the capacity at 100°F by the ratio of yield at 100°F to yield at the higher temperature. Evaluations of the equipment hatch covers are shown both for ASME paragraph NE-3222 (F.S. = 2.50) and Code Case N-284 (F.S. = 1.67). Evaluations of the other elements are according to ASME Service Level C.
2. The estimated maximum pressure capability is based on minimum specified material properties.
3. The capacities of the personnel airlocks are estimated from test results.

Table 3.8.2-3
Analysis and Test Results of Fabricated Heads
(Reference 23)

	Test Model #1	**Test Model #2**
Cylinder radius	96.0 inches	96.0 inches
Knuckle radius	32.64 inches	32.64 inches
Spherical radius	172.8 inches	172.8 inches
Thickness	0.196 inches	0.27 inches
Head height/radius	0.5	0.5
Radius/thickness	490	356
Test initial buckling pressure	58 psig	106 psig
Test collapse pressure	229 psig	332 psig
Collapse pressure/initial buckling pressure	3.95	3.13
BOSOR-5 predicted buckling pressure	73.6 psig	106.6 psig

Table 3.8.2-4
Summary of Containment Vessel Models and Analysis Methods

Model	Analysis Method	Program	Purpose
Axisymmetric shell	Modal analysis	ANSYS	To calculate frequencies and mode shapes for comparison against stick model
Lumped mass stick model	Modal analysis	ANSYS	To create equivalent stick model for use in nuclear island seismic analyses
Axisymmetric shell	Static analyses using Fourier harmonic loads	ANSYS	To calculate containment vessel shell stresses
Axisymmetric shell	Nonlinear bifurcation	BOSOR5	To calculate buckling capacity close to base under thermal loads To calculate pressure capacity of top head
Finite element shell	Linear bifurcation	ANSYS	To study local effect of large penetrations and embedment on buckling capacity for axial and external pressure loads
Finite element shell	Modal analysis	ANSYS	To calculate frequencies and mode shapes for local effects of equipment hatches and personnel airlocks
Finite element shell	Static analyses	ANSYS	To calculate local shell stress in vicinity of the equipment hatches and personnel airlocks

Table 3.8.2-5
Maximum Absolute Nodal Acceleration (ZPA)
Steel Containment Vessel

Hard Rock Site Condition

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
281.90	1.48		1.56		1.25	
273.83	1.43		1.50		1.02	
265.83	1.38		1.43		0.85	
255.02	1.31		1.34		0.73	
244.21	1.23	1.28	1.26	1.30	0.68	0.71
224.00	1.09	1.13	1.11	1.17	0.66	0.68
200.00	0.90	0.94	0.94	0.98	0.61	0.63
169.93	0.69	0.71	0.72	0.75	0.53	0.55
162.00	0.63	0.65	0.67	0.68	0.51	0.53
141.50	0.49	0.50	0.54	0.54	0.45	0.47
131.68	0.43	0.44	0.47	0.48	0.41	0.44
112.50	0.40	0.41	0.37	0.38	0.35	0.40
104.12	0.38	0.40	0.38	0.40	0.32	0.38
100.00	0.38	0.40	0.39	0.41	0.31	0.34

Notes:
1. Enveloped response results at the north, south, east, and west edge nodes of the structure are shown. This is the maximum value of the response at any of these edge nodes.
2. Elevation 236'-6" is the mid-height of the polar crane bridge.

Table 3.8.3-1
Shear and Flexural Stiffnesses of Structural Module Walls

Case	Analysis Assumption	Shear Stiffness(1),(2)				Flexural Stiffness(1),(2)			
		48" Wall		30" Wall		48" Wall		30" Wall	
		GA x 10^6 lbs	Ratio	GA x 10^6 lbs	Ratio	EI x 10^9 lbs. in^2	Ratio	EI x 10^9 lbs. in^2	Ratio
1	Monolithic section considering steel plates and uncracked concrete. For shear stiffness this is (A_c G_c + A_s G_s).	83.5	1.0	55.8	1.0	47.5	1.0	13.6	1.0
2	Uncracked gross concrete section (full wall thickness considering steel plate as concrete)	73.9	0.89	46.2	0.83	33.2	0.70	8.1	0.60
3	Transformed cracked section considering steel plates and concrete (no concrete tension stiffness)	25.0	0.30	22.6	0.41	22.1	0.47	8.0	0.59

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

Table 3.8.3-2
Summary of Containment Internal Structures Models and Analysis Methods

Computer Program and Model	Analysis Method	Purpose	Concrete Stiffness [1]
3D ANSYS finite element of containment internal structures	Static	To obtain the in-plane and out-of-plane mechanical forces for the design of floors and walls (dead, live, hydrostatic, pressure)	Monolithic Case 1 with E_c reduced by factor of 0.8
3D ANSYS finite element of containment internal structures	Response spectra analyses[2]	To obtain the in-plane and out-of-plane seismic forces for the design of floors and walls	Monolithic Case 1 with E_c reduced by factor of 0.8.
3D ANSYS finite element of containment internal structures	Static analyses	To obtain the in-plane and out-of-plane member forces for thermal loads	Cracked Case 3
The following AP600 analyses are used as background to develop the AP1000 design loads.			
3D ANSYS finite element of containment internal structures fixed at elevation 103′-0″	Harmonic analyses	To evaluate natural frequencies potentially excited by hydrodynamic loads	Uncracked Case 2
	Time history analyses	To obtain dynamic response of IRWST boundary for hydrodynamic loads	Monolithic and cracked Cases 1 & 3

Note:
1. See Table 3.8.3-1 for stiffness case description.
2. See Section 3.7 for discussion of the containment internal structures seismic analyses.

Table 3.8.3-3
Definition of Critical Locations and Thicknesses for Containment Internal Structures(1)(4)

Wall Description (see detail in subsection 3.8.3.5.8.1)	Applicable Column Lines	Applicable Elevation Range	[*Concrete Thickness(2)*]*	Required Thickness of Surface Plates (inches)(3)	[*Thickness of Surface Plates Provided (inches)*]*
Containment Structures					
Module Wall 1	Southwest wall of refueling cavity	Wall separating IRWST and refueling cavity from elevation 103′ to 135′-3″	[*4'-0" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*]*	0.12	[*0.5 -0.01 +0.1*]*
Module Wall 2	South wall of west steam generator compartment	Wall separating IRWST and west steam generator compartment from elevation 103′ to 135′-3″	[*2'-6" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*]*	0.43	[*0.5 -0.01 +0.1*]*
CA02 Module Wall	North east boundary wall of IRWST	Wall separating IRWST and maintenance floor from elevation 103′ to 135′-3″	[*2'-6" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*]*	0.36	[*0.5 -0.01 +0.1*]*

Notes:
1. The applicable column lines and elevation levels are identified and included in Figures 1.2-9, 3.7.2-12 (sheets 1 through 12), 3.7.2-19 (sheets 1 through 3) and on Table 1.2-1.
[*2. The concrete thickness includes the steel face plates. Thickness greater than 3'-0" have a construction tolerance of +1", -3/4". Thickness less than or equal to 3'-0" have a construction tolerance of +1/2", -3/8".*]*
3. These plate thicknesses represent the thickness required for operating and design basis loads except for designed openings or penetrations. These values apply for each face of the applicable wall unless specifically indicated on the table. For load combinations with thermal loads, the evaluation is performed as described in Subsection 3.8.3.5.3.4.
[*4. As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thicknesses.*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-4 (Sheet 1 of 3)
Design Summary of Southwest Wall of Refueling Cavity Design Loads, Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Mid-Height

	TX	TY	TXY	MX	MY	MXY	NX	NY	
Load/Comb.	**k/ft**	**k/ft**	**k/ft**	**kft/ft**	**kft/ft**	**kft/ft**	**k/ft**	**k/ft**	**Comments**
Dead (D)	1	-20	1	1	1	0	0	0	–
Hydro (F)	2	0	1	24	30	-2	0	0	–
Live (L)	0	-7	0	3	2	0	0	1	During refueling
Live (L_o)	0	-2	0	1	1	0	0	0	During operation
ADS	2	7	6	19	19	-4	0	1	–
E_s	13	49	56	41	40	15	1	8	During operation
Thermal (T_0)	-74	-34	-47	141	113	0	4	-1	During operation
LC (1)	8	-18	14	70	76	-8	0	3	[*1.4D+1.4F+1.7*L_o*+1.7ADS*]*
LC (2)	5	-40	3	41	46	-2	0	2	[*1.4D+1.4F+1.7*L_r]*
LC (3)	8	-15	14	69	76	-8	0	3	[*1.4D+1.4F+1.7ADS*]*
LC (4)	18	35	65	87	90	17	1	10	[*D+F+*L_o *+|ADS|+*E_s]*
LC (5)	-12	-78	-60	-34	-28	-20	-1	-8	[*D+F+*L_o *-|ADS|-* E_s]*
LC (6)	-56	0	18	228	203	17	5	9	[*D+F+*L_o *+|ADS|+*T_0*+*E_s]*
LC (7)	-86	-112	-108	107	85	-19	3	-9	[*D+F+*L_o *-|ADS|+*T_0*-*E_s]*
LC (8)	18	39	65	86	90	17	1	10	[*0.9D+1.0F+1.0|ADS|+1.0*E_s]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 101870

Plate thickness required for load combinations excluding thermal: 0.12 inches (Maximum)

[*Plate thickness provided:* *0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal: 12.9 ksi

[*Yield stress at temperature:* *65.0 ksi (Minimum)*]*

Maximum stress intensity range for load combinations including thermal: 19.6 ksi

Allowable stress intensity range for load combinations including thermal: 130.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-4 (Sheet 2 of 3)
Design Summary of Southwest Wall of Refueling Cavity Design Loads, Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Base

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-31	1	-1	-4	0	0	1	–
Hydro (F)	3	1	2	-2	-41	-1	-1	17	–
Live (L)	-1	-6	0	0	-3	0	0	1	During refueling
Live (L_o)	0	-2	0	0	-1	0	0	0	During operation
ADS	2	6	7	-2	-32	-1	-1	10	–
E_s	11	55	65	12	87	3	3	18	During operation
Thermal (T_0)	-189	-31	-76	165	86	-2	-5	0	During operation
LC (1)	7	-35	16	-8	118	-3	-3	41	[*1.4D+1.4F+1.7L$_o$+1.7ADS*]*
LC (2)	2	-53	5	-5	-68	-2	-1	25	[*1.4D+1.4F+1.7L$_r$*]*
LC (3)	7	-32	16	-8	117	-3	-3	41	[*1.4D+1.4F+1.7ADS*]*
LC (4)	15	29	75	11	73	3	4	45	[*D+F+L$_o$ +|ADS|+E$_s$*]*
LC (5)	-11	-93	-68	-17	165	-6	-5	-10	[*D+F+L$_o$ -|ADS|- E$_s$*]*
LC (6)	-174	-2	-1	175	160	1	-1	44	[*D+F+L$_o$ +|ADS|+T$_0$+E$_s$*]*
LC (7)	-200	124	144	147	78	-8	-10	-11	[*D+F+L$_o$ -|ADS|+T$_0$-E$_s$*]*
LC (8)	16	34	74	11	74	3	4	45	[*0.9D+1.0F+1.0|ADS|+1.0E$_s$*]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 101788

Plate thickness required for load combinations excluding thermal: 0.09 inches (Maximum)

[*Plate thickness provided:* *0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal: 15.5 ksi

[*Yield stress at temperature:* *65.0 ksi (Minimum)*]*

Maximum stress intensity range for load combinations including thermal: 24.8 ksi

Allowable stress intensity range for load combinations including thermal: 130.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-4 (Sheet 3 of 3)
Design Summary of Southwest Wall of Refueling Cavity Design Loads, Load Combinations, and Comparison to Acceptance Criteria North End Bottom Corner

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-2	-29	-7	0	-3	0	0	0	–
Hydro (F)	3	1	4	-10	-16	4	1	3	–
Live (L)	-1	-4	0	0	-1	0	0	0	During refueling
Live (L_o)	0	-1	0	0	0	0	0	0	During operation
ADS	3	-2	7	-7	-18	2	1	2	–
E_s	23	43	64	14	68	6	6	7	During operation
Thermal (T_0)	-173	-88	-42	138	94	-15	-30	13	During operation
LC (1)	6	-44	8	-27	-57	8	4	7	[*$1.4D+1.4F+1.7L_o+1.7ADS$*]*
LC (2)	0	-46	-4	-16	-28	5	3	4	[*$1.4D+1.4F+1.7L_r$*]*
LC (3)	6	-42	9	-27	-57	8	4	7	[*$1.4D+1.4F+1.7ADS$*]*
LC (4)	26	16	68	10	66	11	9	11	[*$D+F+L_o+\|ADS\|+E_s$*]*
LC (5)	-25	-74	-74	-32	-104	-4	-6	-5	[*$D+F+L_o-\|ADS\|-E_s$*]*
LC (6)	-147	-73	26	148	160	-4	-21	25	[*$D+F+L_o+\|ADS\|+T_0+E_s$*]*
LC (7)	-198	-162	-116	106	-11	-20	-36	8	[*$D+F+L_o-\|ADS\|+T_0-E_s$*]*
LC (8)	26	19	69	11	67	11	9	11	[*$0.9D+1.0F+1.0\|ADS\|+1.0E_s$*]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 101794

Plate thickness required for load combinations excluding thermal: 0.09 inches (Maximum)

[*Plate thickness provided:* *0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal: 11.1 ksi

[*Yield stress at temperature:* *65.0 ksi (Minimum)*]*

Maximum stress intensity range for load combinations including thermal: 21.3 ksi

Allowable stress intensity range for load combinations including thermal: 130.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-5 (Sheet 1 of 3)
Design Summary of South Wall of West Steam Generator Compartment Design Loads, Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Mid-Height

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-23	0	1	1	0	0	0	–
Hydro (F_o)	-2	3	-5	20	22	0	0	-1	–
Live (L)	0	-7	-1	1	0	0	0	0	During refueling
Live (L_o)	0	-2	0	0	0	0	0	0	During operation
ADS	-1	9	-10	17	16	-1	0	1	–
E_s	7	65	58	30	26	5	1	4	During operation
Thermal (T_0)	-42	-53	19	69	59	2	-1	-1	During operation
LC (1)	-5	-17	-24	59	59	-2	1	0	[*1.4D+1.4F+1.7L*$_o$*+1.7ADS*]*
LC (2)	-4	-39	-7	32	32	-1	1	-1	[*1.4D+1.4F+1.7*L_r]*
LC (3)	-5	-13	-24	59	59	-2	1	0	[*1.4D+1.4F+1.7ADS*]*
LC (4)	6	52	64	69	65	5	2	4	[*D+F+*L_o *+|ADS|+*E_s]*
LC (5)	-11	-96	-73	-26	-19	-6	-1	-6	[*D+F+*L_o *-|ADS|-* E_s]*
LC (6)	-36	-1	83	138	124	8	1	3	[*D+F+*L_o *+|ADS|+*T_0*+*E_s]*
LC (7)	-53	-149	-54	43	41	-4	-2	-7	[*D+F+*L_o *-|ADS|+*T_0*-*E_s]*
LC (8)	6	56	64	68	64	5	2	4	[*0.9D+1.0F+1.0|ADS|+1.0*E_s]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 104228

Plate thickness required for load combinations excluding thermal:	0.15 inches (Maximum)
[*Plate thickness provided:*	*0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal:	16.5 ksi
[*Yield stress at temperature:*	*36.0 ksi (Minimum)*]*
Maximum stress intensity range for load combinations including thermal:	16.5 ksi
Allowable stress intensity range for load combinations including thermal:	72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-5 (Sheet 2 of 3)
Design Summary of South Wall of West Steam Generator Compartment Design Loads, Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Base

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-3	-28	-1	0	2	0	0	0	–
Hydro (F)	3	4	-9	-4	-38	0	0	15	–
Live (L)	-1	-5	-1	0	-1	0	0	0	During refueling
Live (L_o)	0	-2	0	0	0	0	0	0	During operation
ADS	2	10	-10	-3	-30	0	0	9	–
E_s	14	74	52	9	62	1	1	14	During operation
Thermal (T_0)	-136	-13	43	79	65	5	4	-1	During operation
LC (1)	3	-20	-32	-10	-102	-1	-1	36	[*$1.4D+1.4F+1.7L_o+1.7ADS$*]*
LC (2)	-2	-43	-16	-5	-52	-1	0	21	[*$1.4D+1.4F+1.7L_r$*]*
LC (3)	4	-17	-32	-10	-102	-1	-1	36	[*$1.4D+1.4F+1.7ADS$*]*
LC (4)	16	58	52	9	55	1	1	38	[*$D+F+L_o+\|ADS\|+E_s$*]*
LC (5)	-17	-110	-73	-16	-128	-2	-2	-8	[*$D+F+L_o-\|ADS\|-E_s$*]*
LC (6)	-120	45	95	88	120	6	5	37	[*$D+F+L_o+\|ADS\|+T_0+E_s$*]*
LC (7)	-152	-122	-30	63	-63	3	3	-9	[*$D+F+L_o-\|ADS\|+T_0-E_s$*]*
LC (8)	16	62	52	9	55	1	1	38	[*$0.9D+1.0F+1.0\|ADS\|+1.0E_s$*]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 101943

Plate thickness required for load combinations excluding thermal:	0.14 inches (Maximum)
[*Plate thickness provided:*	*0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal:	17.6 ksi
[*Yield stress at temperature:*	*36.0 ksi (Minimum)*]*
Maximum stress intensity range for load combinations including thermal:	18.7 ksi
Allowable stress intensity range for load combinations including thermal:	72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-5 (Sheet 3 of 3)
Design Summary of South Wall of West Steam Generator Compartment Design Loads, Load Combinations, and Comparison to Acceptance Criteria West End Bottom Corner

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-6	-37	3	-1	6	1	-1	-3	–
Hydro (F)	5	14	-10	-6	-14	3	2	4	–
Live (L)	-2	-11	1	0	1	0	0	-1	During refueling
Live (L_o)	-1	-4	0	0	1	0	0	-1	During operation
ADS	9	36	-10	-3	-15	2	2	4	–
E_s	49	248	53	10	72	4	10	31	During operation
Thermal (T_0)	-125	-61	91	26	64	-4	-27	-23	During operation
LC (1)	14	21	-28	-15	-34	9	5	7	[*1.4D+1.4F+1.7*L_o*+1.7ADS*]*
LC (2)	-4	-52	-8	-10	-8	6	0	-1	[*1.4D+1.4F+1.7*L_r]*
LC (3)	15	27	-28	-15	-35	9	5	8	[*1.4D+1.4F+1.7ADS*]*
LC (4)	57	256	57	6	80	10	13	35	[*D+F+*L_o *+|ADS|+*E_s]*
LC (5)	-59	-311	-71	-20	-93	-2	-12	-36	[*D+F+*L_o *-|ADS|-* E_s]*
LC (6)	-67	195	147	32	144	6	-14	13	[*D+F+*L_o *+|ADS|+*T_0*+*E_s]*
LC (7)	-184	-372	20	7	-29	-6	-39	-58	[*D+F+*L_o *-|ADS|+*T_0*-*E_s]*
LC (8)	59	264	56	6	79	10	14	36	[*0.9D+1.0F+1.0|ADS|+1.0*E_s]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 101933

Plate thickness required for load combinations excluding thermal: 0.43 inches (Maximum)

[*Plate thickness provided:* *0.50 inches -0.01 +0.10*

As described in subsection 3.8.3.1.3, some CA01 and CA05 module wall faceplate thicknesses are greater than the 0.5-inch nominal faceplate thickness.]*

Maximum principal stress for load combinations including thermal: 31.9 ksi

[*Yield stress at temperature:* *36.0 ksi (Minimum)*]*

Maximum stress intensity range for load combinations including thermal: 36.0 ksi

Allowable stress intensity range for load combinations including thermal: 72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-6 (Sheet 1 of 3)
Design Summary of North-East Wall of IRWST Design Loads,
Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Mid-Height

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-18	-1	0	1	-1	-2	-2	–
Hydro (F)	0	1	-1	16	17	3	2	0	–
Live (L)	0	-13	-1	5	5	-1	-2	-1	During refueling
Live (L_o)	0	-4	-1	4	5	-1	-2	-1	During operation
ADS	-2	3	0	16	14	5	2	1	–
E_s	16	42	47	35	39	12	11	16	During operation
Thermal (T_0)	-32	-13	48	49	51	-2	-5	-3	During operation
LC (1)	-5	-26	-3	58	58	8	0	-3	[*1.4D+1.4F+1.7*L_o*+1.7ADS*]*
LC (2)	-2	-46	-4	30	33	0	-3	-5	[*1.4D+1.4F+1.7*L_r]*
LC (3)	-5	-19	-1	51	48	10	3	-1	[*1.4D+1.4F+1.7ADS*]*
LC (4)	16	24	45	72	76	17	11	14	[*D+F+*L_o *+|ADS|+*E_s]*
LC (5)	-19	-67	-49	-31	-31	-16	-15	-21	[*D+F+*L_o *-|ADS|-* E_s]*
LC (6)	-16	11	93	121	127	15	6	11	[*D+F+*L_o *+|ADS|+*T_0*+*E_s]*
LC (7)	-51	-80	-1	18	20	-18	-20	-24	[*D+F+*L_o *-|ADS|+*T_0*-*E_s]*
LC (8)	16	29	46	68	71	18	13	16	[*0.9D+1.0F+1.0|ADS|+1.0*E_s]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 140027

Plate thickness required for load combinations excluding thermal:	0.13 inches (Maximum)
[*Plate thickness provided:*	*0.50 inches -0.01 +0.10*]*
Maximum principal stress for load combinations including thermal:	18.1 ksi
[*Yield stress at temperature:*	*36.0 ksi (Minimum)*]*
Maximum stress intensity range for load combinations including thermal:	18.1 ksi
Allowable stress intensity range for load combinations including thermal:	72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-6 (Sheet 2 of 3)
Design Summary of North-East Wall of IRWST Design Loads, Load Combinations, and Comparison to Acceptance Criteria Mid-Span at Bottom-Elevation 107'-2"

	TX	TY	TXY	MX	MY	MXY	NX	NY	
Load/Comb.	**k/ft**	**k/ft**	**k/ft**	**kft/ft**	**kft/ft**	**kft/ft**	**k/ft**	**k/ft**	**Comments**
Dead (D)	0	-19	1	0	0	0	0	0	–
Hydro (F)	1	2	-1	2	-8	2	0	11	–
Live (L)	-1	-7	0	0	-3	0	0	1	During refueling
Live (L_o)	-1	-1	0	0	-2	0	0	1	During operation
ADS	0	4	1	2	-7	3	0	8	–
E_s	23	54	47	13	48	8	6	17	During operation
Thermal (T_0)	-92	-58	71	64	63	-1	0	-1	During operation
LC (1)	0	-20	3	7	-27	8	-1	32	[*1.4D+1.4F+1.7L_o+1.7ADS*]*
LC (2)	1	-37	1	3	-16	3	-1	18	[*1.4D+1.4F+1.7L_r*]*
LC (3)	1	-18	3	6	-24	8	0	30	[*1.4D+1.4F+1.7ADS*]*
LC (4)	24	39	49	17	45	13	5	38	[*D+F+L_o +|ADS|+E_s*]*
LC (5)	-23	-77	-47	-12	-65	-8	-7	-13	[*D+F+L_o -|ADS|- E_s*]*
LC (6)	-68	-18	120	81	108	12	5	37	[*D+F+L_o +|ADS|+T_0+E_s*]*
LC (7)	-114	-134	24	52	-2	-9	-7	-14	[*D+F+L_o -|ADS|+T_0-E_s*]*
LC (8)	25	42	49	17	47	13	6	37	[*0.9D+1.0F+1.0|ADS|+1.0E_s*]*

Notes:
x-direction is horizontal; y-direction is vertical.
Element number 140005

Plate thickness required for load combinations excluding thermal:	0.12 inches (Maximum)
[*Plate thickness provided:*	*0.50 inches -0.01 +0.10*]*
Maximum principal stress for load combinations including thermal:	17.0 ksi
[*Yield stress at temperature:*	*36.0 ksi (Minimum)*]*
Maximum stress intensity range for load combinations including thermal:	21.9 ksi
Allowable stress intensity range for load combinations including thermal:	72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-6 (Sheet 3 of 3)
Design Summary of North-East Wall of IRWST Design Loads, Load Combinations, and Comparison to Acceptance Criteria
North End Bottom Corner-Elevation 107’-2"

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-19	1	0	-4	0	0	0	–
Hydro (F)	2	12	6	5	10	12	-7	-11	–
Live (L)	0	-10	1	-1	-4	1	1	1	During refueling
Live (L_o)	0	-3	0	0	-2	1	0	0	During operation
ADS	0	19	7	7	16	11	-5	-11	–
E_s	8	136	38	26	98	17	14	31	During operation
Thermal (T_0)	38	28	60	-39	24	-11	17	34	During operation
LC (1)	2	17	21	16	31	35	-17	-33	[*$1.4D+1.4F+1.7L_o+1.7ADS$*]*
LC (2)	2	-28	10	4	2	17	-8	-13	[*$1.4D+1.4F+1.7L_r$*]*
LC (3)	2	21	21	17	35	34	-18	-33	[*$1.4D+1.4F+1.7ADS$*]*
LC (4)	10	144	52	37	118	40	12	32	[*$D+F+L_o+\|ADS\|+E_s$*]*
LC (5)	-7	-165	-38	-29	-110	-16	-25	-52	[*$D+F+L_o-\|ADS\|-E_s$*]*
LC (6)	48	172	111	-2	142	29	29	66	[*$D+F+L_o+\|ADS\|+T_0+E_s$*]*
LC (7)	31	-137	22	-68	-86	-27	-8	-19	[*$D+F+L_o-\|ADS\|+T_0-E_s$*]*
LC (8)	10	149	51	37	120	39	12	32	[*$0.9D+1.0F+1.0\|ADS\|+1.0E_s$*]*

Notes:

x-direction is horizontal; y-direction is vertical.

Element number 140001

Plate thickness required for load combinations excluding thermal: 0.36 inches (Maximum)

[*Plate thickness provided:* *0.50 inches -0.01 +0.10*]*

Maximum principal stress for load combinations including thermal: 29.5 ksi

[*Yield stress at temperature:* *36.0 ksi (Minimum)*]*

Maximum stress intensity range for load combinations including thermal: 30.3 ksi

Allowable stress intensity range for load combinations including thermal: 72.0 ksi (Minimum)

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.3-7
Design Summary of Steel Wall of IRWST

Mechanical Loads Only AISC Interaction Ratio			
Section Location and Element Number	**T Section**	**L Section**	**Load Combination**
TYPICAL COLUMN AT MIDDLE OF WALL			
Top	0.13	0.10	*[$D+F+L_o$ -|ADS|- E_s (LC#5)]**
Mid-height	0.33	0.12	*[$D+F+L_o$ -|ADS|- E_s (LC#5)]**
	0.19	0.27	*[$D+F+L_o$ +|ADS|+E_s (LC#4)]**
Bottom	0.18	0.01	*[$D+F+L_o$ -|ADS|- E_s (LC#5)]**
ENVELOPE OF ALL LOCATIONS AND LOAD COMBINATIONS			
	0.42	0.69	[D+F+ADS (LC#3)]*

Mechanical Plus Thermal Loads Ratio of Stress to AISC or ASME (2 * Sy = 105.6 ksi at T_0 = 260 F)[2]				
Section Location and Element Number	**Flange of T Section**	**Flange of L Section**	**Plate**	**Load Combination**
TYPICAL COLUMN AT MIDDLE OF WALL				
Top	0.17 AISC	0.45 AISC	0.60 AISC	*[$D+F+L_o$ -|ADS|+T_0-E_s (LC#7)]**
	0.11 AISC	0.17 AISC	0.39 AISC	*[$D+F+L_o$ +|ADS|+T_0+E_s (LC#6)]**
Mid-height	0.73 AISC	0.39 AISC	0.35 AISC	*[$D+F+L_o$ -|ADS|+T_0-E_s (LC#7)]**
Bottom	0.25 ASME	0.46 AISC	0.46 AISC	*[$D+F+L_o$ +|ADS|+T_0+E_s (LC#6)]**
	0.16 ASME	0.64 AISC	0.73 AISC	*[$D+F+L_o$ -|ADS|+T_0-E_s (LC#7)]**
ENVELOPE OF ALL LOCATIONS AND LOAD COMBINATIONS				
–	0.63 ASME	0.43 ASME	0.69[2] ASME	*[$D+F+L_o$ +|ADS|+T_0+E_s (LC#6)]**

Notes:
1. Results of the evaluation of mechanical and thermal loads are shown against the AISC allowables when the stresses are less than yield. Portions of the steel wall at the end of the wall exceed yield due to the restraint provided by the adjacent concrete. These areas are evaluated against the ASME allowables as described in Subsection 3.8.3.5.3.4.
2. T_0 = 260 F ADS2 event per Subsection 3.8.3.3.1.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.4-1
[*LOAD COMBINATIONS AND LOAD FACTORS FOR SEISMIC CATEGORY I STEEL STRUCTURES*]*

Combination No.		*Load Combination and Factors*								
		1	***2***	***3***	***4***	***5***	***6***	***7***	***8***	***9***
Load Description										
Dead	D	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Liquid	F	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live	L	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Earth pressure	H	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Normal reaction	R_o	1.0	1.0	1.0	1.0				1.0	1.0
Normal thermal	T_o			1.0	1.0				1.0	1.0
Wind	W		1.0							1.0
Safe shutdown earthquake	E_S			1.0				1.0		
Tornado	W_t				1.0					
Accident pressure	P_a					1.0	1.0	1.0		
Accident thermal	T_a					1.0	1.0	1.0		
Accident thermal reactions	R_a					1.0	1.0	1.0		
Accident pipe reactions	Y_r						1.0	1.0		
Jet impingement	Y_j						1.0	1.0		
Pipe impact	Y_m						1.0	1.0		
Stress Limit Coefficient [(1),(3)] *(except for compression)*		1.0	1.0	1.6	1.6	1.6	1.6	1.7	1.5	1.5
(for compression)		1.0	1.0	1.4	1.4	1.4	1.4	1.6	1.3	1.3

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

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.4-2
[*LOAD COMBINATIONS AND LOAD FACTORS FOR SEISMIC CATEGORY I CONCRETE STRUCTURES*]*

Load Combination and Factors												
Combination No.		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Dead	*D*	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.4	1.0
Liquid	*F*	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.4	1.0
Live	*L*	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.7	1.0
Earth	*H*	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.7	1.0
Design Pressure	P_d										1.5	1.0
Normal reaction	R_o	1.7	1.7	1.0	1.0				1.3	1.3	1.7	1.0
Normal thermal	T_o			1.0	1.0				1.2	1.2		
Wind	*W*		1.7							1.3		
Safe shutdown earthquake	E_s			1.0				1.0				1.0
Tornado	W_t				1.0							
Accident pressure	P_a					1.4	1.25	1.0				
Accident thermal	T_a					1.0	1.0	1.0				
Accident thermal reactions	R_a					1.0	1.0	1.0				
Accident pipe reactions	Y_r						1.0	1.0				
Jet impingement	Y_j						1.0	1.0				
Pipe impact	Y_m						1.0	1.0				

Notes:

1. *Design for mechanical loads is in accordance with ACI-349 Strength Design Method for all load combinations. Design for combinations including thermal loads is described in subsection 3.8.3.5.3.4. Shield building design for combinations including thermal loads is described in subsection 3.8.4.5.5.4.*
2. *Where any load reduces the effects of other loads, the corresponding coefficient for that load is taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with the other loads. Otherwise the coefficient for the load is taken as zero.*
3. *Loads due to maximum precipitation are evaluated using load combination 4 with the maximum precipitation in place of the tornado load.*
4. *Load combinations 10 and 11 are applicable to nuclear island basemat design only. P_d, the containment design pressure, is 59 psig.*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8.4-3
Acceptance Tests for Concrete Aggregates

Method of Test	Designation
Organic impurities in sand	ASTM C 40
Effect of organic impurities on strength of mortar	ASTM C 87
Soundness of aggregates	ASTM C 88
Material finer than No. 200 sieve	ASTM C 117
Specific gravity and absorption - coarse aggregates	ASTM C 127
Specific gravity and absorption - fine aggregates	ASTM C 128
Los Angeles abrasion of small-size coarse aggregates	ASTM C 131
Sieve analysis	ASTM C 136
Friable particles	ASTM C 142
Potential reactivity of aggregates (chemical)	ASTM C 289
Petrographic examination of aggregates	ASTM C 295
Resistance to degradation of large-size coarse aggregates by abrasion and impact in the Los Angeles machine	ASTM C 535
Potential alkali reactivity of carbonate rocks for concrete aggregates	ASTM C 586
Resistance of concrete to rapid freezing and thawing	ASTM C 666
Flat and elongated particles	ASTM D4791

Table 3.8.4-4
Criteria for Water Used in Production of Concrete

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

Table 3.8.4-5
Not Used

Table 3.8.4-6 (Sheet 1 of 2)
Materials Used in Structural and Miscellaneous Steel

Standard	Construction Material
ASTM A1	Carbon steel rails
ASTM A36/A36M	Rolled shapes, plates, and bars
ASTM A53	Welded and Seamless Steel Pipe, Grade B
ASTM A106	Seamless Carbon Steel Pipe For High Temperature Service
ASTM A108	Weld studs
ASTM A123	Zinc coatings (hot galvanized)
ASTM A167	Stainless and Heat-Resisting Chromium Nickel Steel Plate, Sheet and Strip
ASTM A193	Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service
ASTM A194	Carbon and Alloy Steel Nuts for Bolts for High-Pressure or High-Temperature Service, or Both
ASTM A240	Duplex 2101 stainless steel (designation S32101)
ASTM A242	High-strength low alloy structural steel
ASTM A276	Stainless and Heat-Resisting Steel Bars and Shapes
ASTM A307	Low carbon steel bolts
ASTM A312	Seamless and Welded Austenitic Stainless Steel Pipe
ASTM A325	High strength bolts
ASTM A354	Quenched and tempered alloy steel bolts (Grade BC)
ASTM A441	High-strength low alloy structural manganese vanadium steel
ASTM A490	Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength
ASTM A496	High strength deformed wire
ASTM A500	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A501	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
ASTM A505	Standard Specification for Steel, Sheet and Strip, Alloy, Hot-Rolled and Cold-Rolled
ASTM A514	High-Yield Strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding
ASTM A517	Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered
ASTM A540	Alloy Steel Bolting for Special Applications

Table 3.8.4-6 (Sheet 2 of 2)
Materials Used in Structural and Miscellaneous Steel

Standard	Construction Material
ASTM A564	Hot-Rolled and Cold-Finished, Age-Hardening Stainless and Heat-Resisting Steel Bars and Shapes
ASTM A570	Hot-Rolled Carbon Steel Sheets and Strip, Structural Quality, Grades C, D, and E
ASTM A572	High-strength low alloy structural steel
ASTM A588	High-strength low alloy structural steel
ASTM A607	Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low Alloy, Columbium, and/or Vanadium
ASTM A615	Deformed and Plain Billet Steel Bars for Concrete Reinforcement
ASTM A618	Hot-Formed Welded and Seamless High-Strength Low Alloy Structural Tubing
ASTM A706	Low Alloy Steel Deformed Bars for Concrete Reinforcement
ASTM A970	Headed Steel Bars for Concrete Reinforcement
ASTM A992	Rolled shapes, plates, and bars
ASTM-F1554	Steel anchor bolts, 36, 55, and 105-ksi yield strength

Table 3.8.5-1
Minimum Required Factor of Safety
for Overturning and Sliding of Structures

Load Combination	Overturning	Sliding	Flotation
D + H + B + W	1.5	1.5	-
D + H + B + E_s	1.1	1.1	-
D + H + B + W_t	1.1	1.1	-
D + F	-	-	1.1
D + B	-	-	1.5

where:

D = dead load excluding the fluid loads
H = lateral earth pressure
W = wind load
E_s = safe shutdown earthquake load
W_t = tornado load
F = buoyant force due to the design basis flood
B = buoyant force on submerged structure due to high ground water table

Table 3.8.5-2
Factors of Safety for Flotation, Overturning and Sliding of Nuclear Island Structures

Environmental Effect	Factor of Safety[1]
Flotation	
High Ground Water Table	3.7
Design Basis Flood	3.5
Sliding	
Design Wind, North-South	14.0
Design Wind, East-West	10.1
Design Basis Tornado, North-South	7.7
Design Basis Tornado, East-West	5.9
Safe Shutdown Earthquake, North-South	1.1[2]
Safe Shutdown Earthquake, East-West	1.1[2]
Overturning	
Design Wind, North-South	51.5
Design Wind, East-West	27.9
Design Basis Tornado, North-South	17.7
Design Basis Tornado, East-West	9.6
Safe Shutdown Earthquake, North-South	1.77
Safe Shutdown Earthquake, East-West	1.17

Notes:
1. Factor of safety is calculated for the envelope of the soil and rock sites described in Subsection 3.7.1.4.
2. From non-linear sliding analysis using friction elements, the horizontal movement is negligible (0.12 inch without buoyant force consideration, and 0.19 inch with buoyant force considered).

Table 3.8.5-3
Definition of Critical Locations, Thicknesses and Reinforcement for Nuclear Island Basemat(1) (in^2/ft)

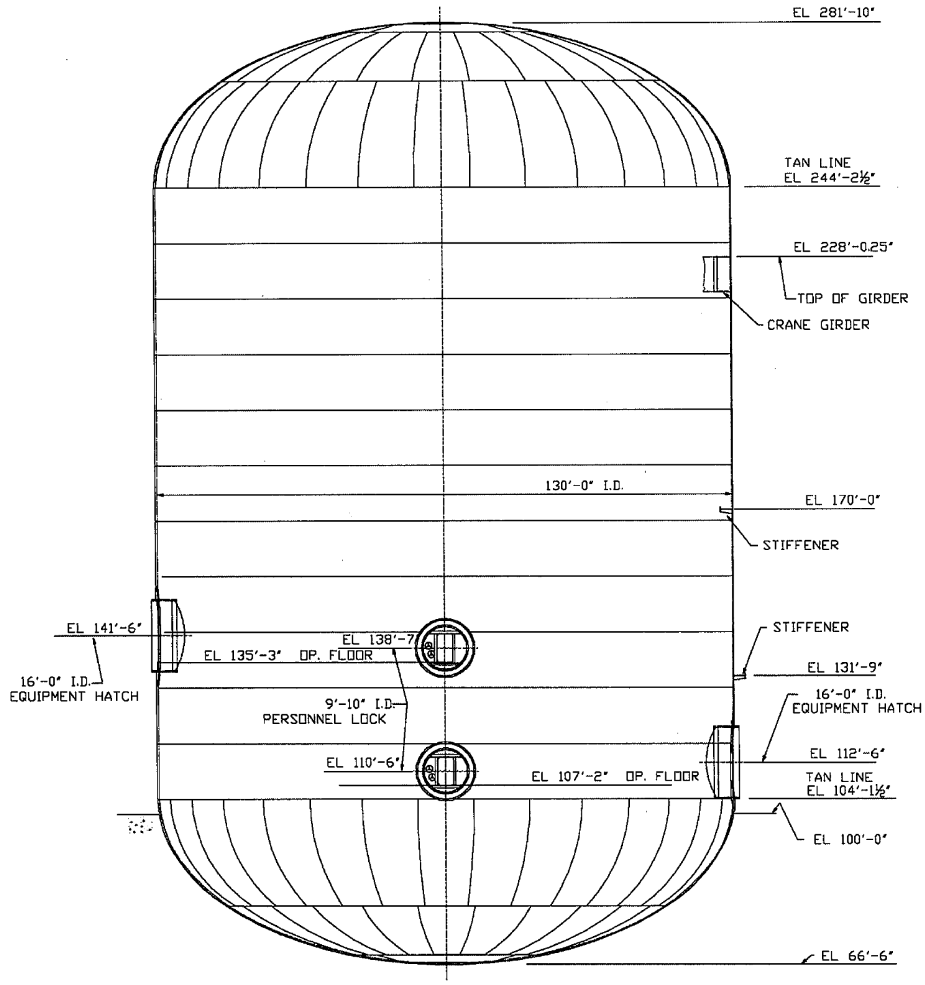
Basemat Segment (see detail in subsection 3.8.5.4.4)	Location	Required(3)(6)			*[Provided (Minimum)(4)]**		
		North-South	East-West	Shear	*North-South*	*East-West*	*Shear*
Auxiliary Building Basemat Elevation 66′ 6″							
Column line K to L and from Shield Building to Col. Line 11(2)							
	Top Face	Note 5	1.5		[*2.25*	*2.25*	
	Bottom Face	Note 5	1.6		*2.25*	*2.25*	
				0.23			*0.25*]*
Column line 1 to 2 and from Column Line K-2 to N wall(2)							
General area	Top Face	Note 5	Note 5		[*2.25*	*2.25*	
Central zone	Top Face	2.72	Note 5		*3.25*	*2.25*	
	Bottom Face	2.25	Note 5		*2.25*	*2.25*	
				0.47			*0.50*]*

Notes:
1. The applicable column lines and elevation levels are identified and included in Figures 1.2-9 and 3.7.2-12 (sheets 1 through 12).
[2. *The thickness of these sections is 6′0″ with a construction tolerance of +4 inches, -3/4 inch.*]*
3. These concrete reinforcement values represent the minimum reinforcement required for structural requirements except for designed openings, penetrations, sumps or elevator pits.
[4. *These concrete reinforcement values represent the provided reinforcement for structural requirements except for designed openings, penetrations, sumps or elevator pits.*]*
5. Reinforcement demand is low. Basemat reinforcement is fairly uniform across all of the basemat and the governing location is in other segments (see Figure 3.8.5-3, sheets 3 to 6).
6. Thermal loads have been considered in the design of critical sections. The required reinforcement values shown do not include the load case where seismic and normal thermal loads are numerically combined as the normal thermal loads were assessed to be insignificant. When the seismic and normal thermal loads are numerically combined, the value of required reinforcement may increase; however, in all cases the required reinforcement is less than the provided reinforcement and thus the design of the critical section reinforcement is acceptable.

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Table 3.8-201
Waterproof Membrane Inspections, Tests, Analyses, and Acceptance Criteria

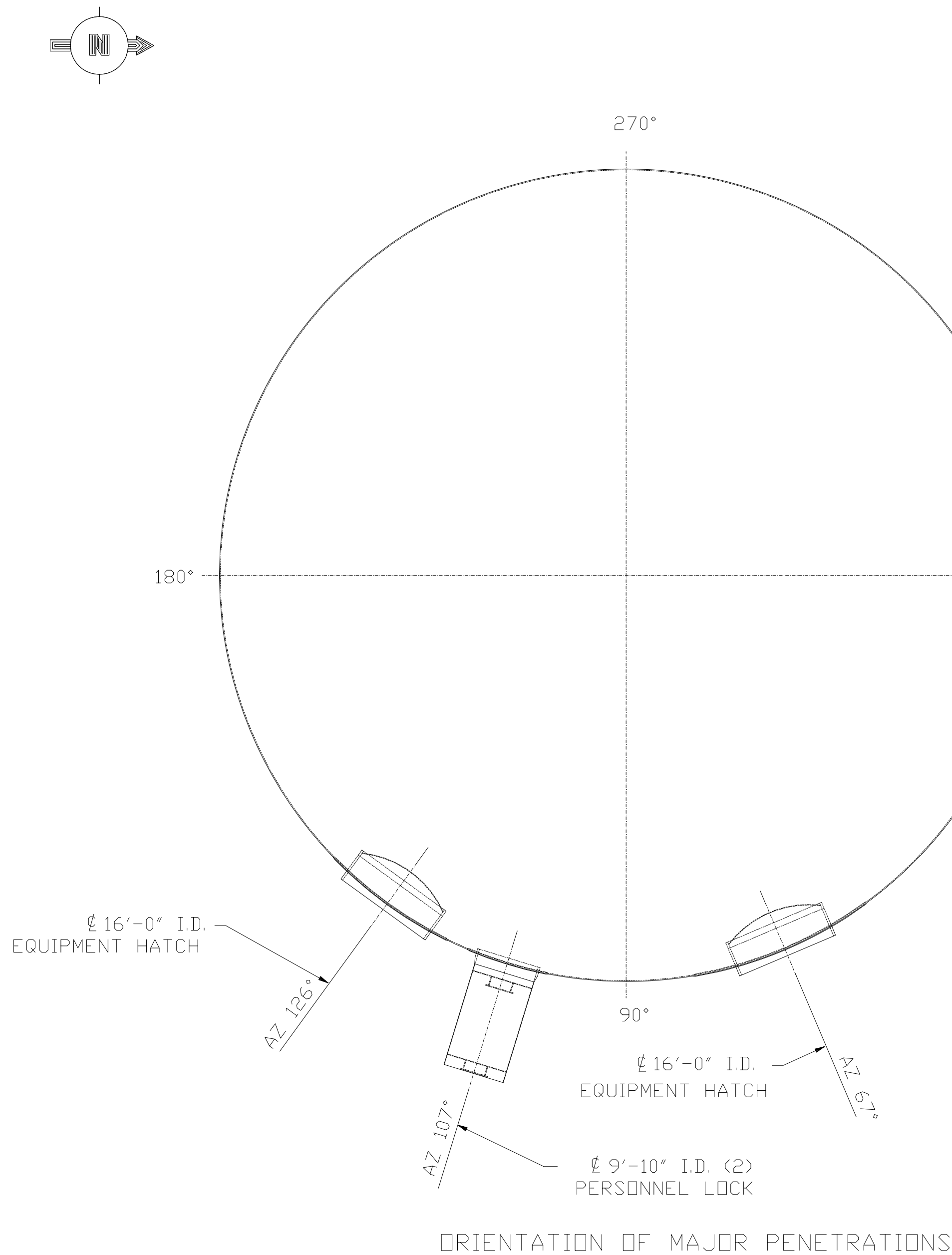
Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1) The friction coefficient to resist sliding is 0.7 or higher.	Testing will be performed to confirm that the mudmat-waterproofing-mudmat interface beneath the Nuclear Island basemat has a minimum coefficient of friction to resist sliding of 0.7.	A report exists and documents that the as-built waterproof system (mudmat-waterproofing-mudmat interface) has a minimum coefficient of friction of 0.7 as demonstrated through material qualification testing.



Equipment hatches rotated out of position to show configuration: see Sheet 2 for azimuth orientation.

Note: Course layout, plate/insert plate geometry, and weld seams are shown for illustrative purposes only.

Figure 3.8.2-1 (Sheet 1 of 3)
Containment Vessel General Outline



Equipment hatch at azimuth of 126° is the lower hatch.

Equipment hatch at azimuth of 67° is the upper hatch.

Figure 3.8.2-1 (Sheet 2 of 3)
Containment Vessel General Outline

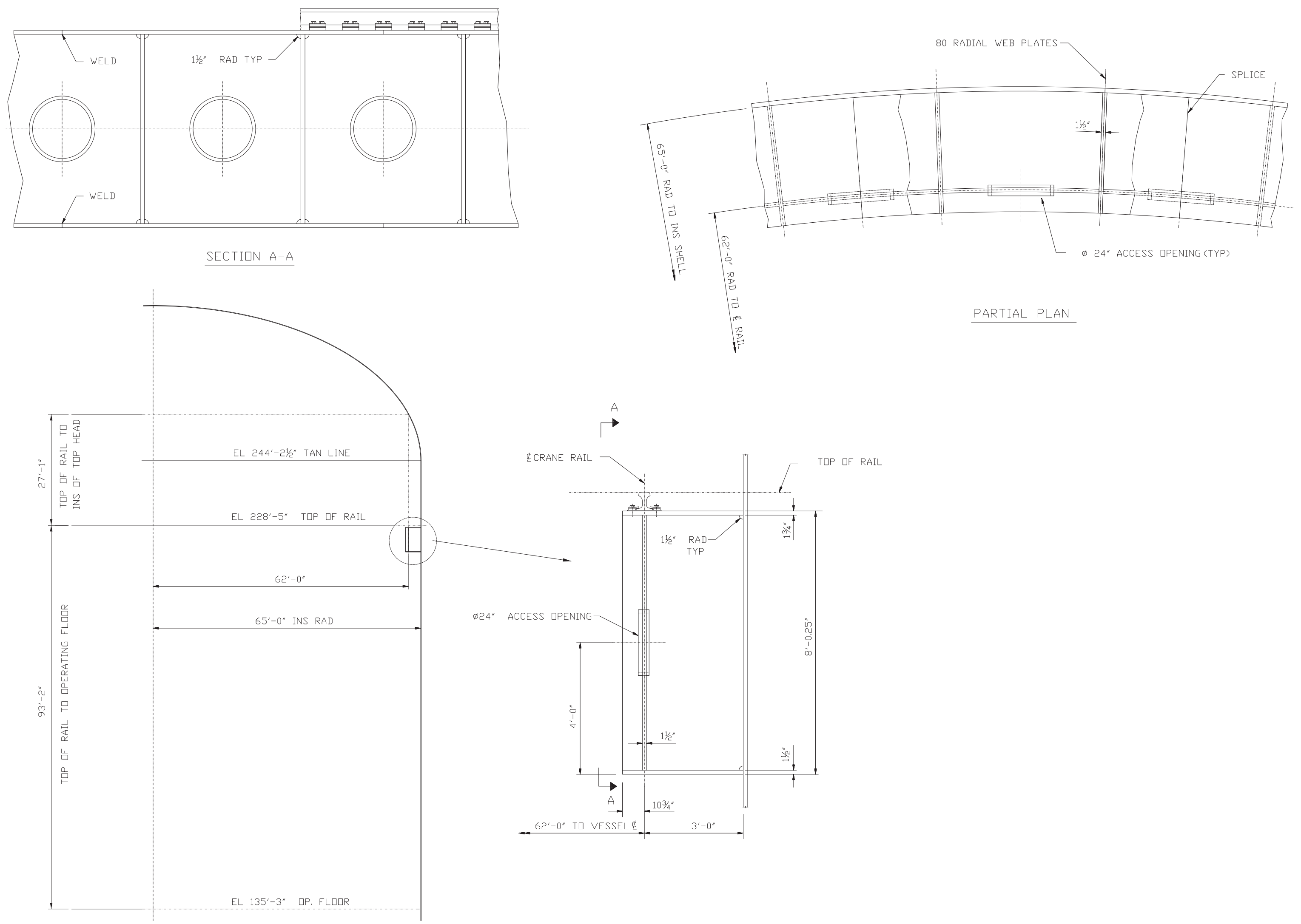
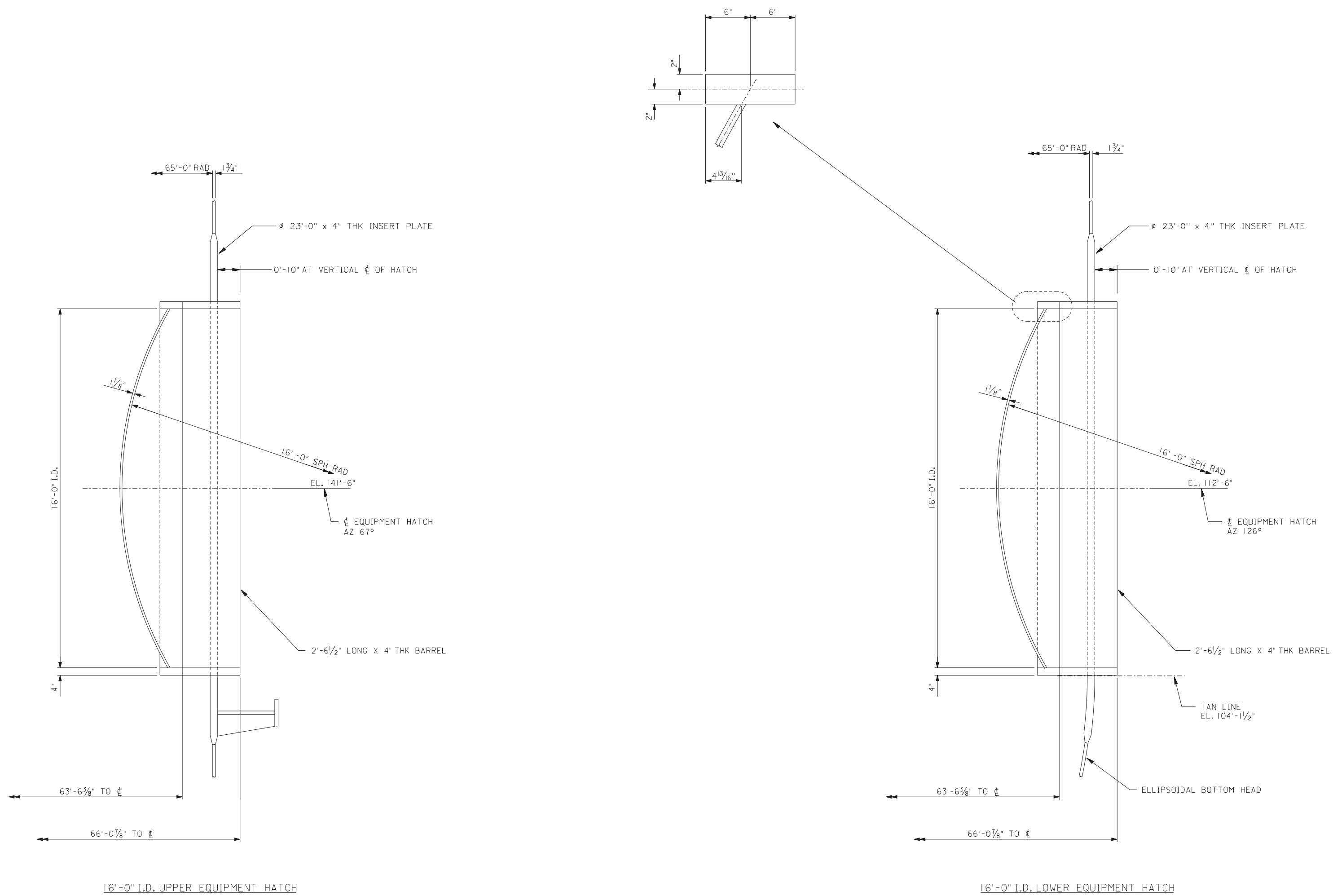


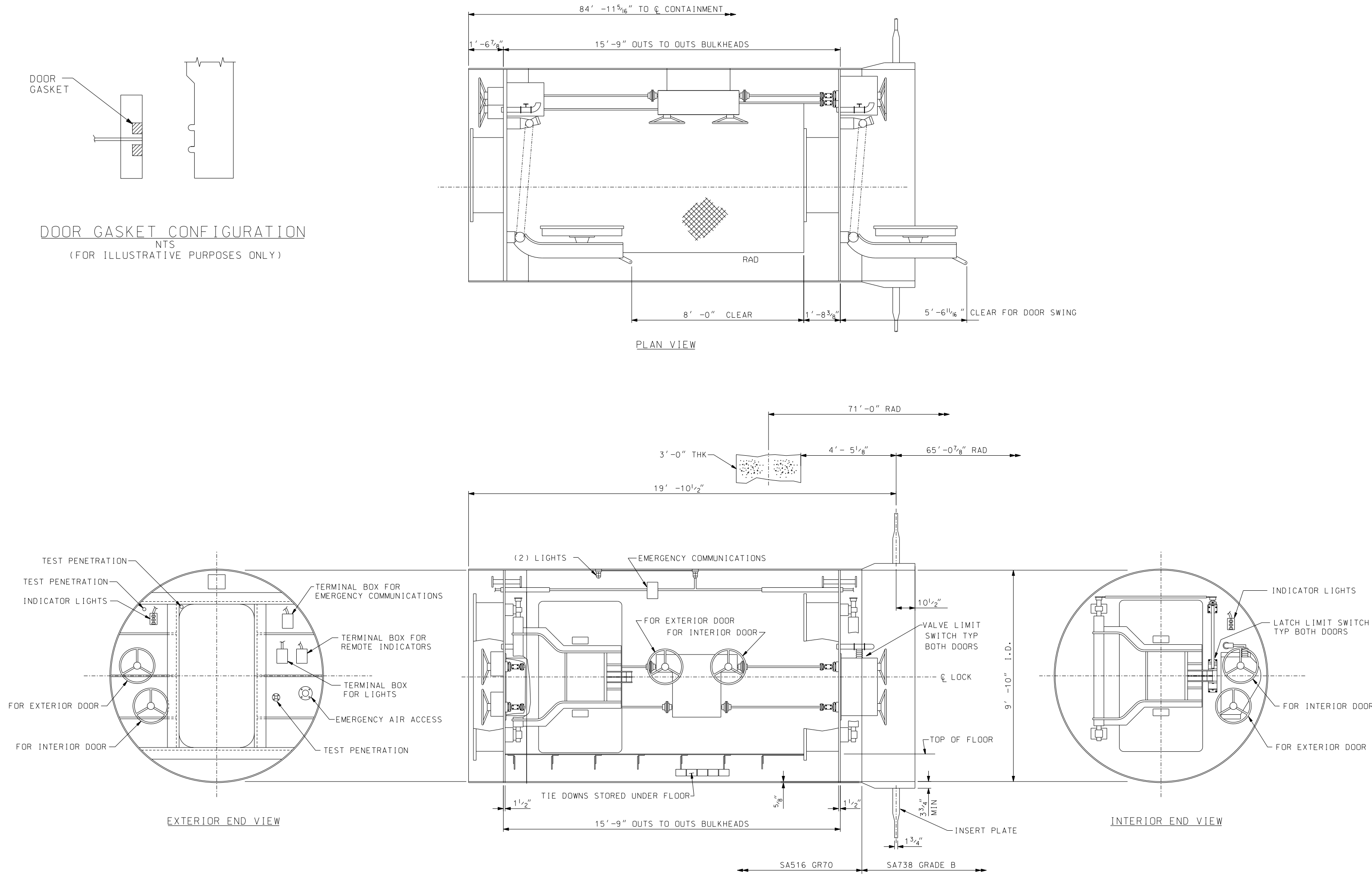
Figure 3.8.2-1 (Sheet 3 of 3)
Containment Vessel General Outline



MATERIAL: SA738 GRADE B

See subsection 3.8.2.1.3 for information that is designated as Tier 2*.

Figure 3.8.2-2
Equipment Hatches



NOTE: FOR TIER 2* DIMENSIONS, SEE SECTION 3.8.2.1.4

Figure 3.8.2-3
Personnel Airlock

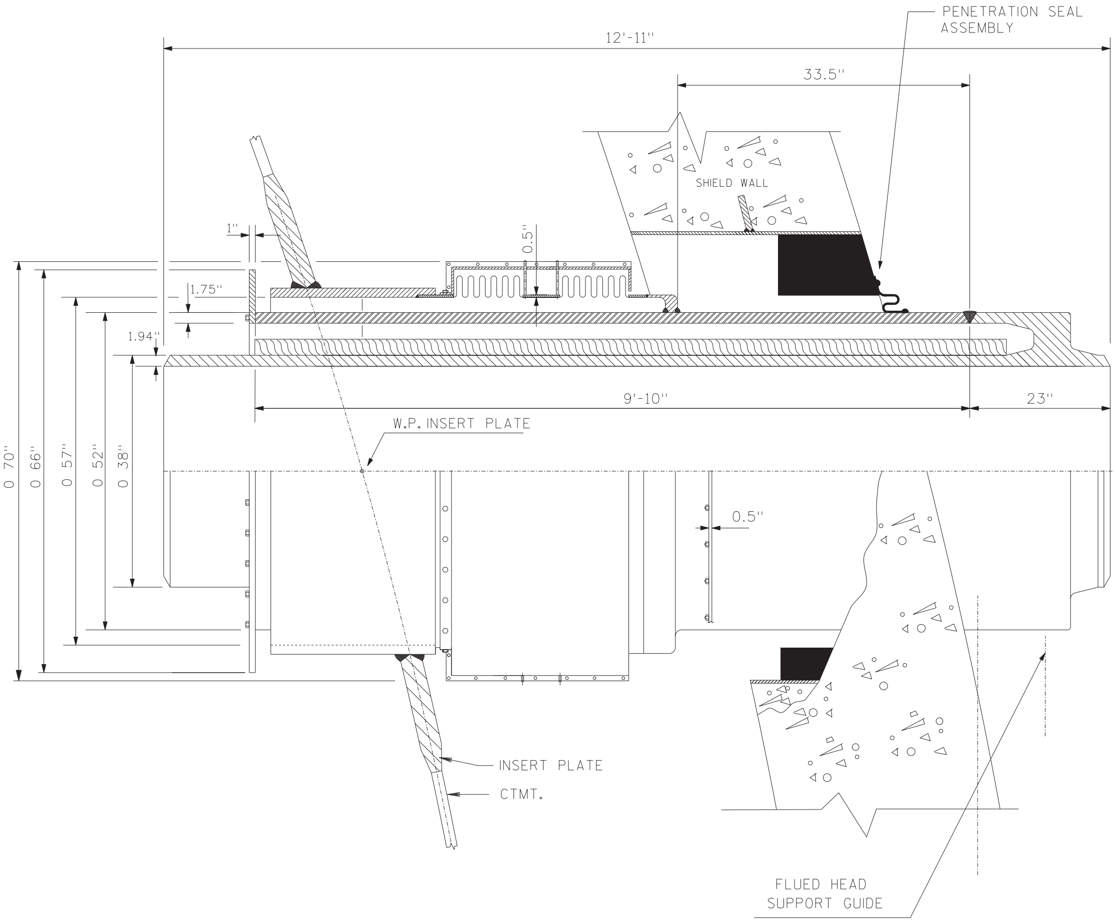


Figure 3.8.2-4 (Sheet 1 of 8)
Containment Penetrations Main Steam

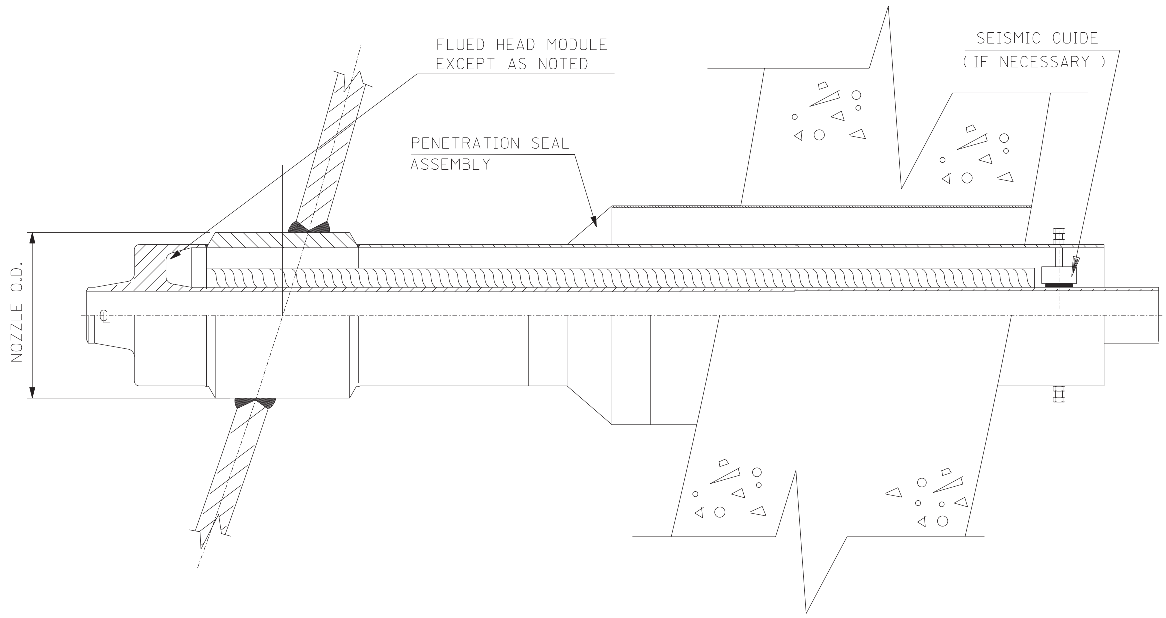
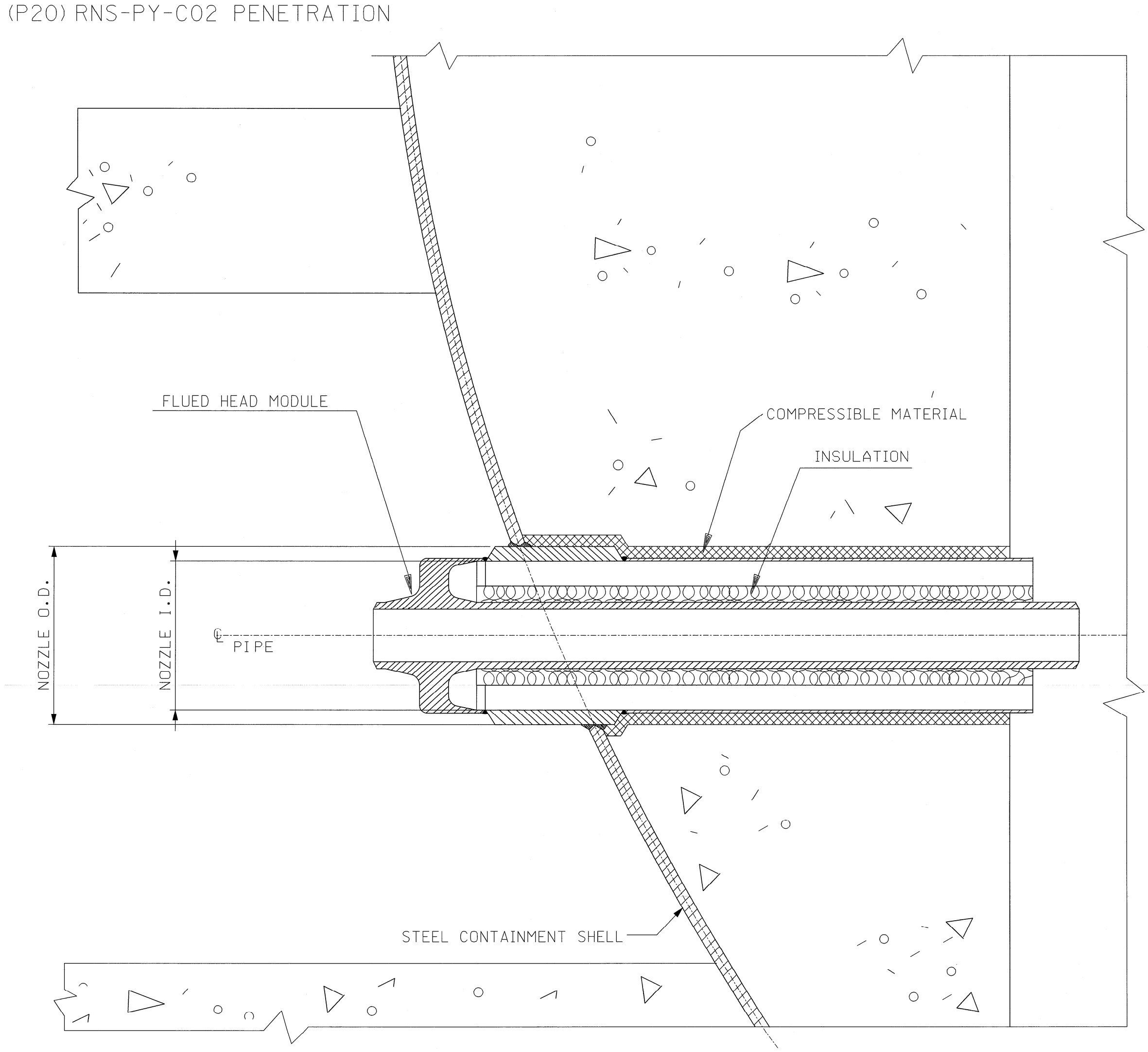


Figure 3.8.2-4 (Sheet 2 of 8)
Containment Penetrations Startup Feedwater



Note: For Vogtle Unit 3, penetration detail differs slightly for RNS-PY-C01 (P19). See Subsection 3.8.2.1.5.

Figure 3.8.2-4 (Sheet 3 of 8)
Containment Penetrations Normal RHR Piping

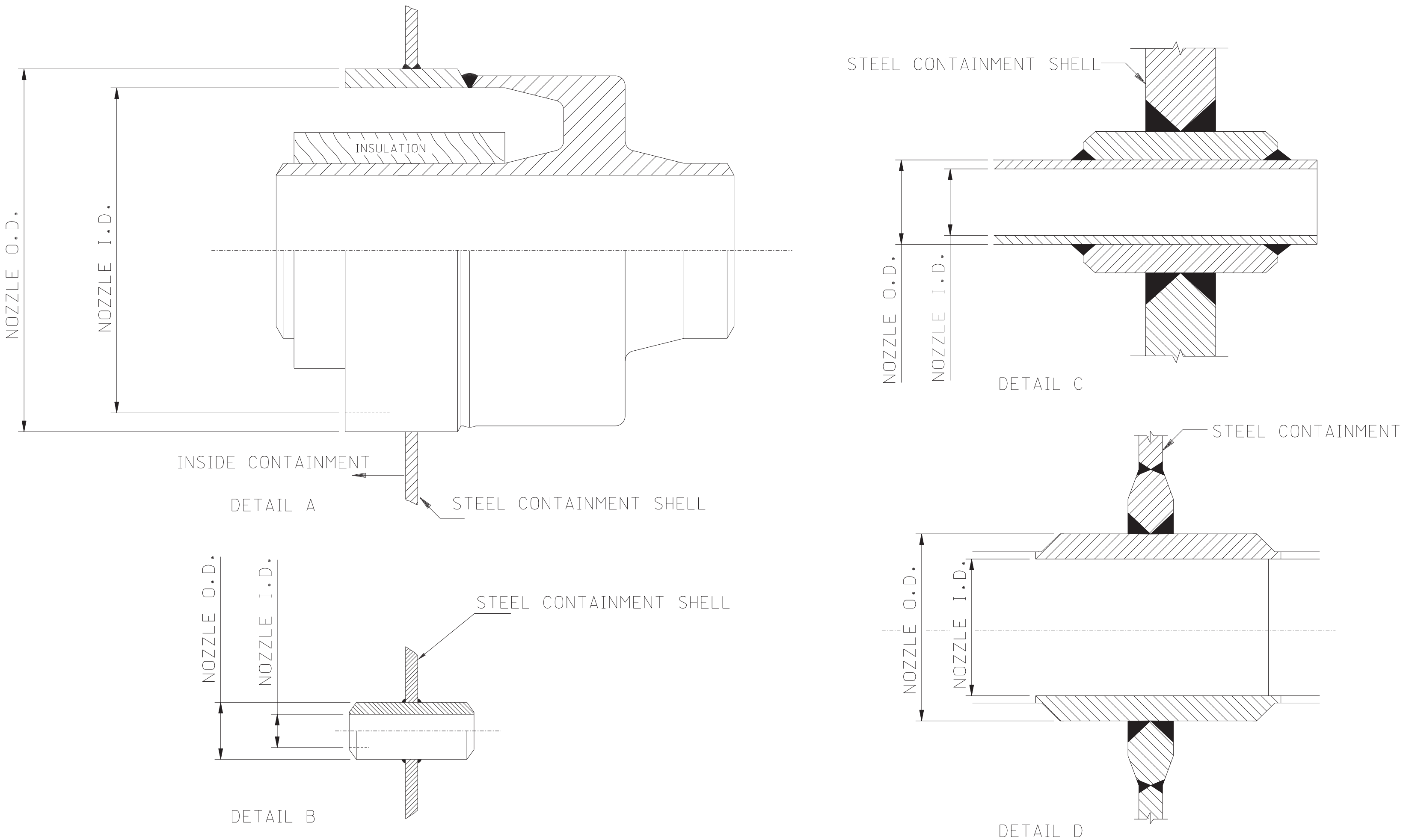


Figure 3.8.2-4 (Sheet 4 of 8)
Containment Penetrations

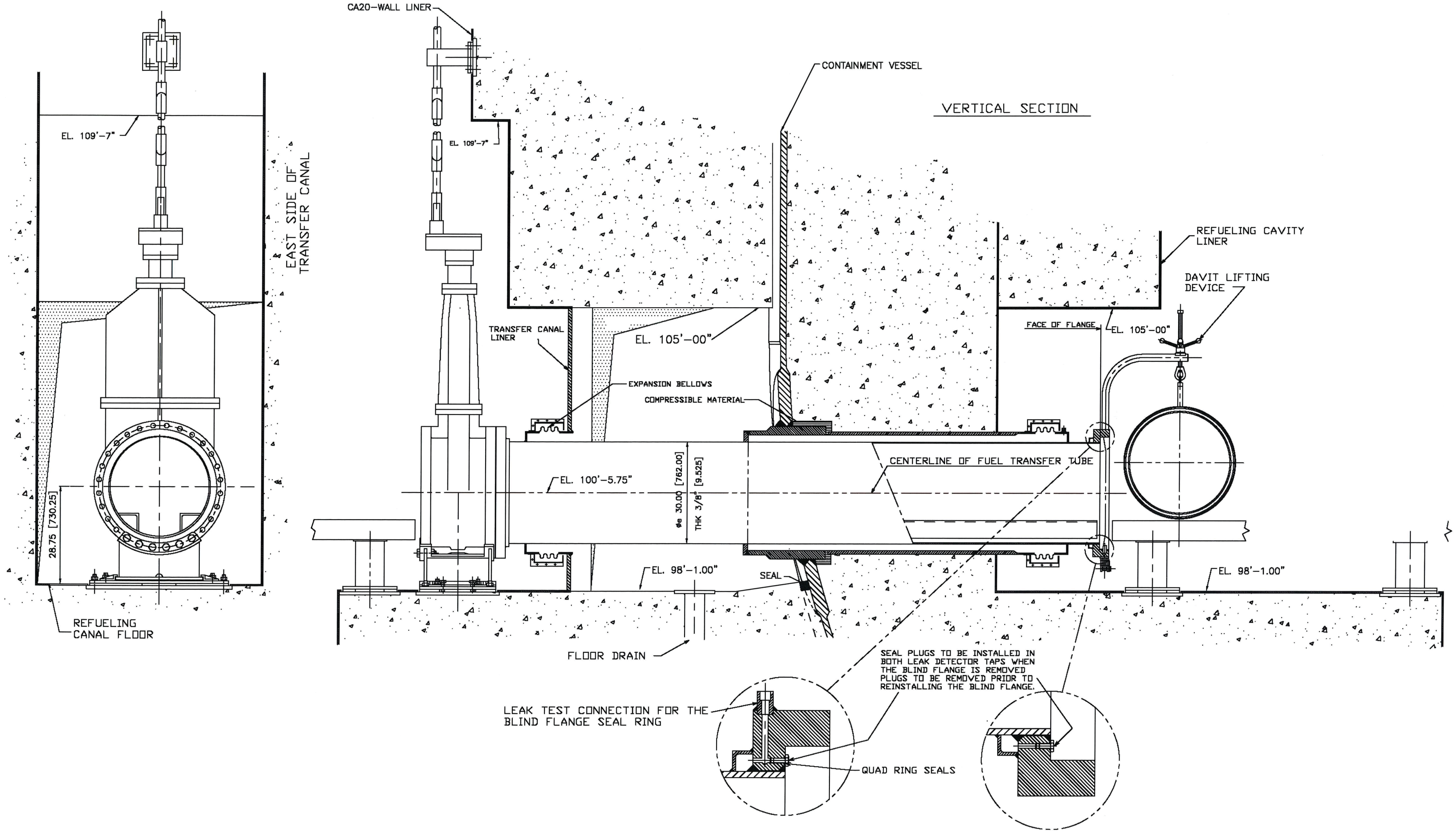


Figure 3.8.2-4 (Sheet 5 of 8)
Containment Penetrations
Fuel Transfer Penetration

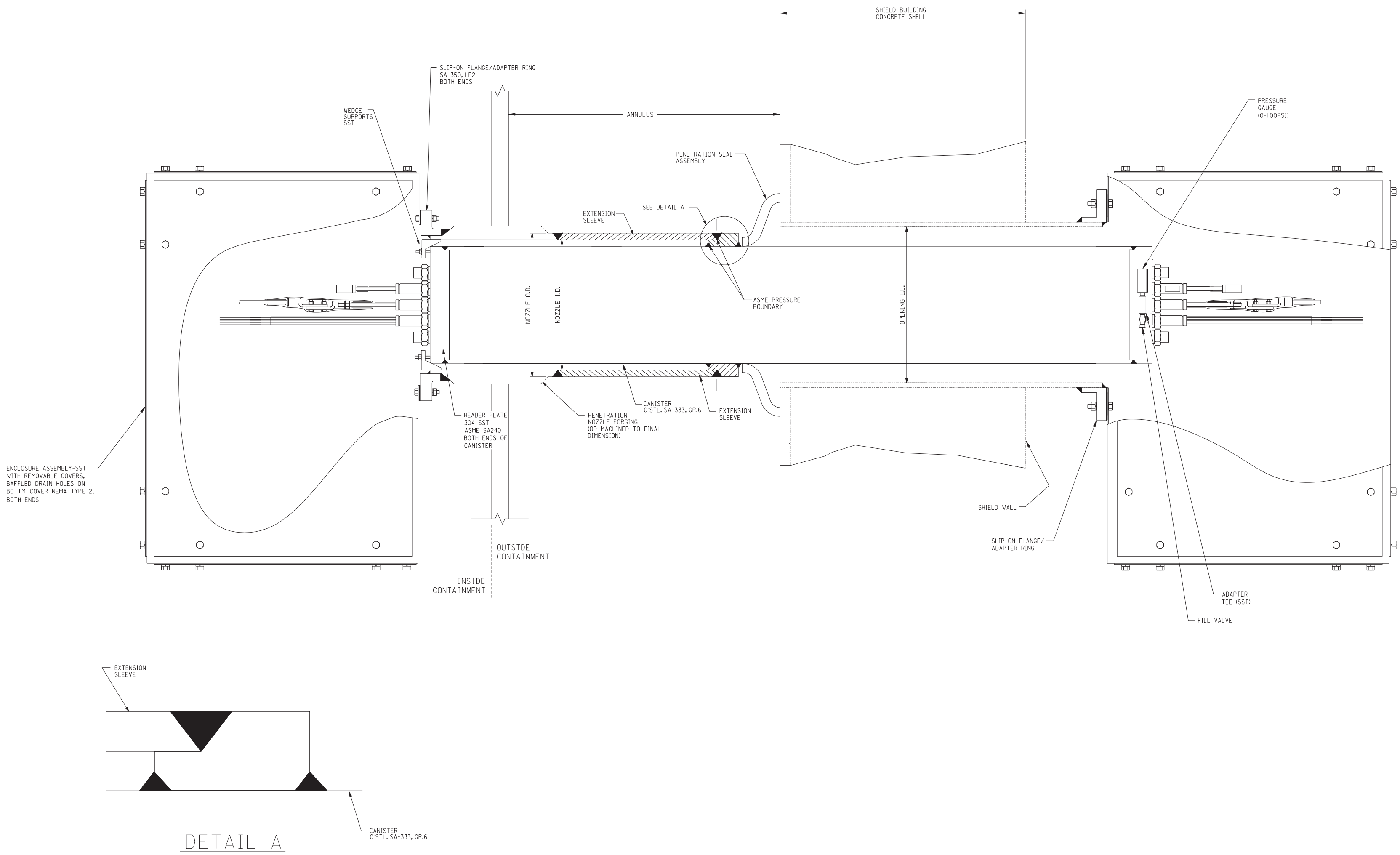


Figure 3.8.2-4 (Sheet 6 of 8)
Containment Penetrations
Typical Electrical Penetration

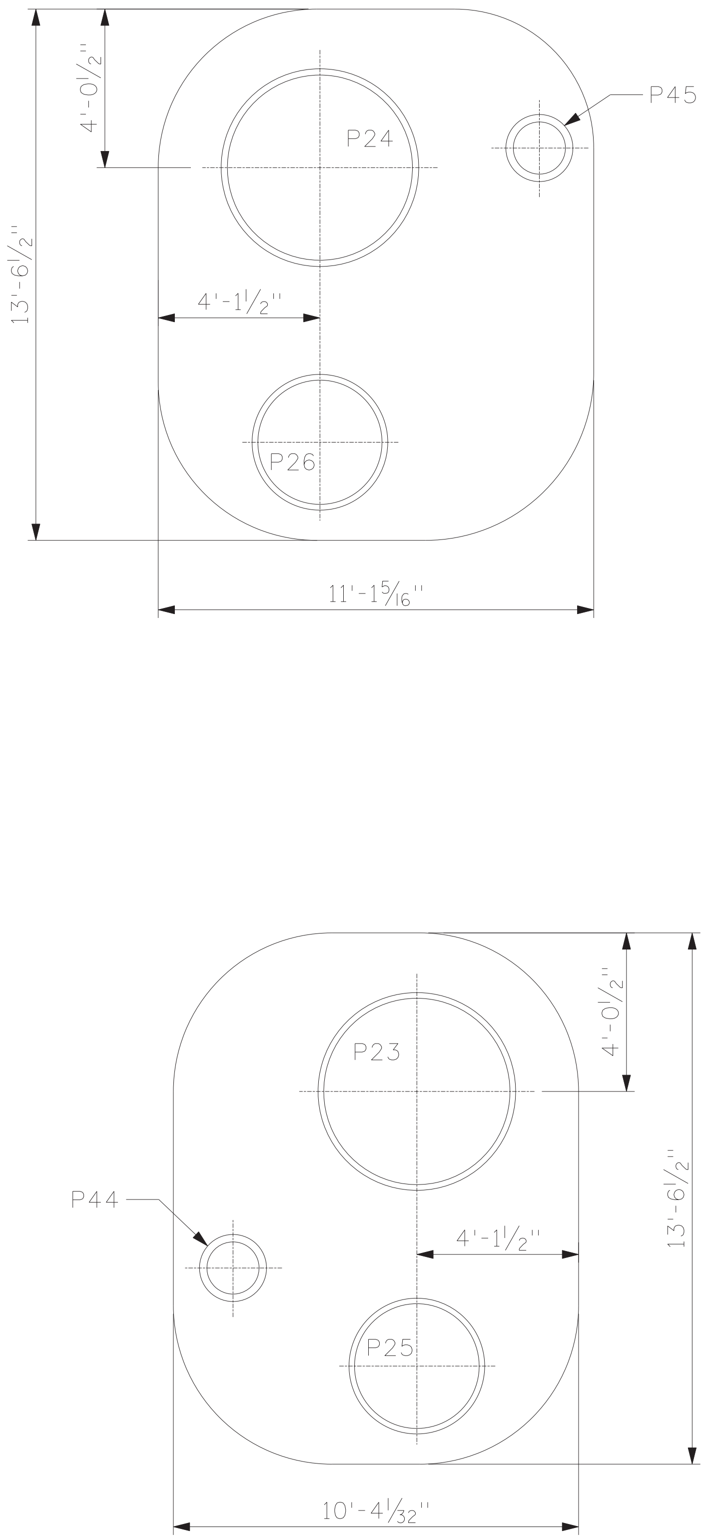


Figure 3.8.2-4 (Sheet 7 of 8)
Containment Penetrations
Steam Line and Feedwater Line Insert Plates

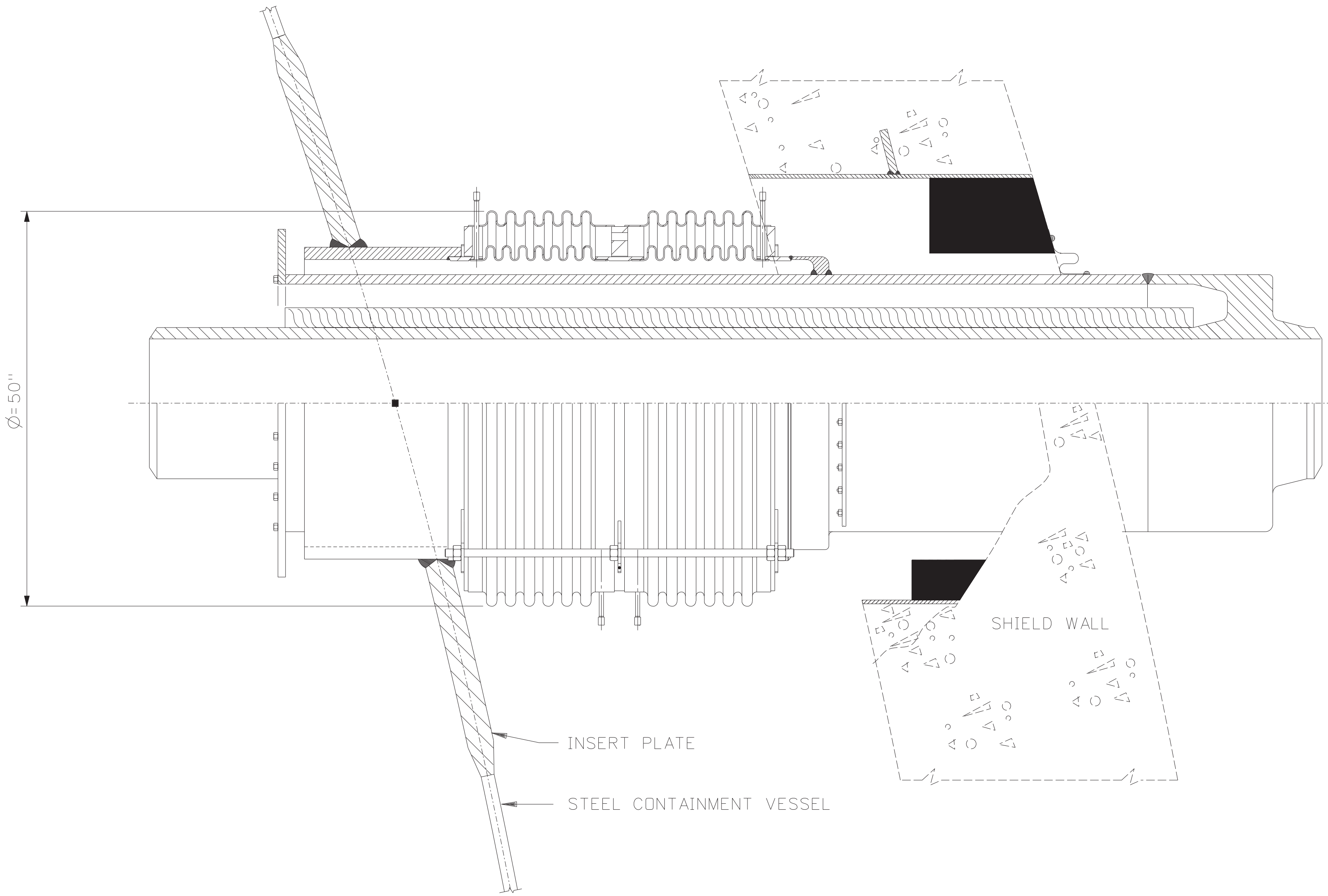


Figure 3.8.2-4 (Sheet 8 of 8)
Containment Penetrations Main Feedwater

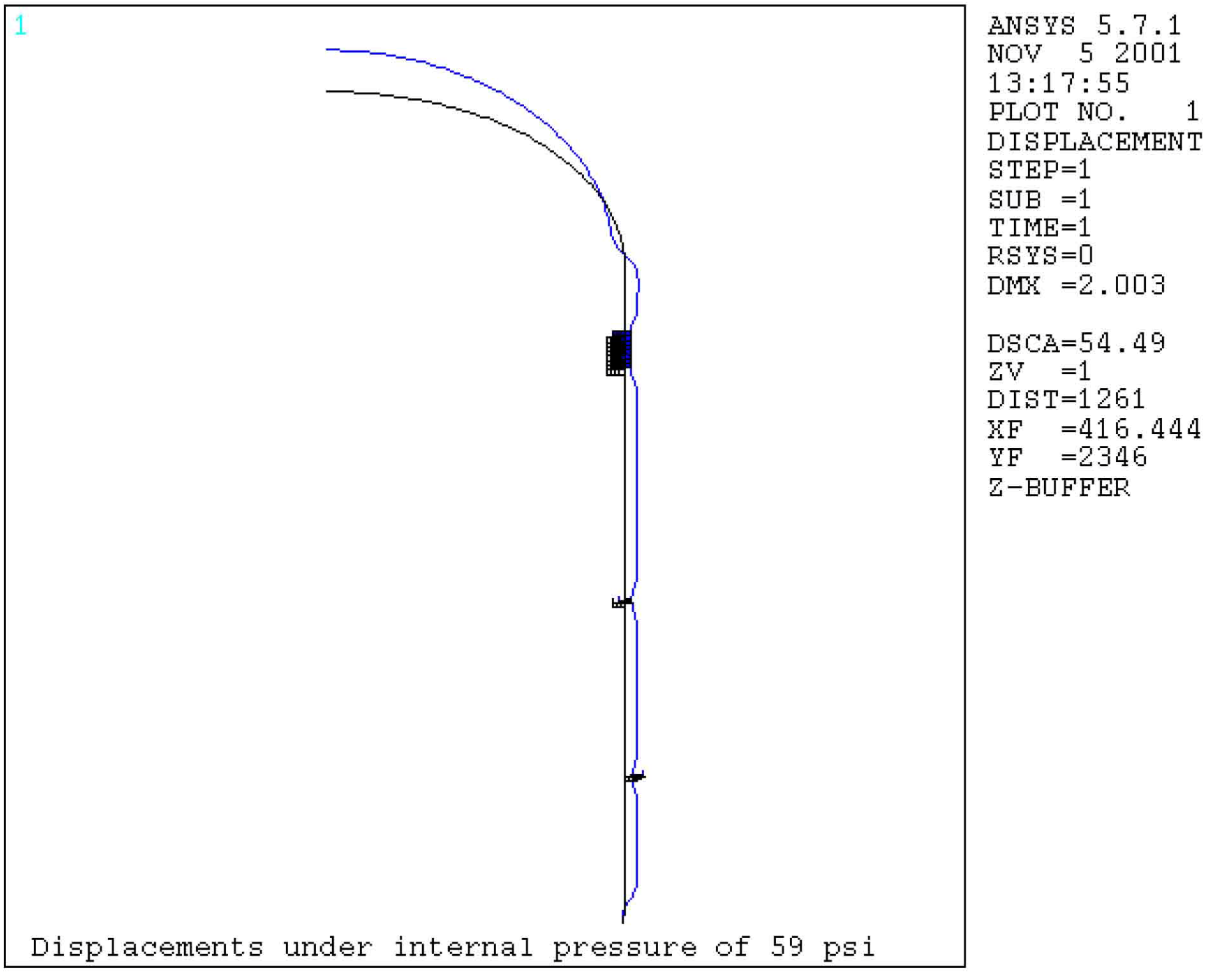
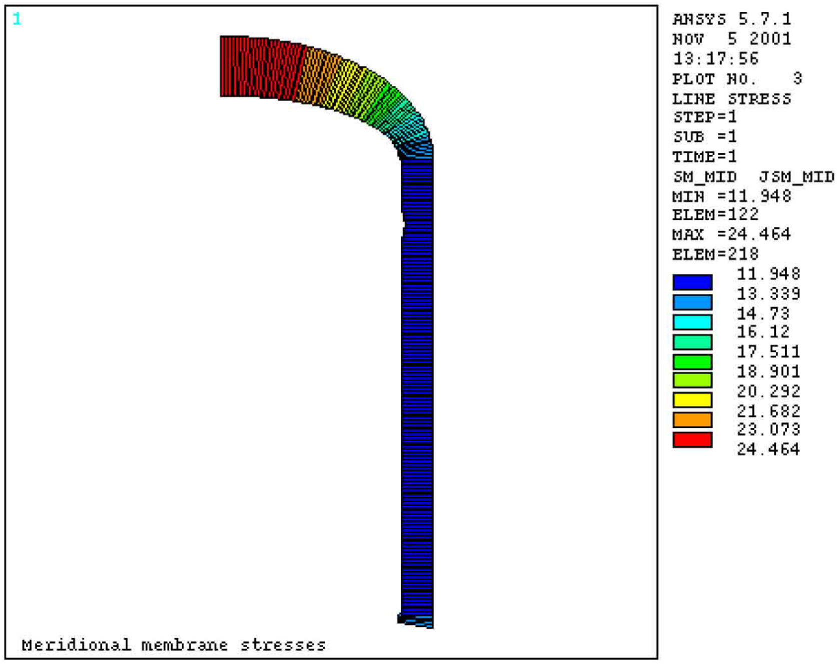
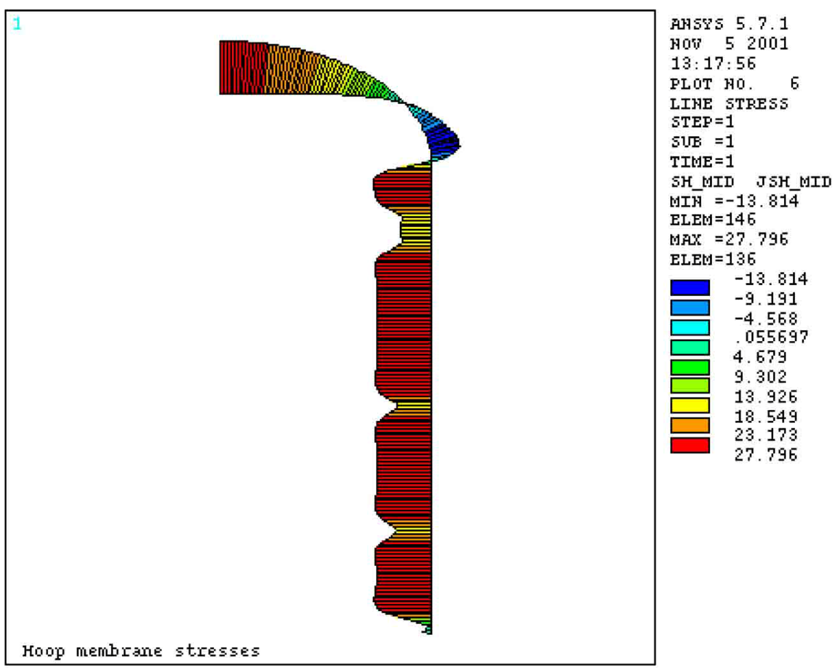


Figure 3.8.2-5 (Sheet 1 of 5)
Containment Vessel Response to Internal Pressure of 59 psig
Displaced Shape Plot



Meridional Membrane Stress (ksi)



Circumferential Membrane Stress (ksi)

Figure 3.8.2-5 (Sheet 2 of 5)
Containment Vessel Response to Internal Pressure of 59 psig
Membrane Stresses (ksi)

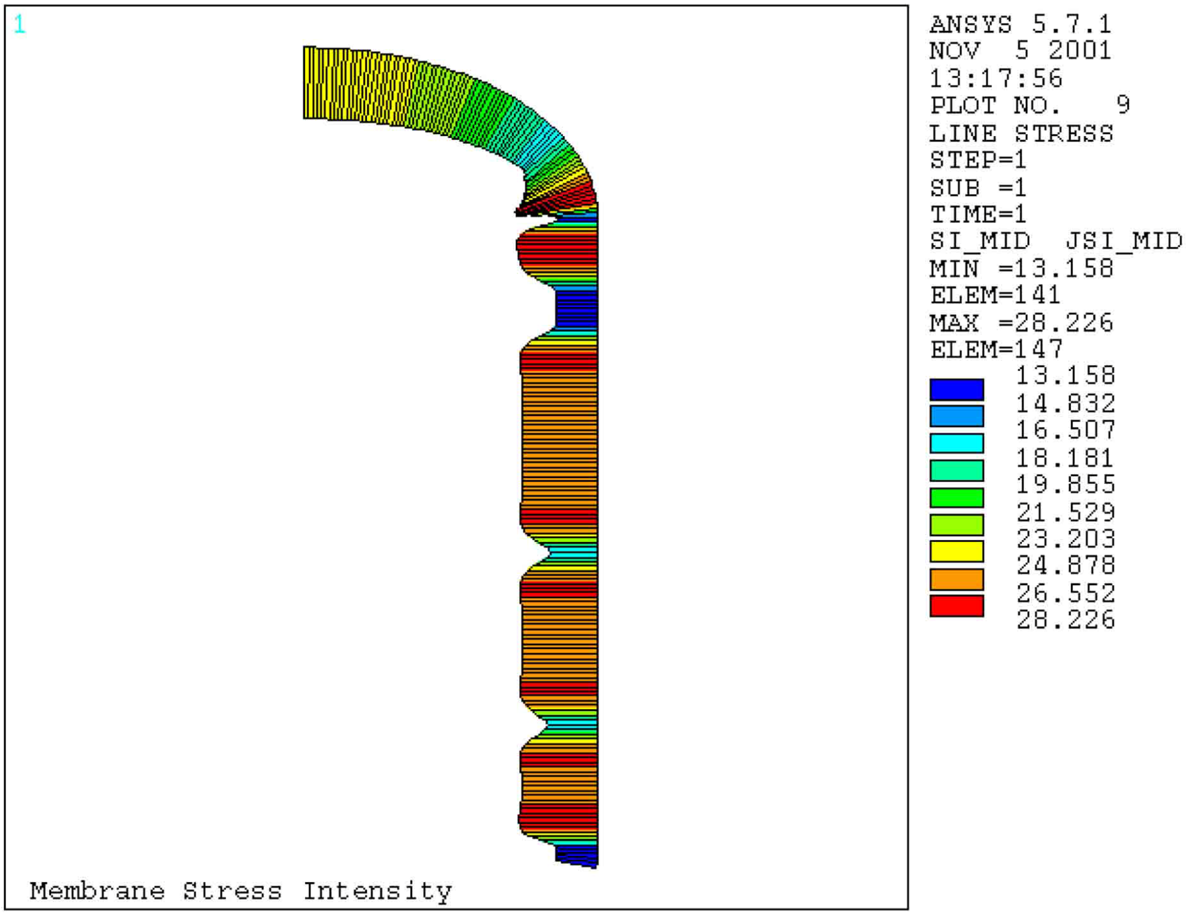
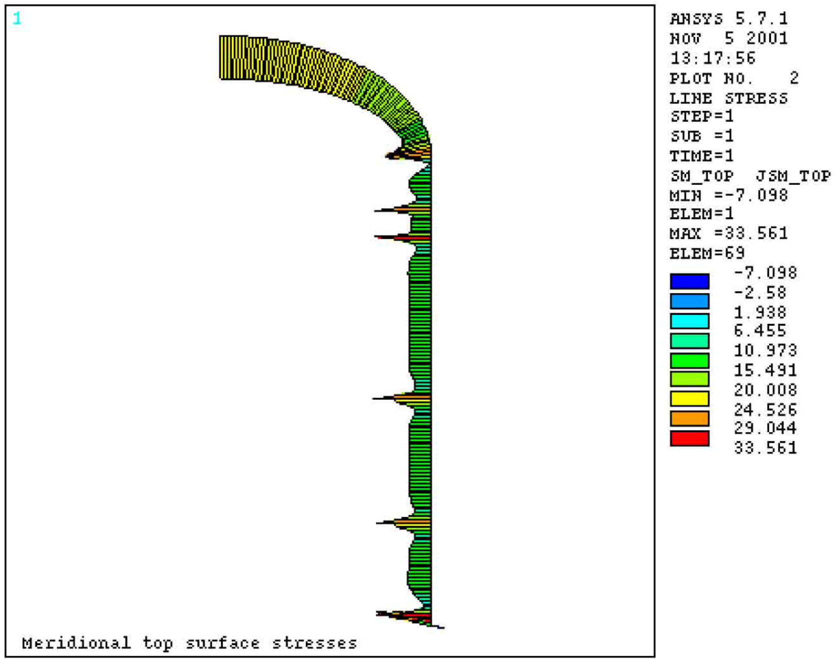
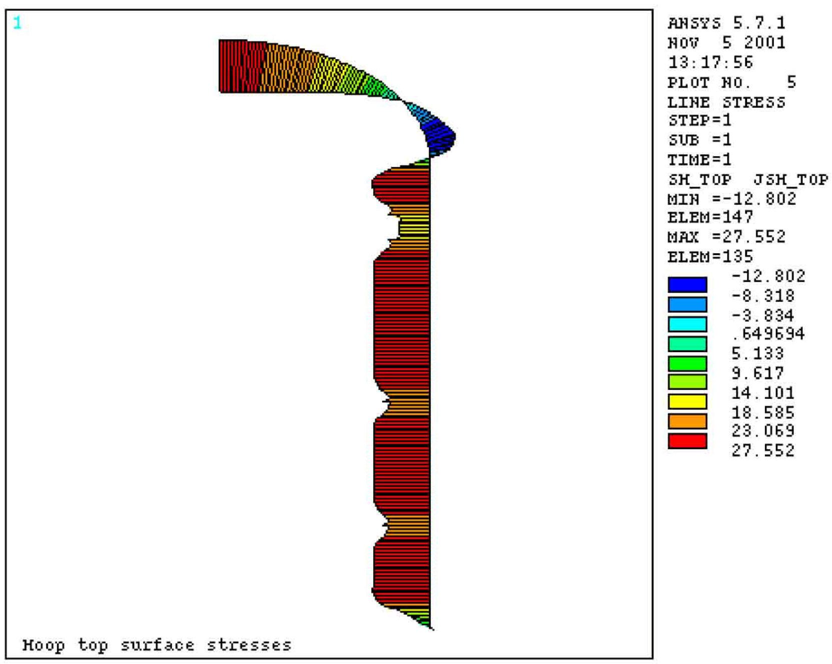


Figure 3.8.2-5 (Sheet 3 of 5)
Containment Vessel Response to Internal Pressure of 59 psig
Surface Meridional Stress (ksi)



Meridional Top Surface Stress (ksi)



Circumferential Top Surface Stress (ksi)

Figure 3.8.2-5 (Sheet 4 of 5)
Containment Vessel Response to Internal Pressure of 59 psig
Outside Surface Stresses (ksi)

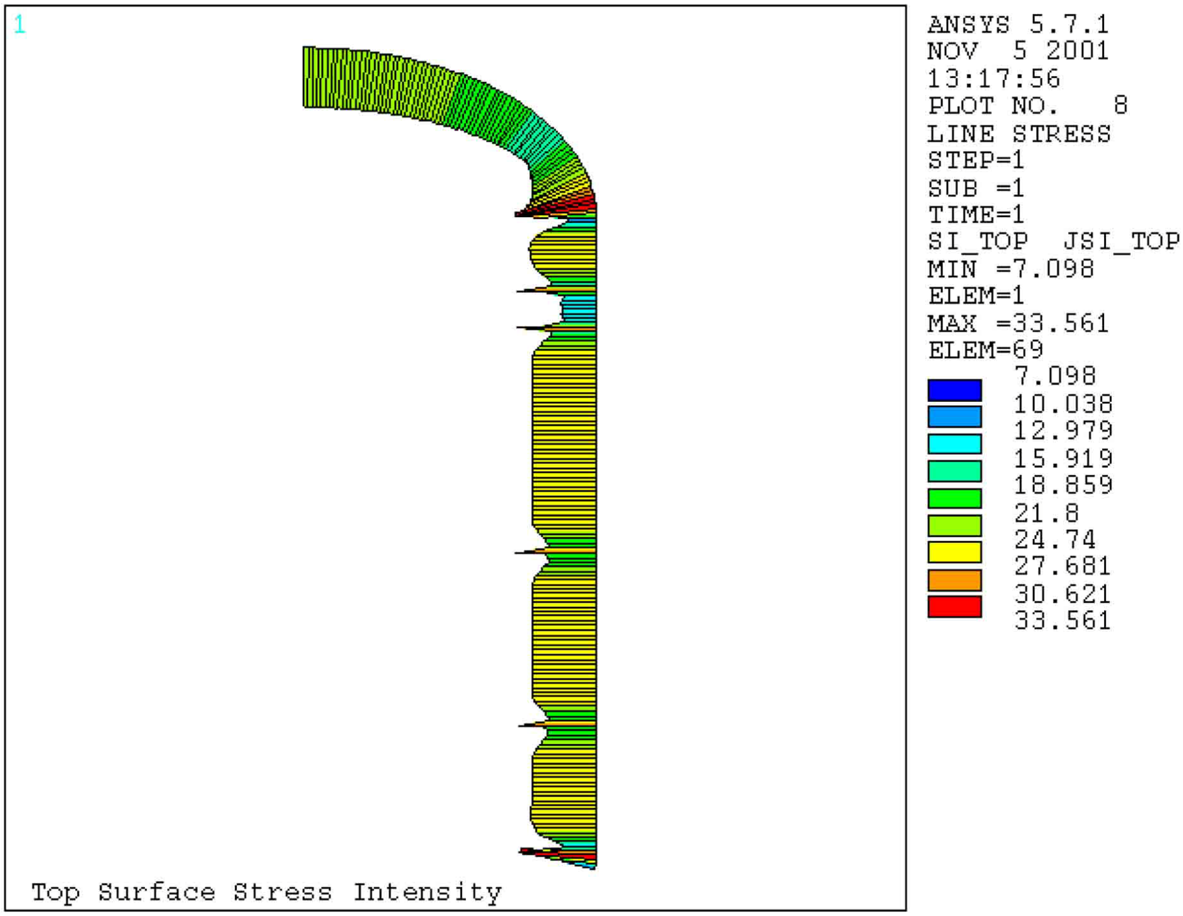


Figure 3.8.2-5 (Sheet 5 of 5)
Containment Vessel Response to Internal Pressure of 59 psig
Outer Stress Intensity (ksi)

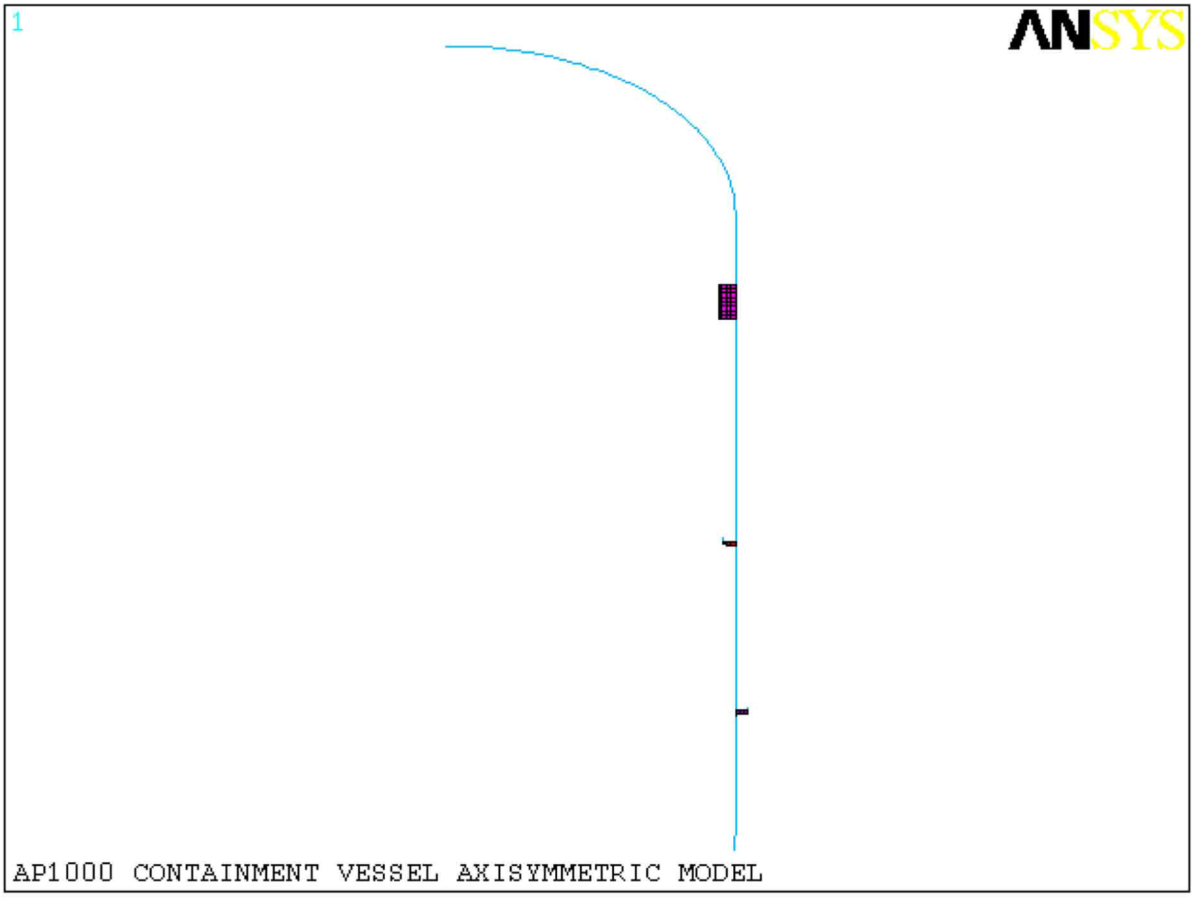
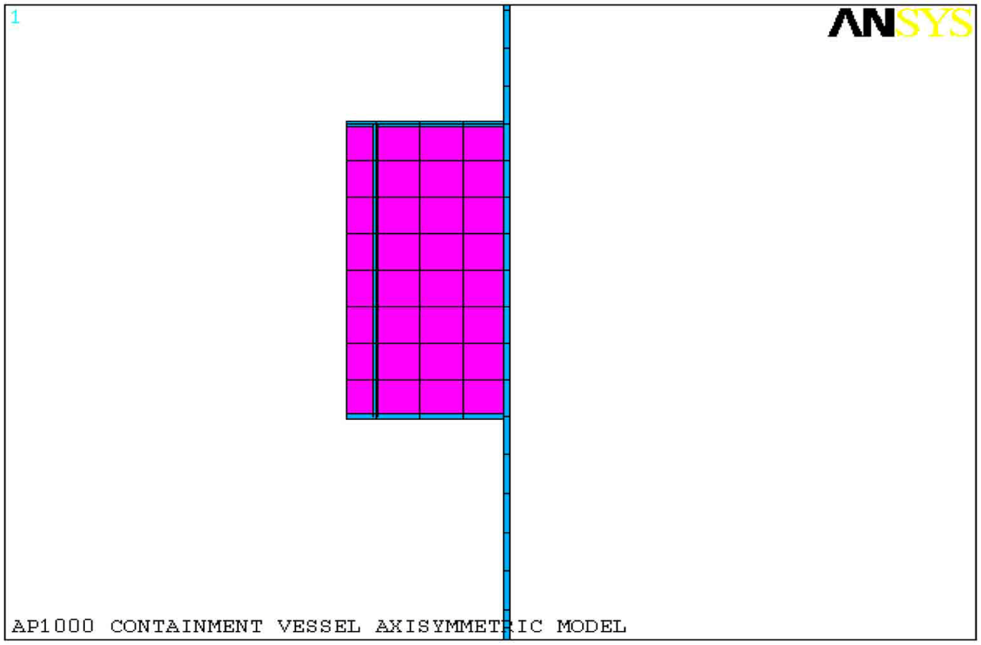
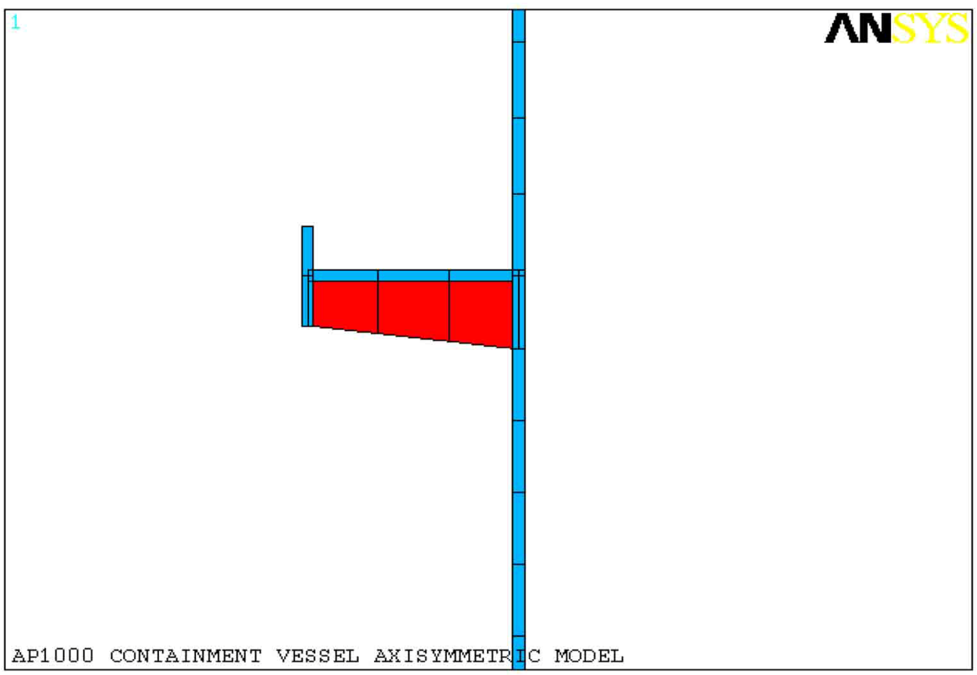


Figure 3.8.2-6 (Sheet 1 of 2)
Containment Vessel Axisymmetric Model



Crane Girder



Internal Stiffener at Elev. 170′-0″

Figure 3.8.2-6 (Sheet 2 of 2)
Containment Vessel Axisymmetric Model

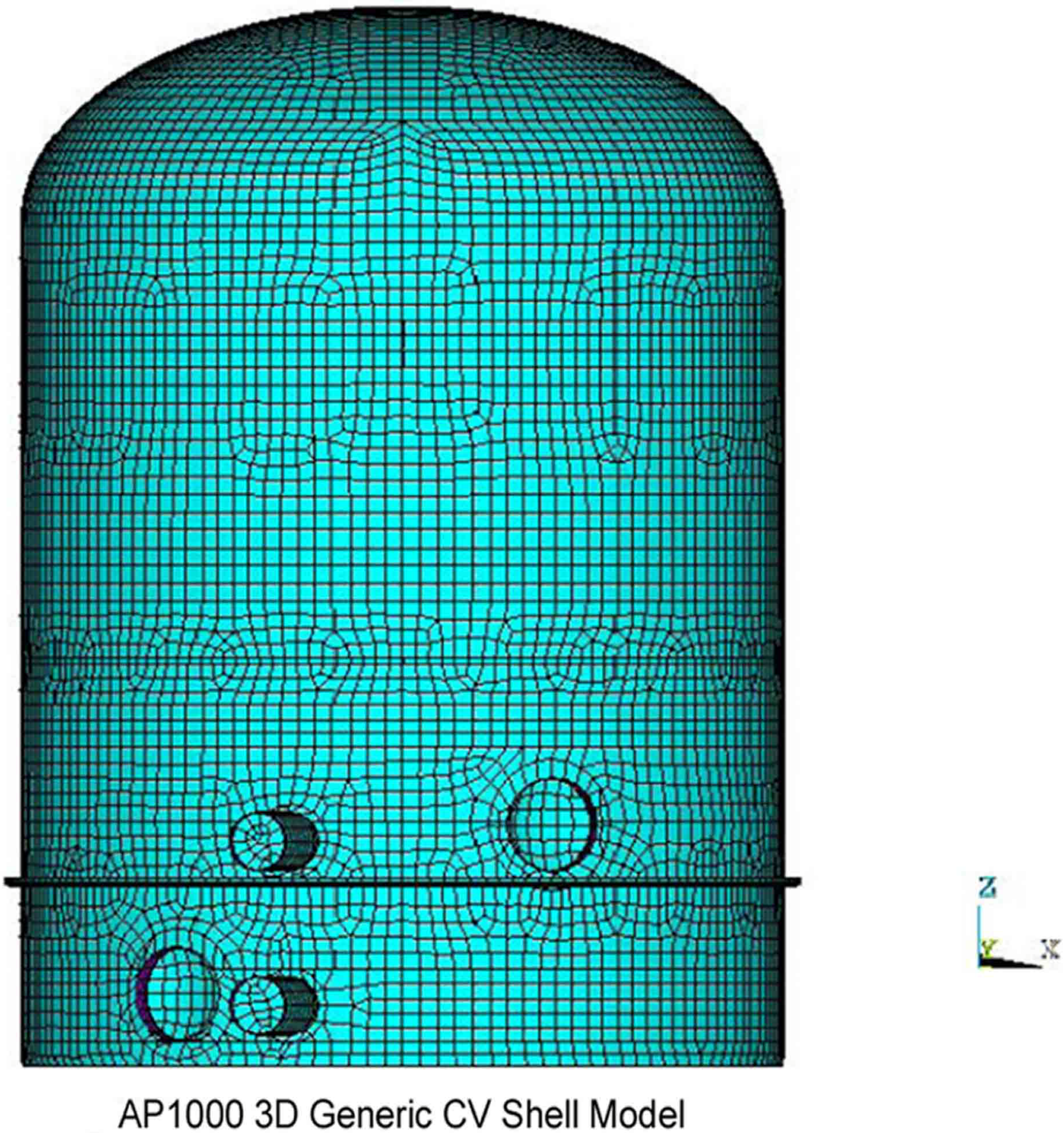


Figure 3.8.2-7
Finite Element Model for Local Buckling Analyses

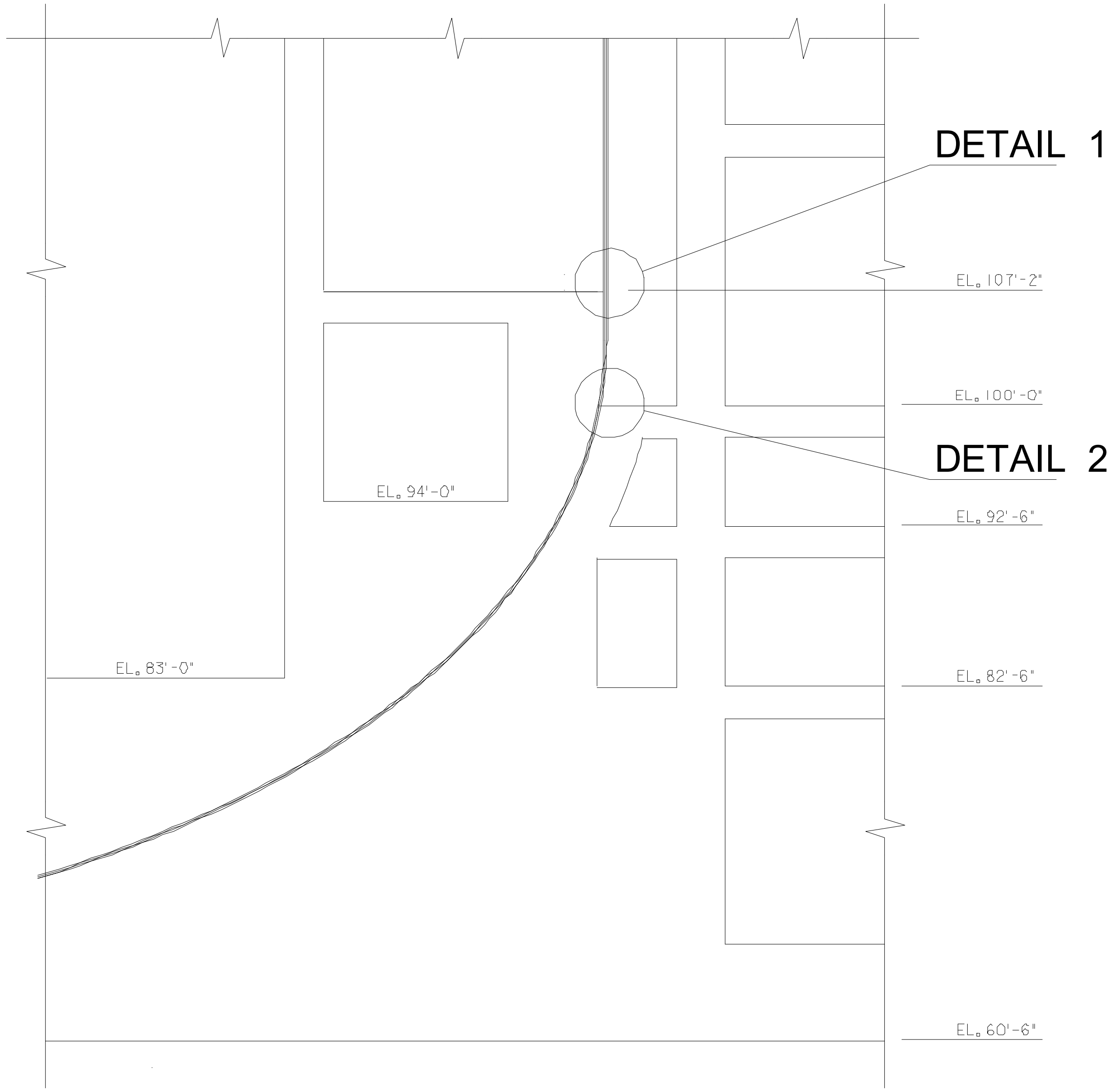
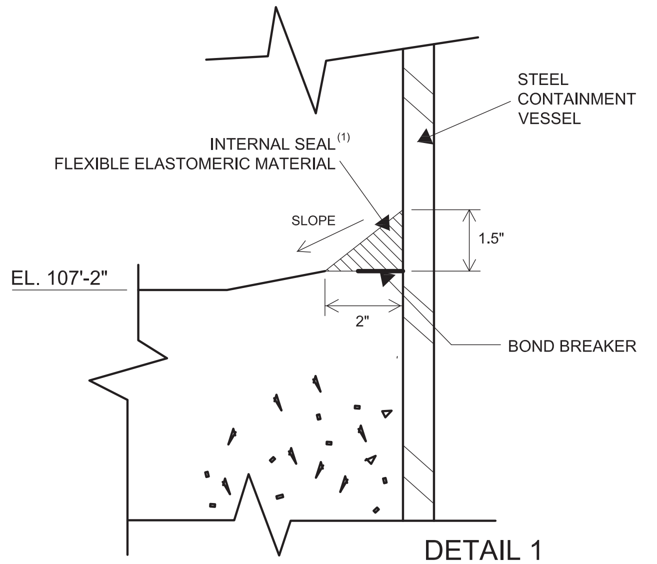
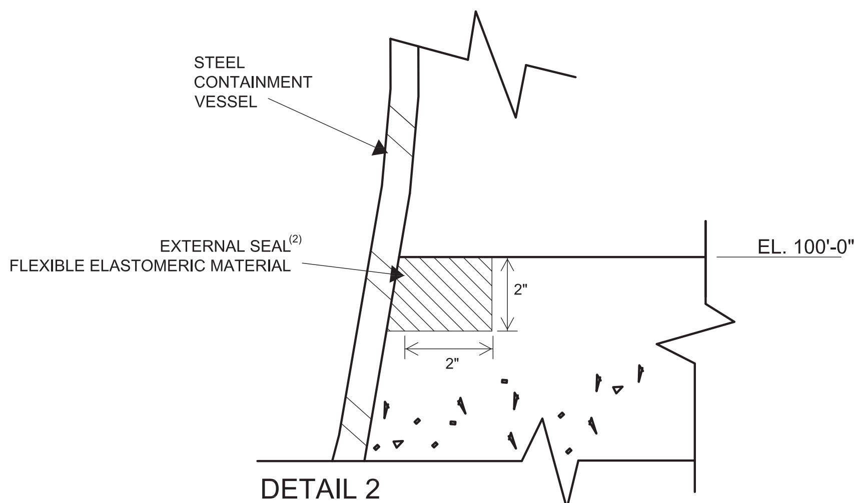


Figure 3.8.2-8 (Sheet 1 of 2)
Location of Containment Seal



DETAIL 1

(1) TO FORM SLOPE AWAY FROM CV



DETAIL 2

(2) PREFORMED FOR CONCRETE BLOCK-OUT

GENERAL NOTE: DIMENSIONS, ELEVATIONS AND CONFIGURATIONS SHOWN ARE APPROXIMATE AND CAN VARY WITH DIFFERENT SEAL MATERIALS. ALTERNATE SEAL CONFIGURATIONS AT ELEVATIONS OTHER THAN TYPICAL CAN BE UTILIZED WHERE REQUIRED TO ACCOMMODATE INTERFERENCES PROVIDED THE SEAL PREVENTS MOISTURE ACCUMULATION BETWEEN THE VESSEL AND CONCRETE.

Figure 3.8.2-8 (Sheet 2 of 2)
Seal Sections and Details

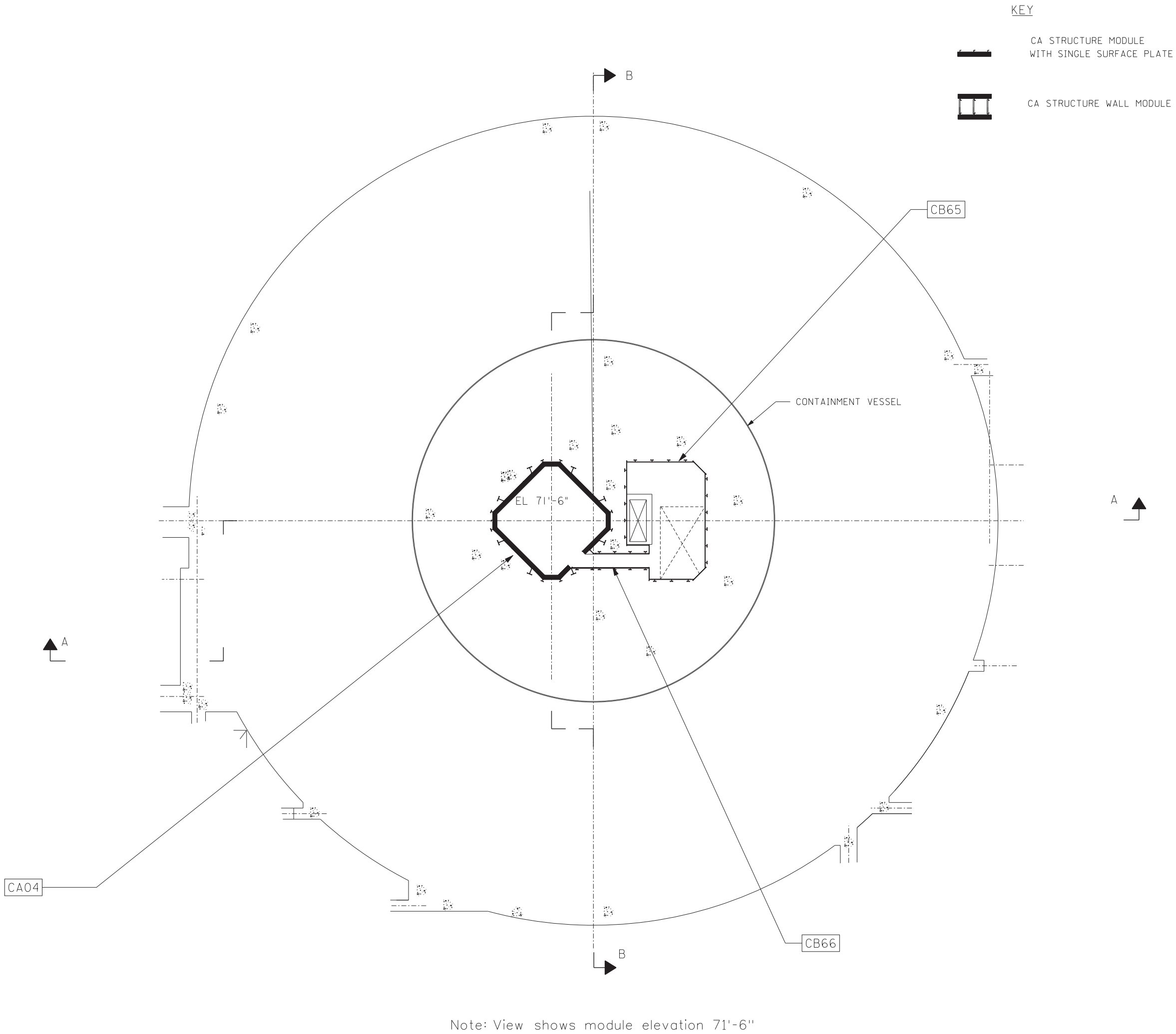


Figure 3.8.3-1 (Sheet 1 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.8.3-1 (Sheet 2 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

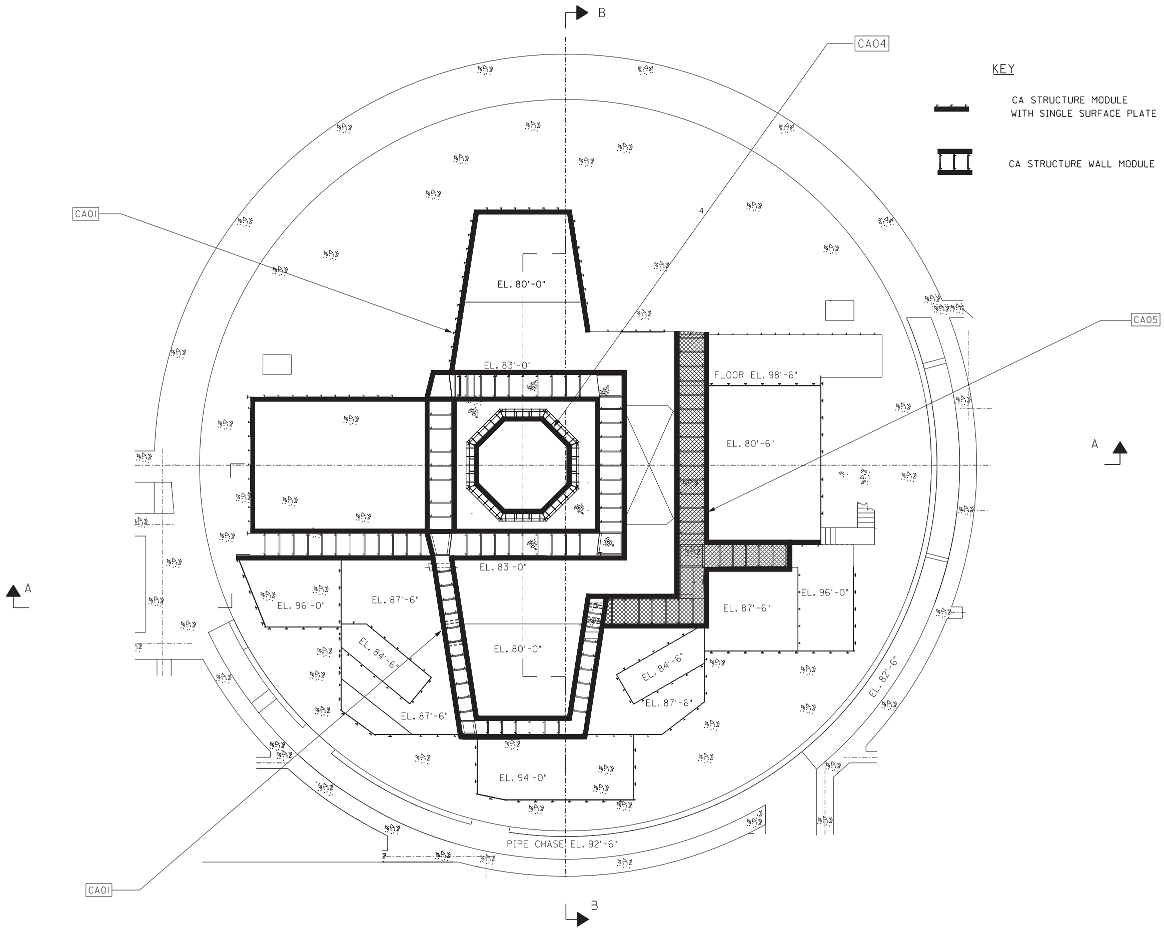


Figure 3.8.3-1 (Sheet 3 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Security-Related Information, Withheld Under 10 CFR 2.390d

Figure 3.8.3-1 (Sheet 4 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

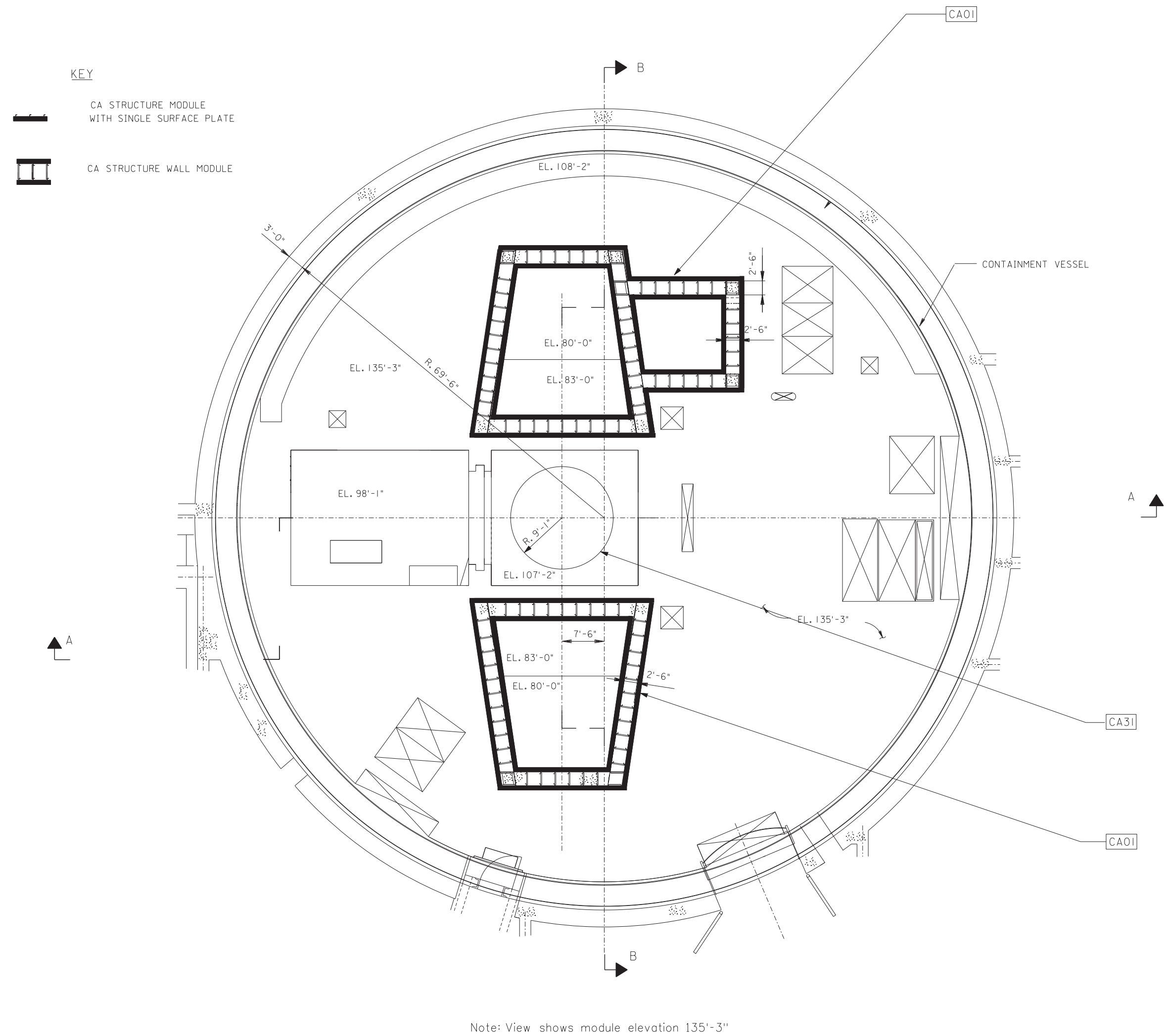


Figure 3.8.3-1 (Sheet 5 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

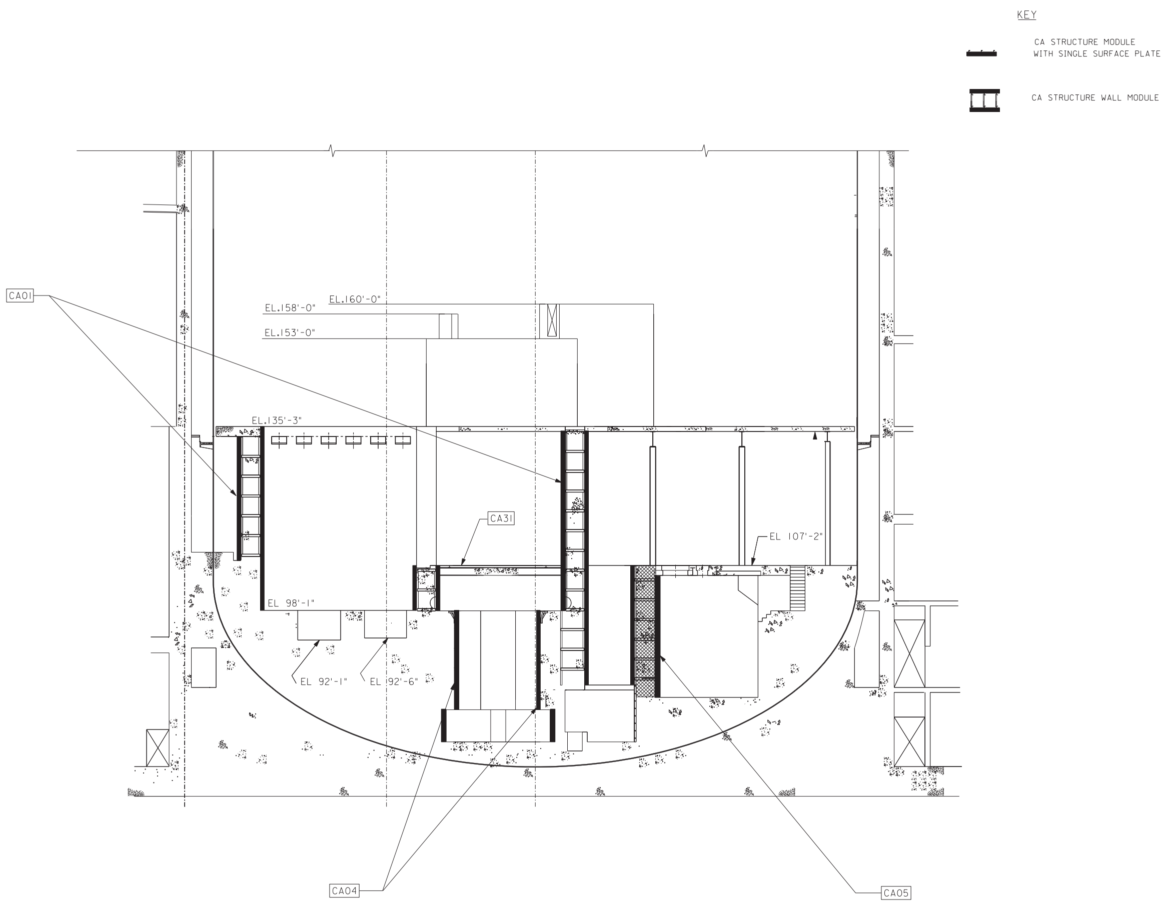


Figure 3.8.3-1 (Sheet 6 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

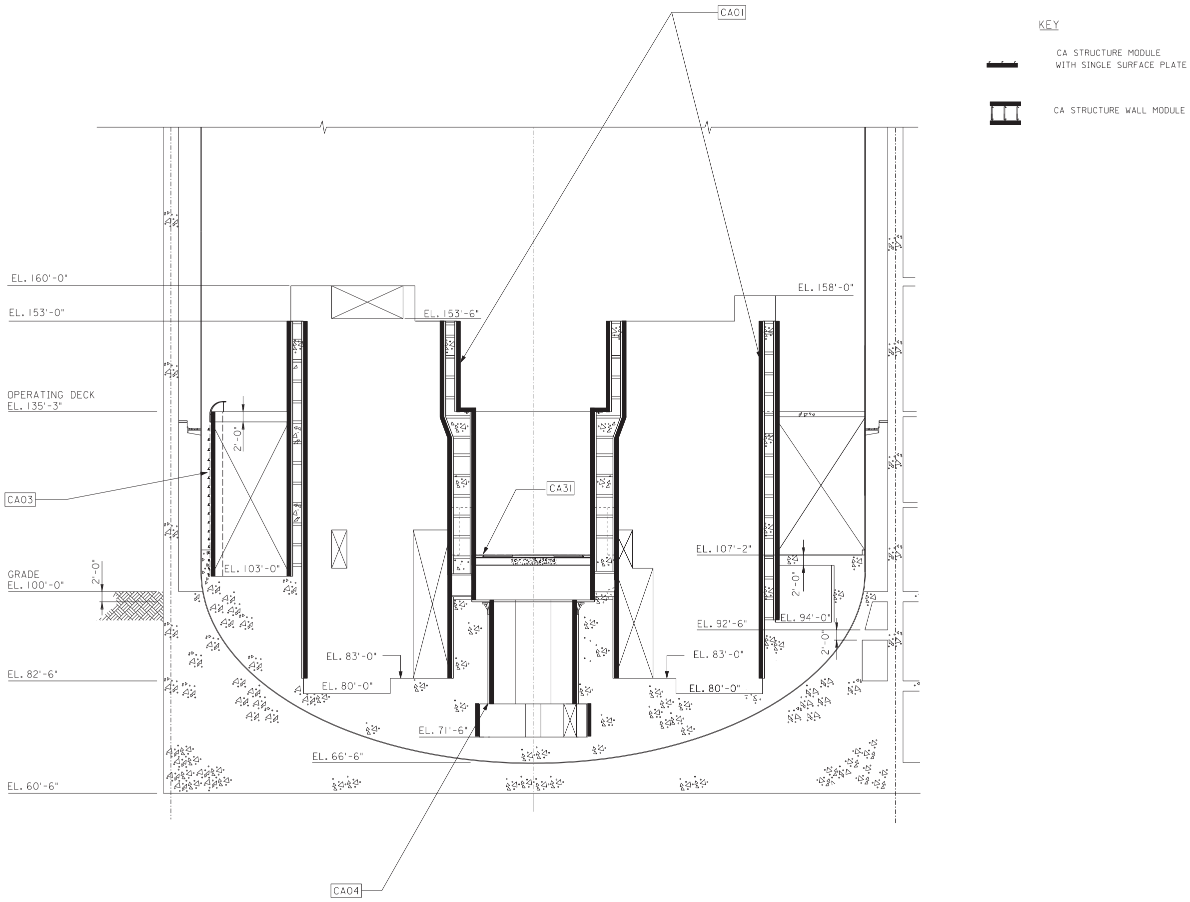
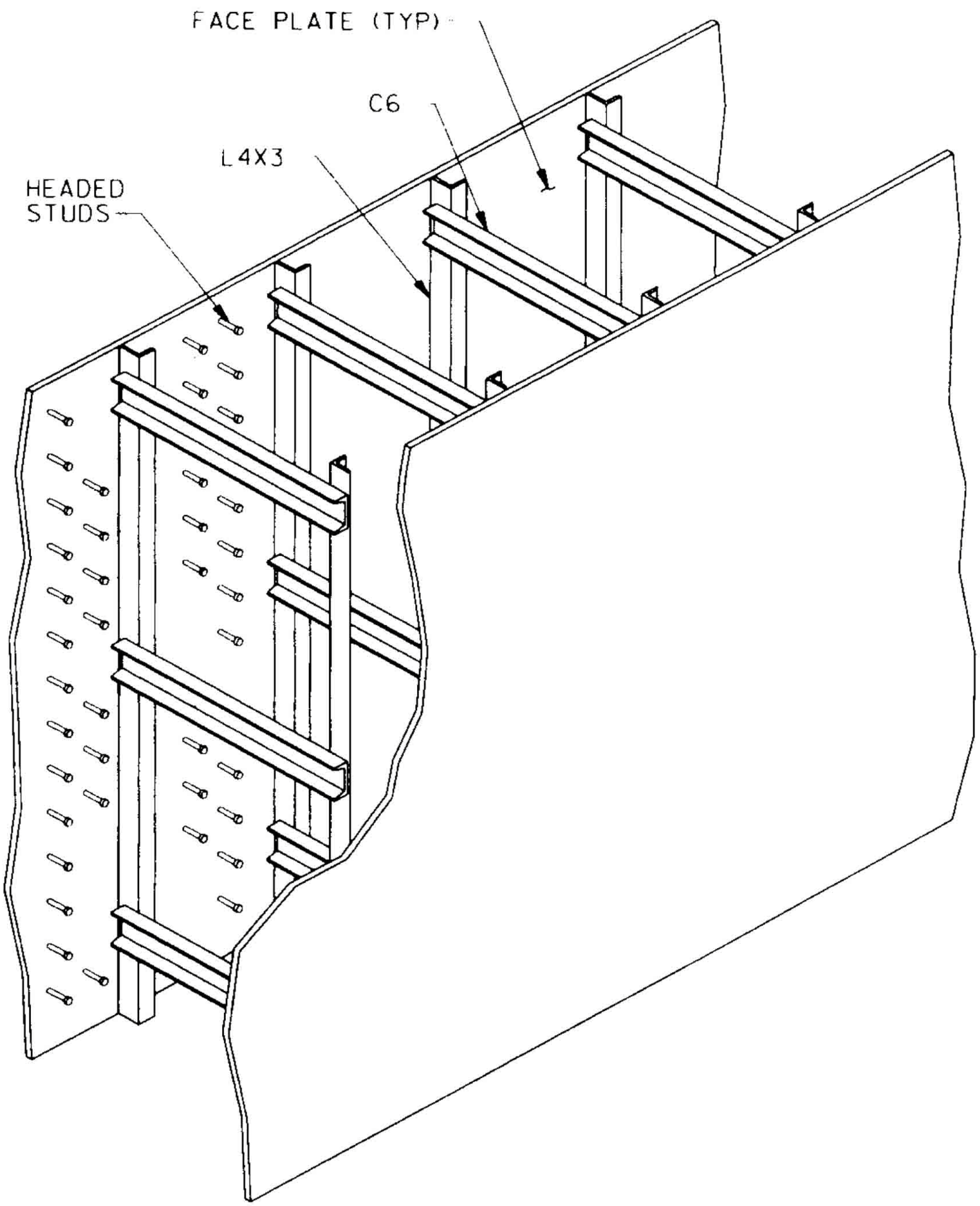


Figure 3.8.3-1 (Sheet 7 of 7)
[*Structural Modules in Containment Internal Structures*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



Note: See Figure 3.8.3-8 for fabrication detail.

Figure 3.8.3-2
[*Typical Structural Wall Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

NOTES:

1. DETAILS SHOWN ARE REPRESENTATIVE OF FLOOR MODULES INSIDE CONTAINMENT. CHANNELS AND WIDE FLANGE BEAMS USED IN SOME LOCATIONS ARE NOT SHOWN. REFER TO SUBSECTION 3.8.3.1.4 AND OTHER NOTES FOR ADDITIONAL INFORMATION ABOUT DESIGN DETAILS AND VARIATIONS.
2. REINFORCEMENT USE, SIZE, AND SPACING IN THE FLOOR MODULE CONCRETE SATISFY THE REQUIREMENTS OF ACI 349. WHERE USED, THE REINFORCEMENT SIZE RANGE IS #7 TO #11.
3. THE REINFORCEMENT AND FLOOR DESIGN ELEMENTS SHOWN ARE FOR LOCATIONS AWAY FROM OPENINGS, PENETRATIONS AND OTHER OBSTRUCTIONS.
4. DESIGN OF THE PLATES, BEAMS, AND STIFFENERS IN THE FLOOR, INCLUDING PLATE SIZE AND SPACING, AND TYPE, SIZE, AND SPACING OF STRUCTURAL SHAPES VARIES AND SATISFIES THE REQUIREMENTS OF AISC N690.
5. REFER TO SUBSECTION 3.8.3.5.4 FOR INFORMATION ABOUT THE SHEAR STUDS IN THE FLOOR MODULE.

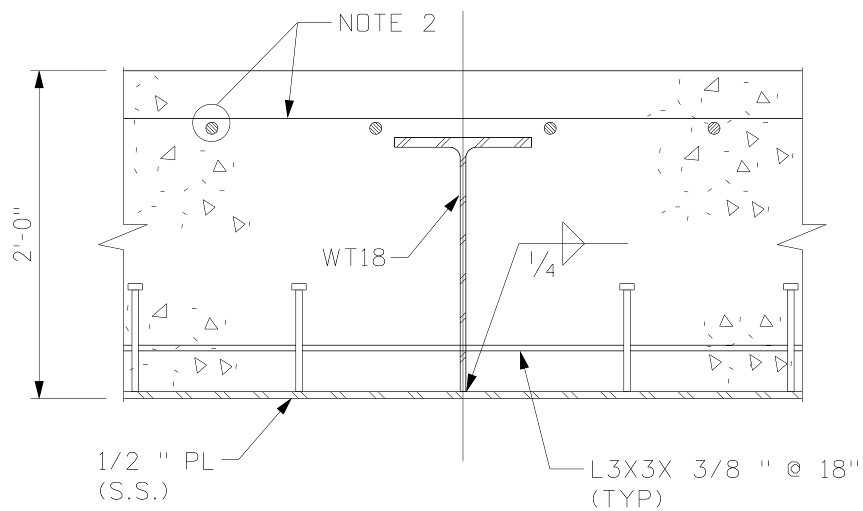
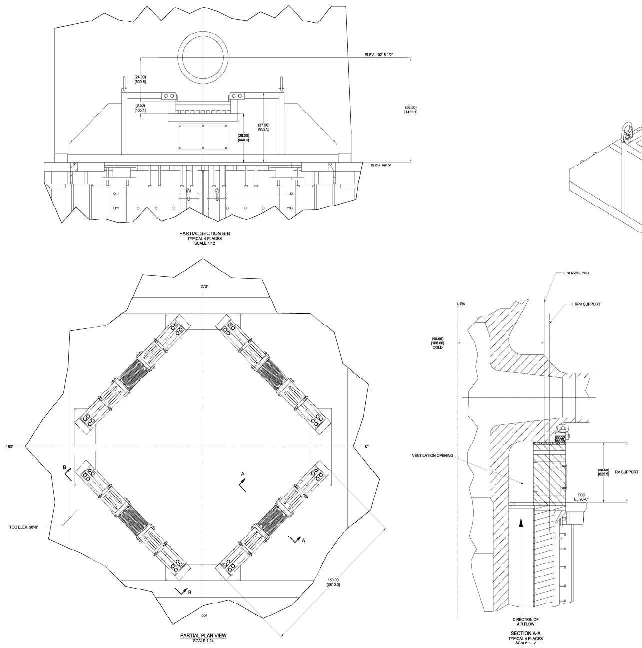
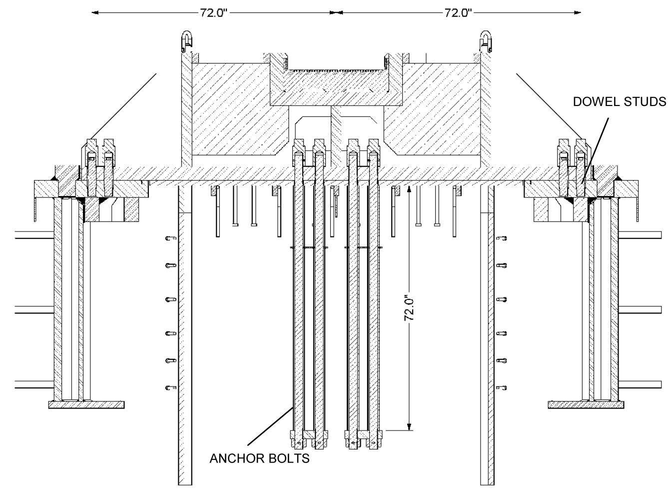


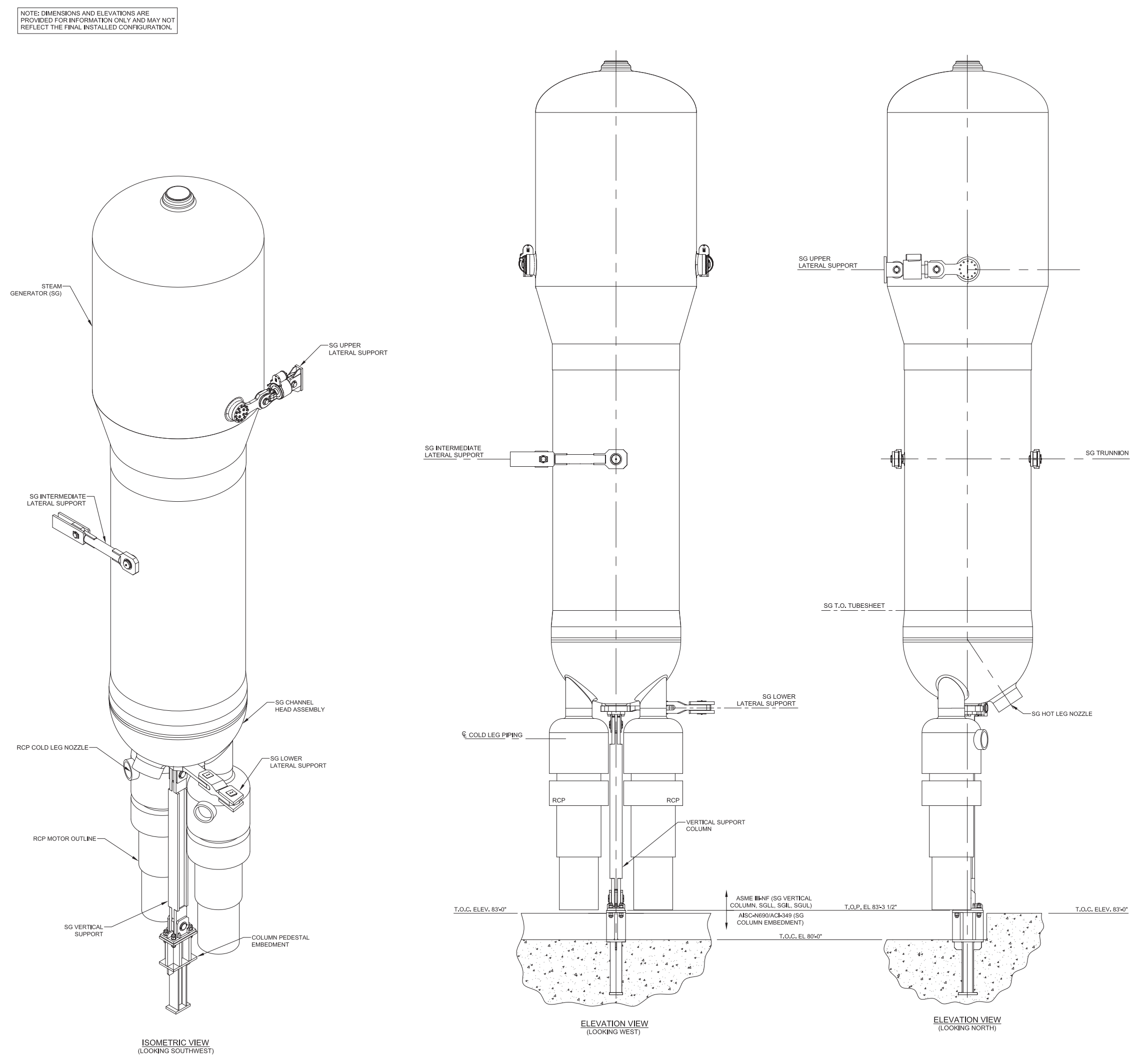
Figure 3.8.3-3
Structural Floor Module





Section through support and embedments

Figure 3.8.3-4
Reactor Vessel Supports



SG Support Assembly

Figure 3.8.3-5 (Sheet 1 of 5)
Steam Generator Supports

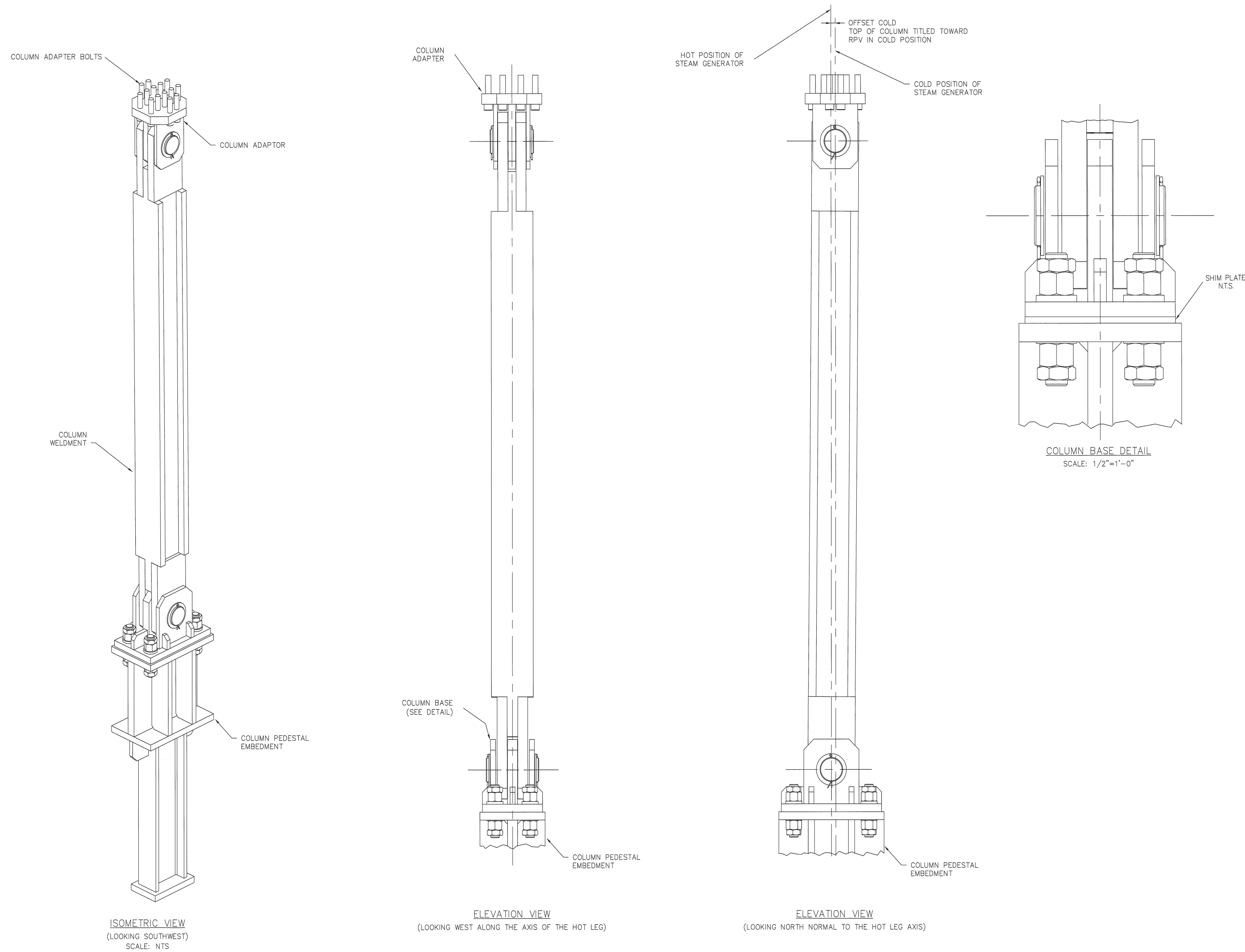
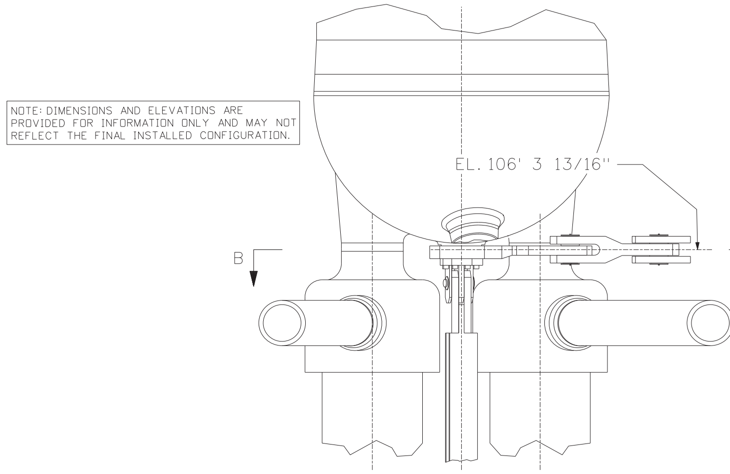
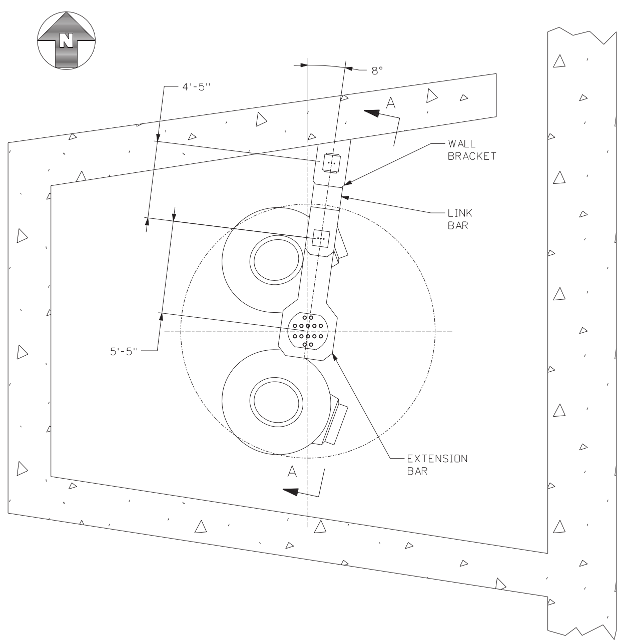


Figure 3.8.3-5 (Sheet 2 of 5)
Steam Generator Supports



ELEVATION VIEW
(ROTATED VIEW A-A)

Lower Lateral Support Elevation View



PLAN VIEW
SECTION B-B

Lower Lateral Support Plan View

Figure 3.8.3-5 (Sheet 3 of 5)
Steam Generator Supports

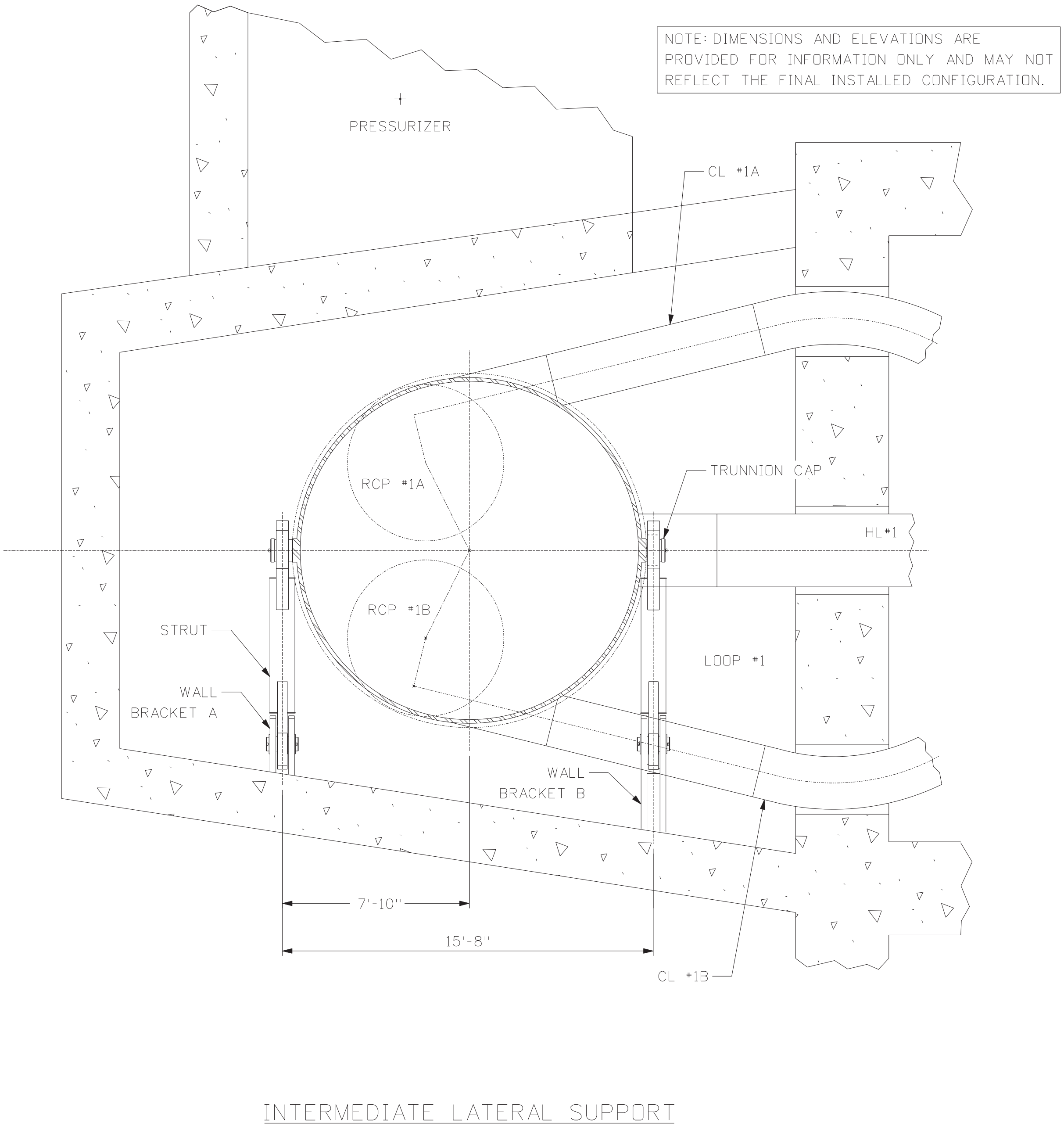


Figure 3.8.3-5 (Sheet 4 of 5)
Steam Generator Supports

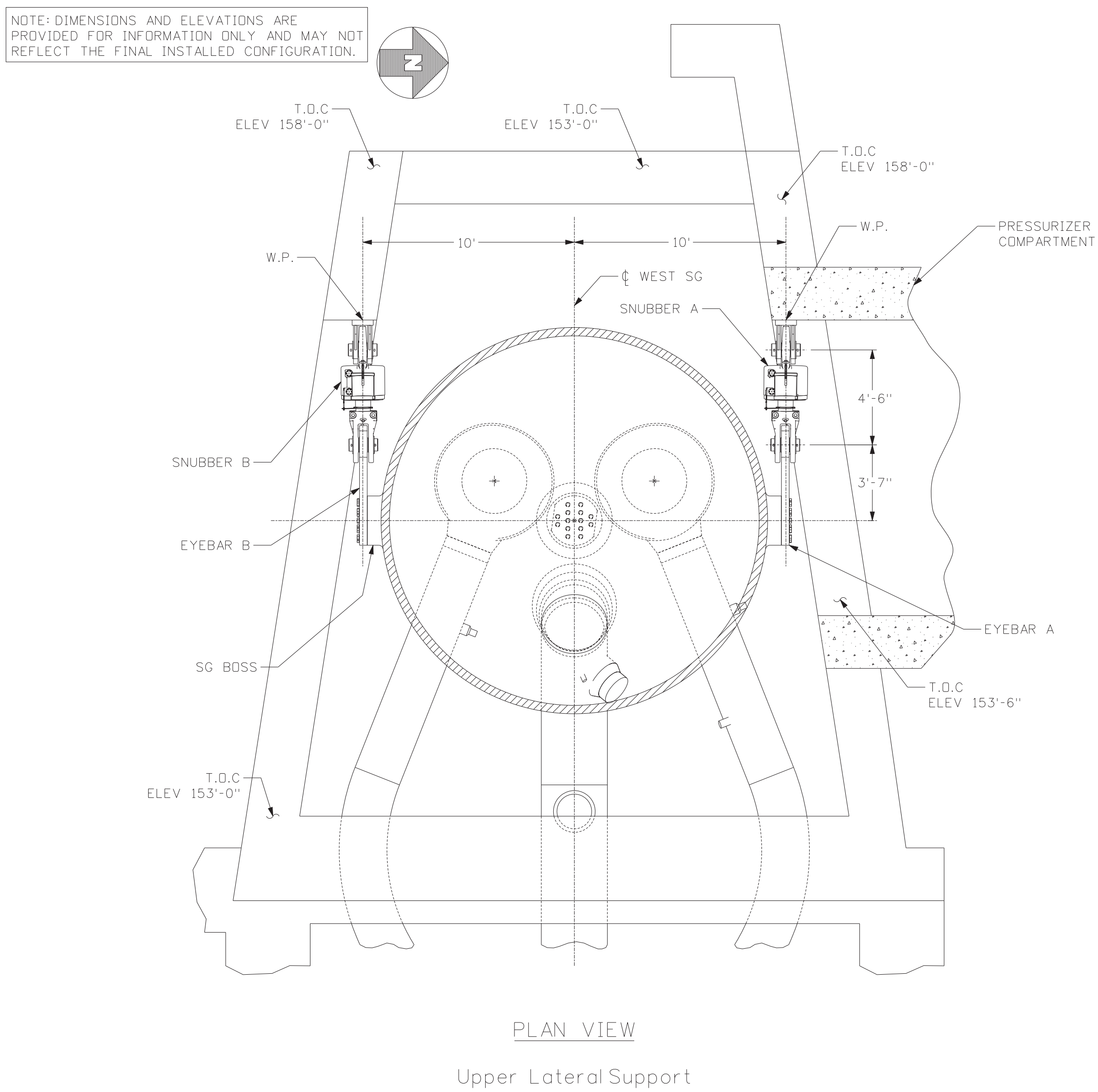


Figure 3.8.3-5 (Sheet 5 of 5)
Steam Generator Supports

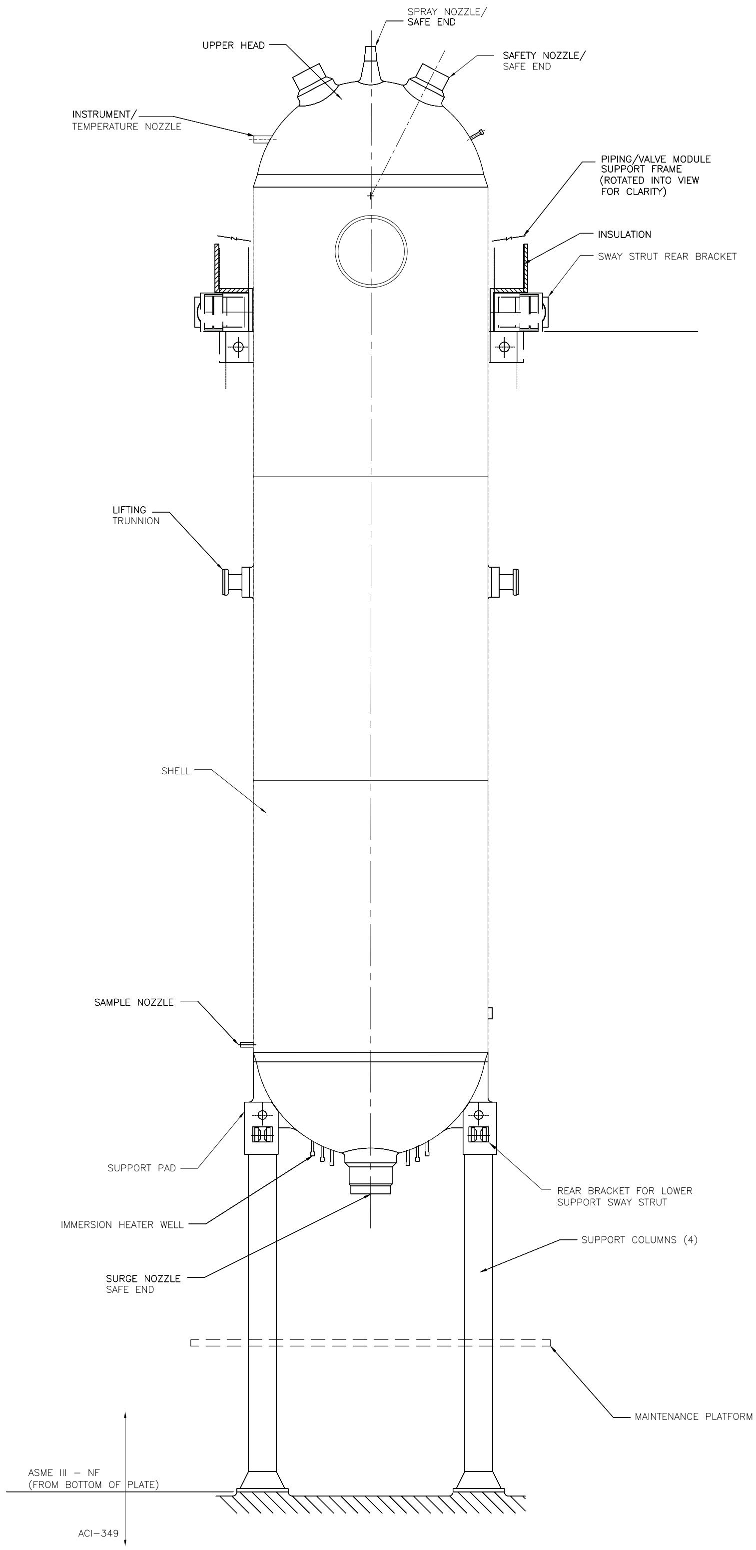


Figure 3.8.3-6 (Sheet 1 of 4)
Pressurizer Support Columns

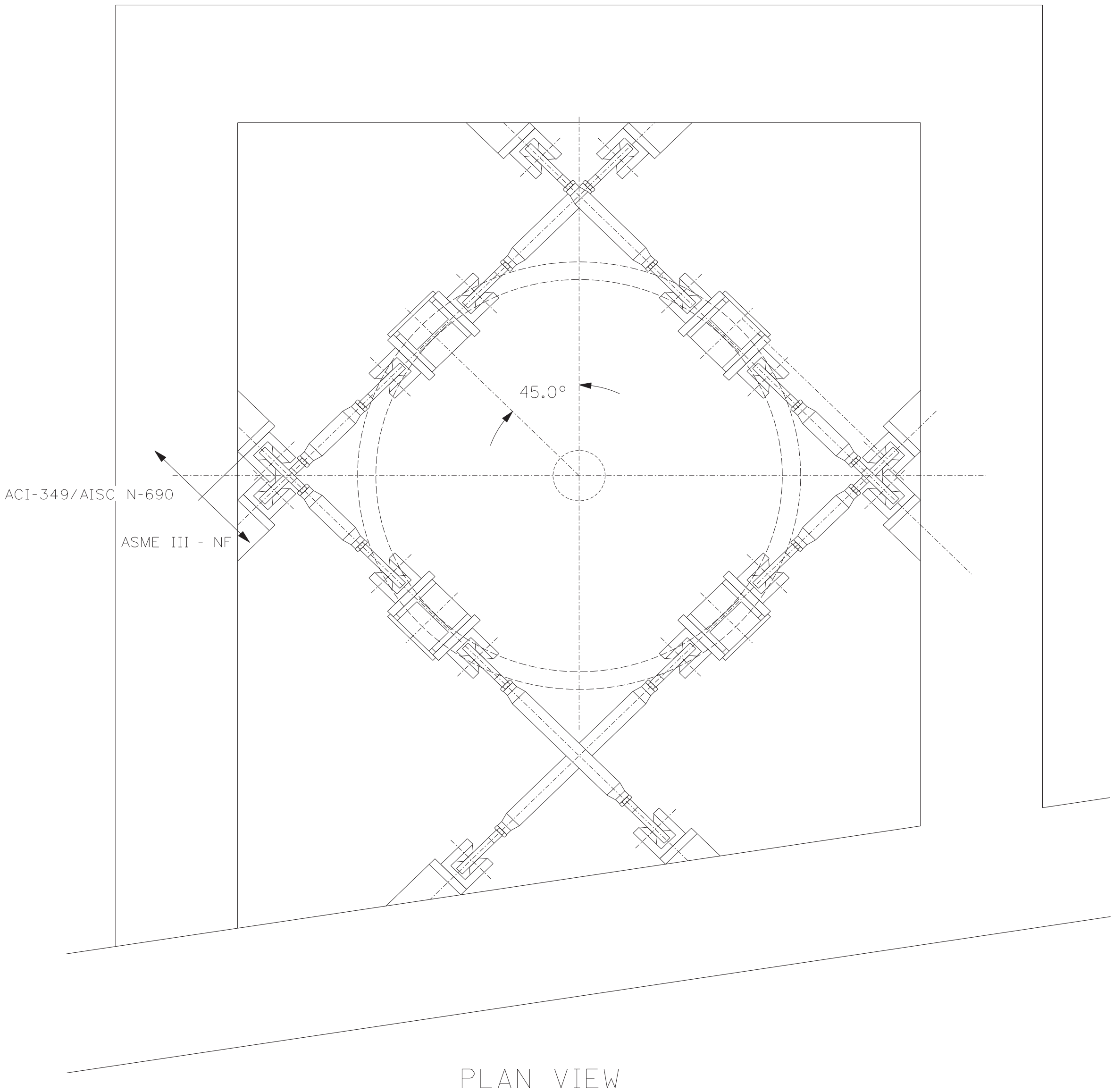
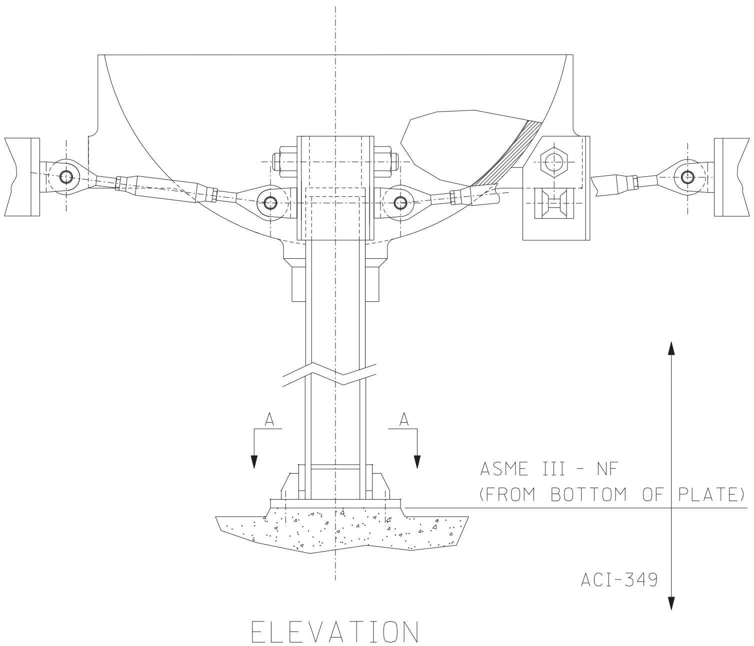


Figure 3.8.3-6 (Sheet 2 of 4)
Pressurizer Lower Lateral Supports



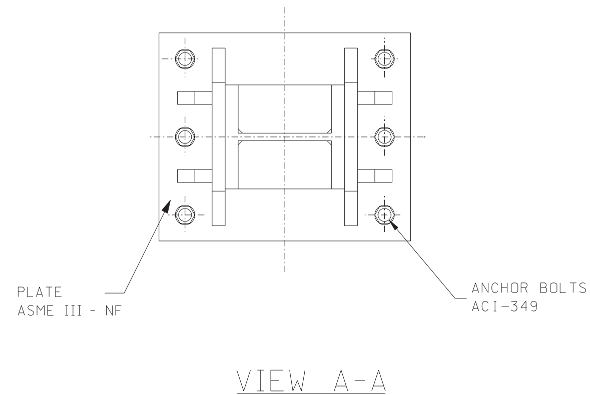
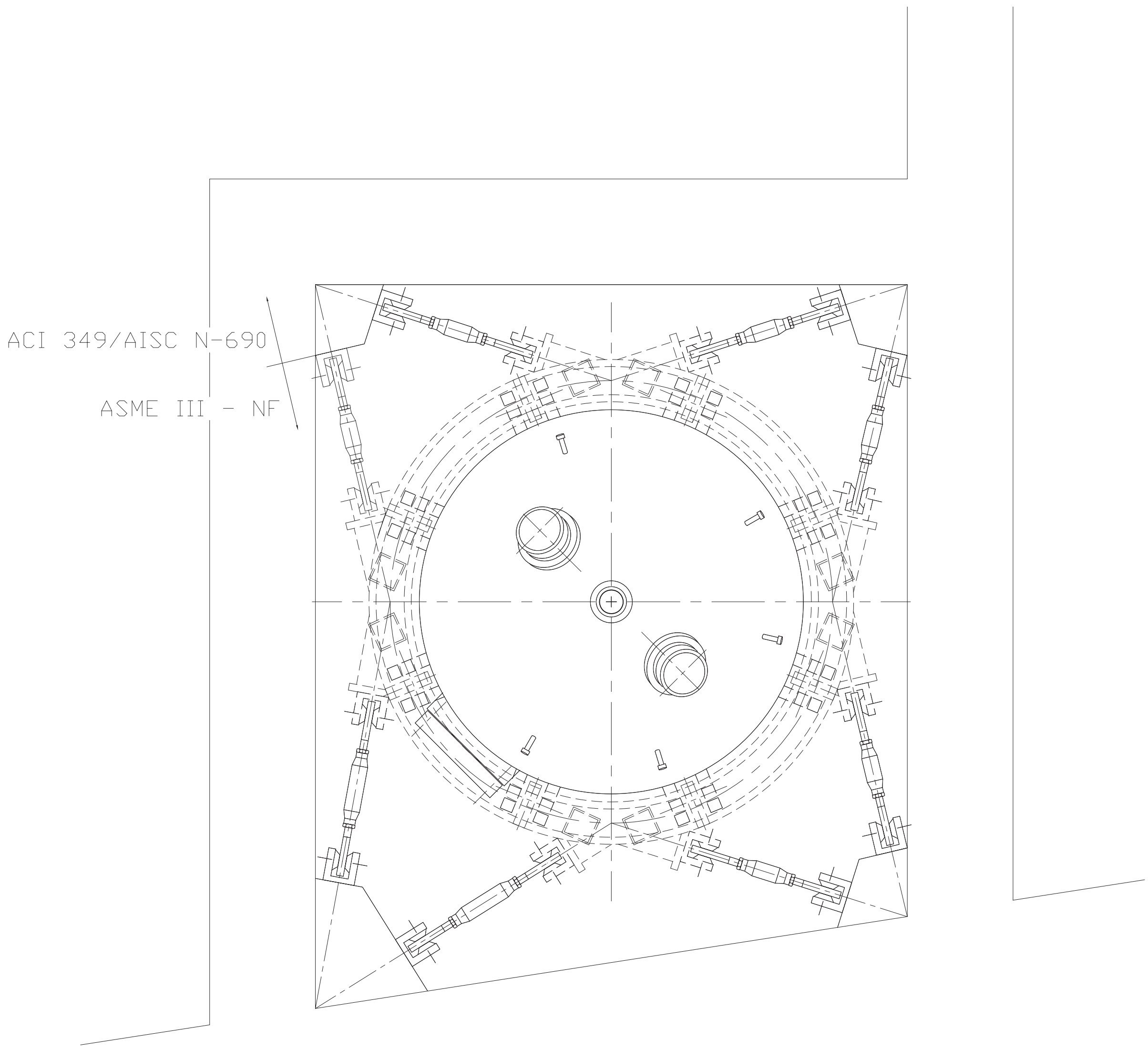


Figure 3.8.3-6 (Sheet 3 of 4)
Pressurizer Lower Supports



PLAN VIEW

Figure 3.8.3-6 (Sheet 4 of 4)
Pressurizer Upper Supports

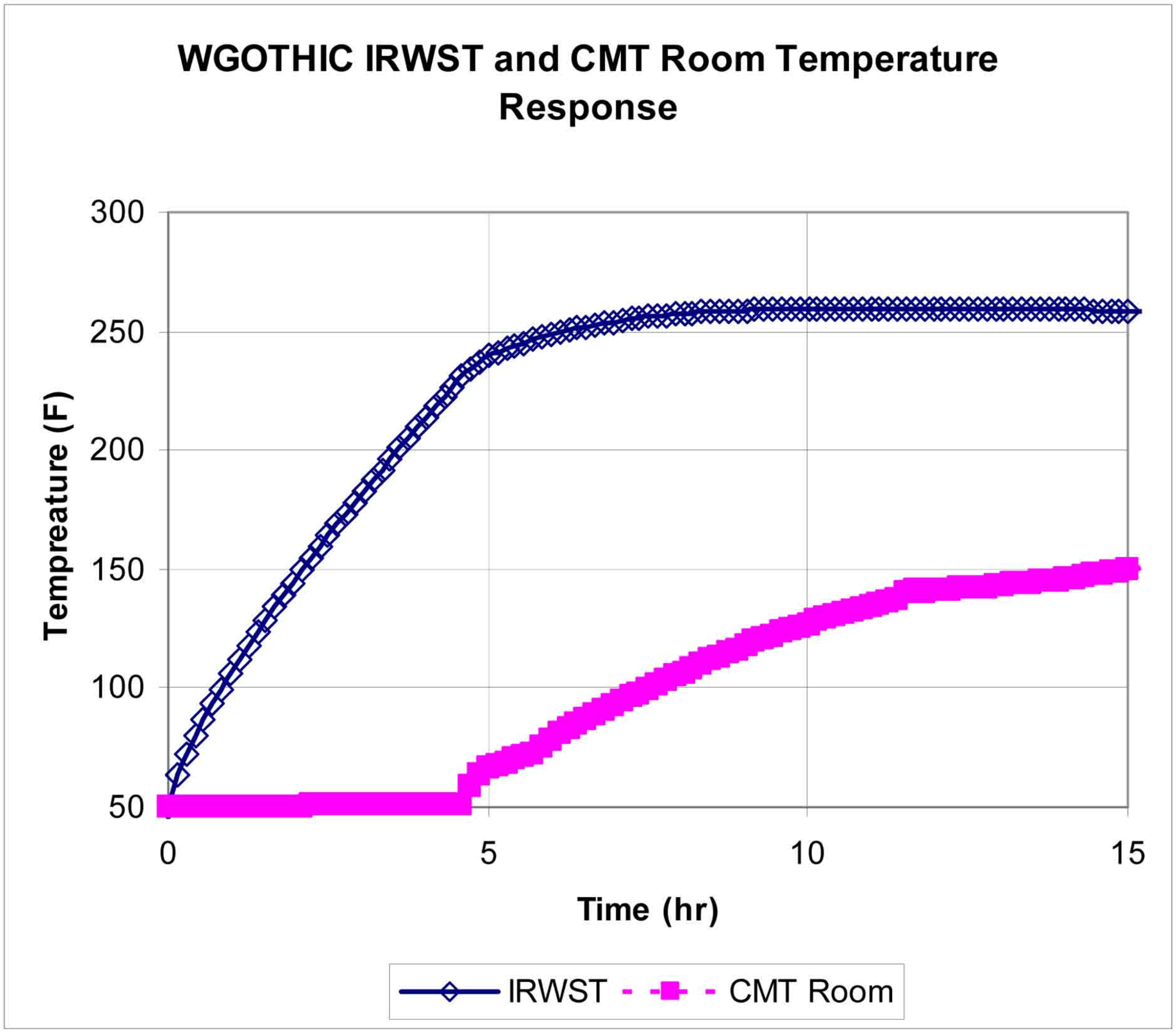
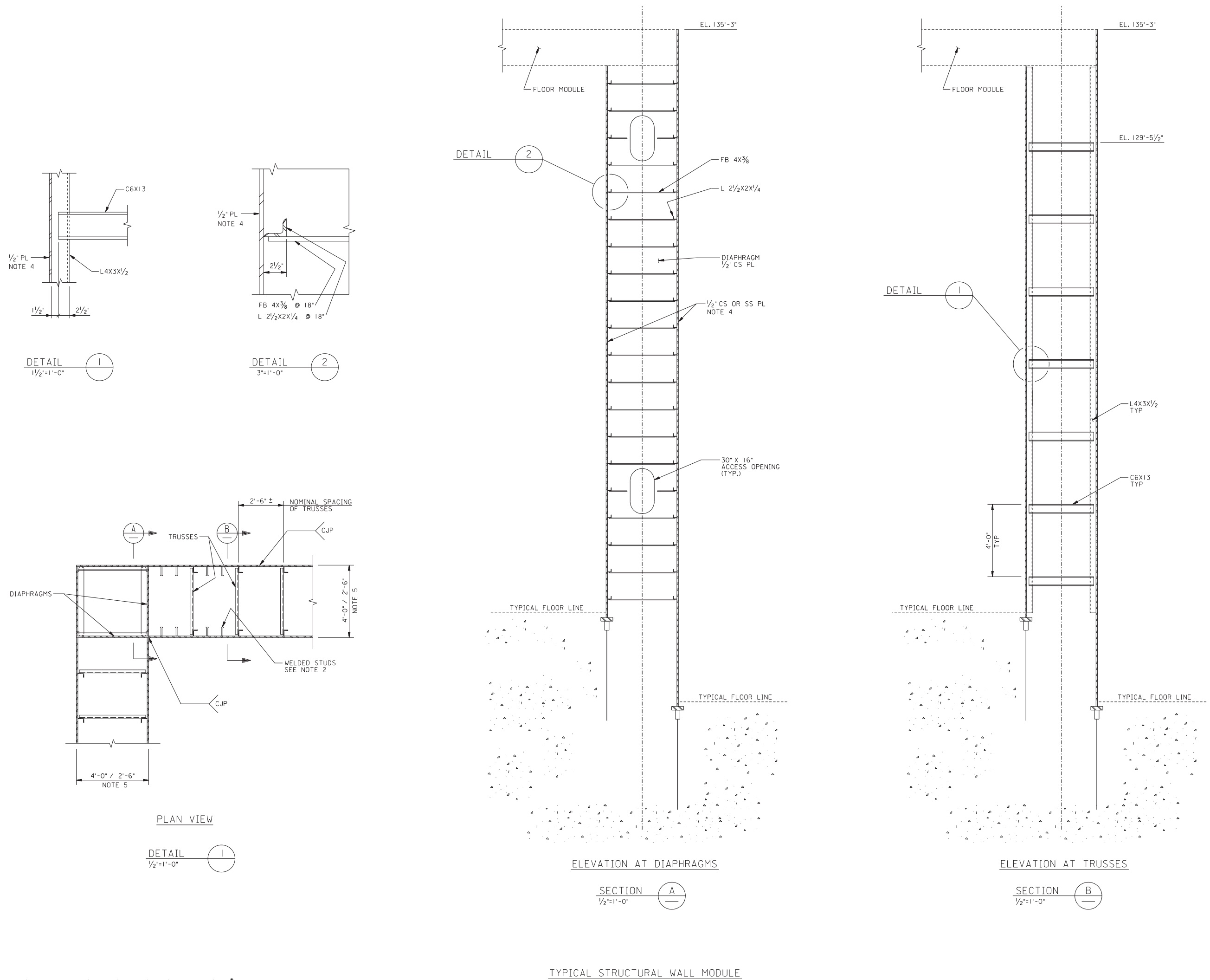


Figure 3.8.3-7
IRWST Temperature Transient



NOTES:

1. ALL WORK SHOWN ON THIS DRAWING IS SEISMIC CATEGORY 1.
2. DESIGN SPACING FOR WELDED STUDS IS AS FOLLOWS, UNLESS OTHERWISE REQUIRED BY THE SPECIFIC CODES AND STANDARDS INVOKED:
   3/4" ø X 6" @ 10" VERTICAL FOR A36 CS
   3/4" ø X 6" @ 10" HORIZONTAL FOR A36 CS
   5/8" ø X 6" @ 6" VERTICAL FOR SS
   5/8" ø X 6" @ 6" HORIZONTAL FOR SS
   THE TOLERANCE FOR THE SPACING IS ± 10% OF THE DESIGN SPACING
3. THE TYPICAL DETAILS SHOWN REPRESENT THE FUNDAMENTAL APPROACH FOR THE COMPOSITE WALL MODULES. THE FINAL DESIGN DETAILS MAY DIFFER FROM THOSE SHOWN FOR THE FOLLOWING REASONS.
   - ACCESSIBILITY FOR INSPECTION DURING FABRICATION AND CONSTRUCTION
   - LESSONS LEARNED FROM IMPLEMENTATION OF THE DESIGN
   - EASE OF FABRICATION AND CONSTRUCTION
   - RESOLUTION OF CONSTRUCTABILITY ISSUES AND SEQUENCES
   - VARIATION IN MODULE SIZE AND CONFIGURATION

   CHANGES MADE DURING DETAILED DESIGN ARE IN ACCORDANCE WITH THE SPECIFIC CODES AND STANDARDS INVOKED.
4. AS DESCRIBED IN SECTION 3.8.3.1.3. SOME CA01 AND CA05 MODULE WALL FACEPLATE THICKNESSES ARE GREATER THAN THE 0.5 INCH NOMINAL FACEPLATE THICKNESS.
5. SOME MODULE WALLS THAT SEPARATE EQUIPMENT SPACES FROM PERSONNEL ACCESS AREAS HAVE A WALL THICKNESS WHICH IS DIFFERENT THAN THE DIMENSIONS SHOWN. FIGURE 3.8.3-1 SHEETS 2 AND 4 IDENTIFY THE CA01 AND CA05 WALL THICKNESS DIFFERENCES.

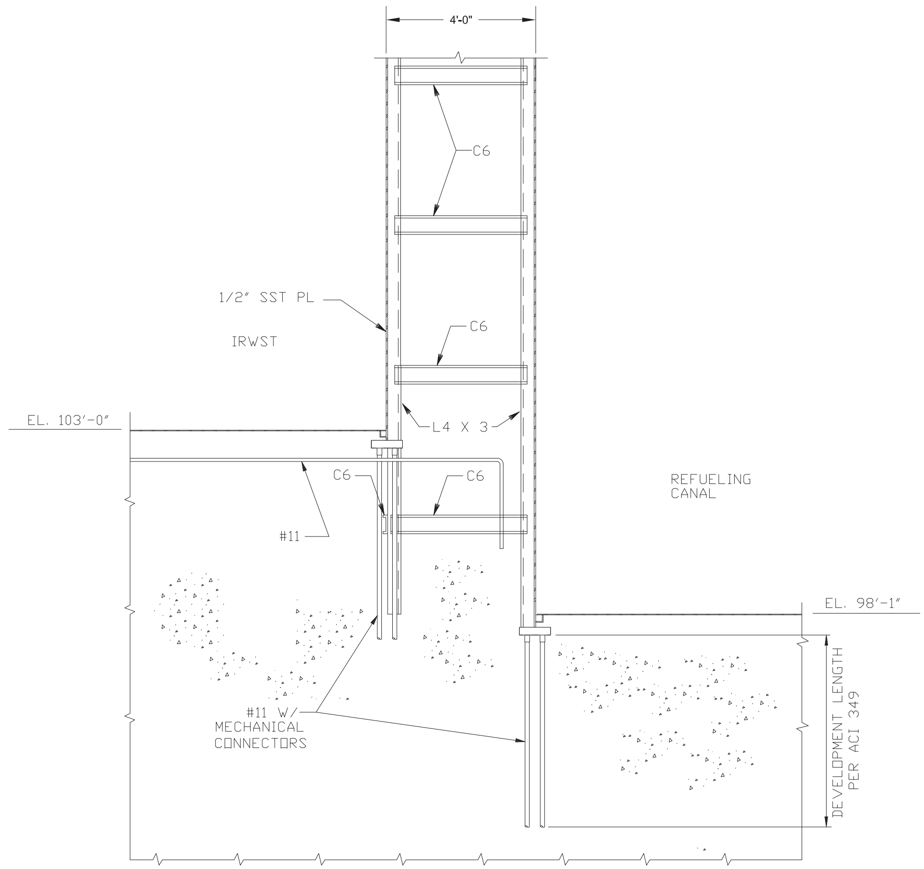
See subsection 3.8.3.1.3 for information that is designated as Tier 2*.

Figure 3.8.3-8 (Sheet 1 of 3)
[*Structural Modules – Typical Design Details*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

NOTES:

THE TYPICAL DETAILS SHOWN REPRESENT THE FUNDAMENTAL APPROACH FOR THE COMPOSITE WALL MODULES. THE FINAL DESIGN DETAILS MAY DIFFER FROM THOSE SHOWN FOR THE FOLLOWING REASONS.
- ACCESSIBILITY FOR INSPECTION DURING FABRICATION AND CONSTRUCTION
- LESSONS LEARNED FROM IMPLEMENTATION OF THE DESIGN
- EASE OF FABRICATION AND CONSTRUCTION
- RESOLUTION OF CONSTRUCTABILTY ISSUES AND SEQUENCES
- VARIATION IN MODULE SIZE AND CONFIGURATION CHANGES MADE DURING DETAILED DESIGN ARE IN ACCORDANCE WITH THE SPECIFIC CODES AND STANDARDS INVOKED.
- AS DESCRIBED IN SECTION 3.8.3.1.3, SOME CA01 AND CA05 MODULE WALL FACEPLATE THICKNESSES ARE GREATER THAN THE 0.5 INCH NOMINAL FACEPLATE THICKNESS.
- SOME MODULE WALLS THAT SEPARATE EQUIPMENT SPACES FROM PERSONNEL ACCESS AREAS HAVE A WALL THICKNESS WHICH IS DIFFERENT THAN THE DIMENSIONS SHOWN. FIGURE 3.8.3-1 SHEETS 2 AND 4 IDENTIFY THE CA01 AND CA05 WALL THICKNESS DIFFERENCES.

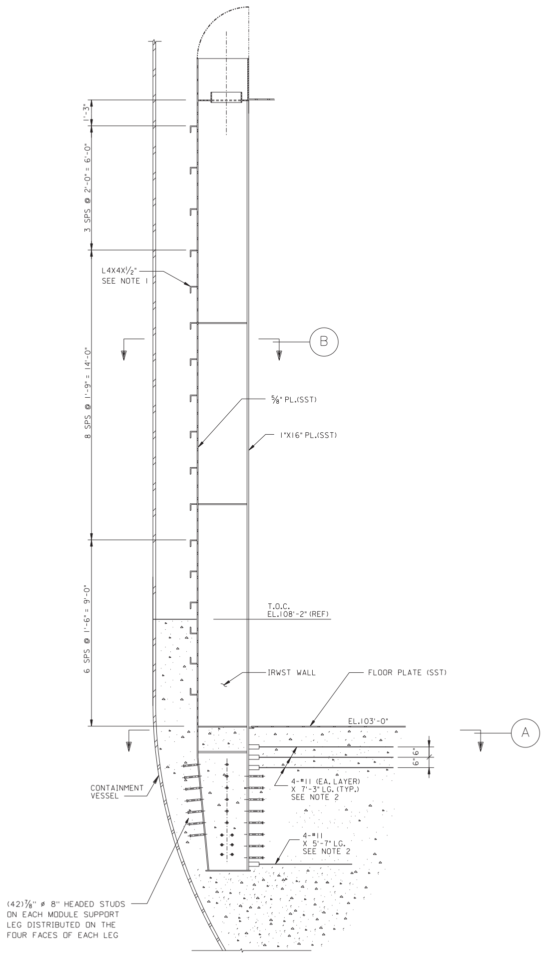


CHANGES TO THE MECHANICAL CONNECTOR DETAIL SHOWN WILL MAINTAIN A DIRECT LOAD PATH TO TRANSFER LOADS FROM THE MODULE FACE PLATES TO THE REINFORCEMENT BARS IN THE BASE CONCRETE THROUGH THE USE OF INTERVENING PLATES, MECHANICAL CONNECTORS, AND WELDS.

See subsection 3.8.3.1.3 for information that is designated as Tier 2*.

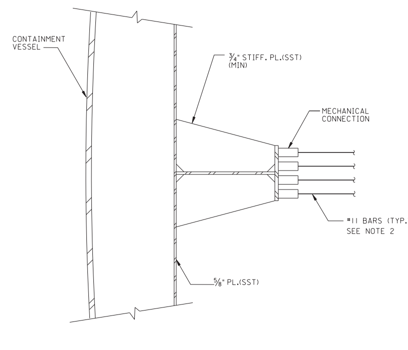
Figure 3.8.3-8 (Sheet 2 of 3)
[*Structural Modules – Typical Design Details*]*

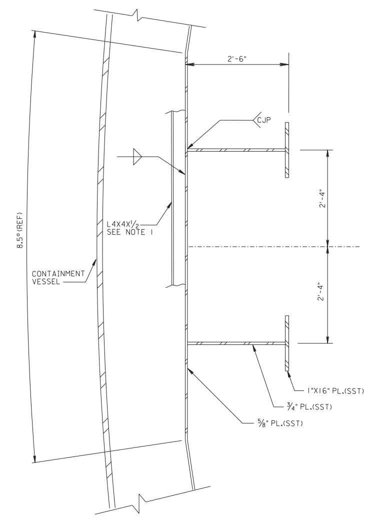
*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



TYPICAL SECTION

SEE SUBSECTION 3.8.3.1.3 FOR INFORMATION THAT IS DESIGNATED AS TIER 2*.





GENERAL NOTES:
THE TYPICAL DETAILS SHOWN REPRESENT THE FUNDAMENTAL APPROACH FOR THE COMPOSITE WALL MODULES. THE FINAL DESIGN DETAILS MAY DIFFER FROM THOSE SHOWN FOR THE FOLLOWING REASONS.

- ACCESSIBILITY FOR INSPECTION DURING FABRICATION AND CONSTRUCTION
- LESSONS LEARNED FROM IMPLEMENTATION OF THE DESIGN
- EASE OF FABRICATION AND CONSTRUCTION
- RESOLUTION OF CONSTRUCTABILITY ISSUES AND SEQUENCES
- VARIATION IN MODULE SIZE AND CONFIGURATION

CHANGES MADE DURING DETAILED DESIGN ARE IN ACCORDANCE WITH THE SPECIFIC CODES AND STANDARDS INVOKED.

NOTES:
1. SIZE SHOWN FOR ANGLES USED AS STIFFENERS IS DESIGN MINIMUM.
2. ANCHOR BARS FOR SUBMODULES ADJACENT TO OTHER IRWST WALLS ARE FORMED INTO STANDARD HOOKS AND 1 OR 2 ROWS MAY HAVE 3 BARS TO MINIMIZE INTEFERENCES.
OVERALL LENGTH FOR STANDARD HOOKED BARS VARIES FROM THAT SHOWN.

Figure 3.8.3-8 (Sheet 3 of 3)
[*Structural Modules – Typical Design Details*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

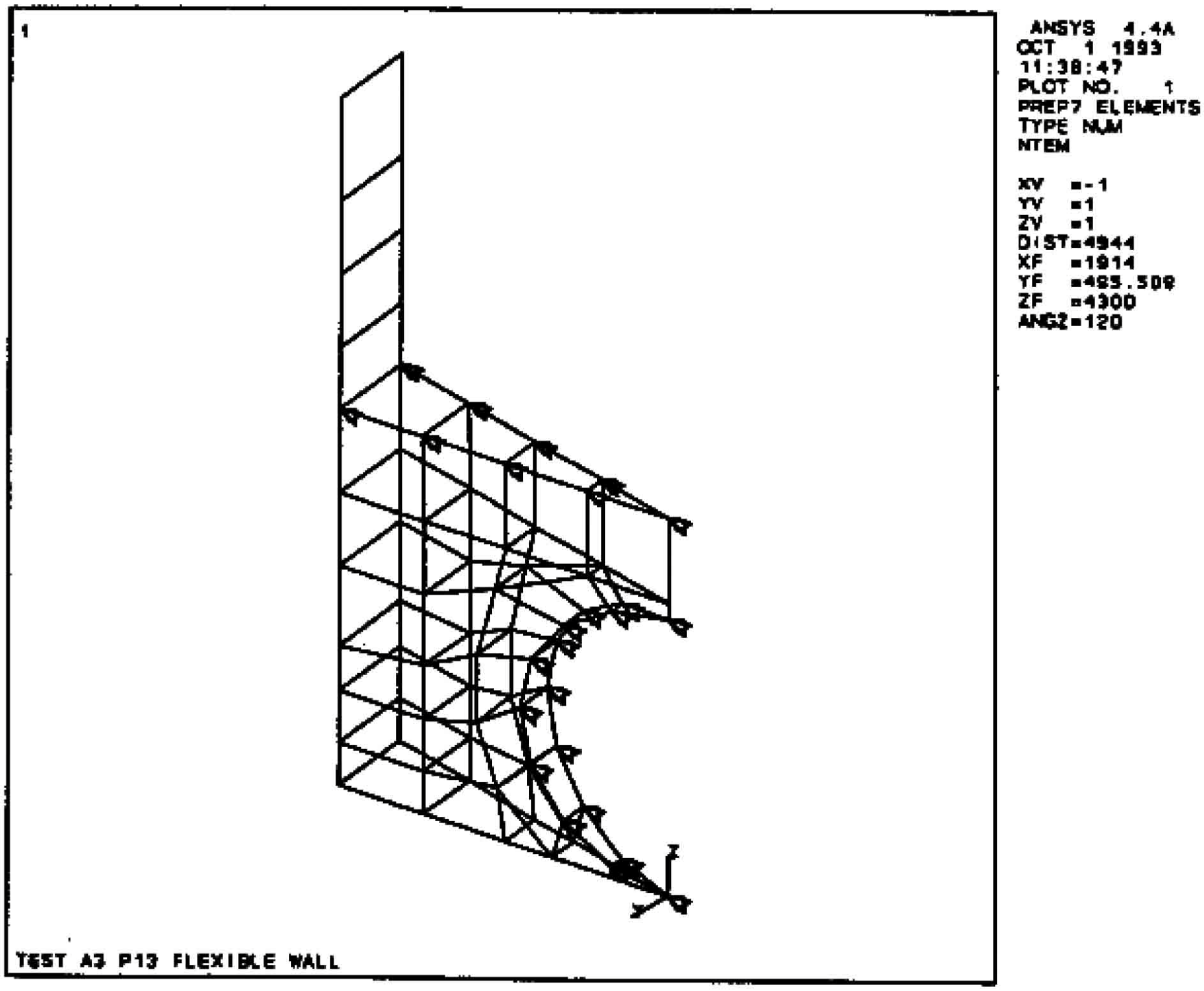


Figure 3.8.3-9
Test Tank Finite Element Model

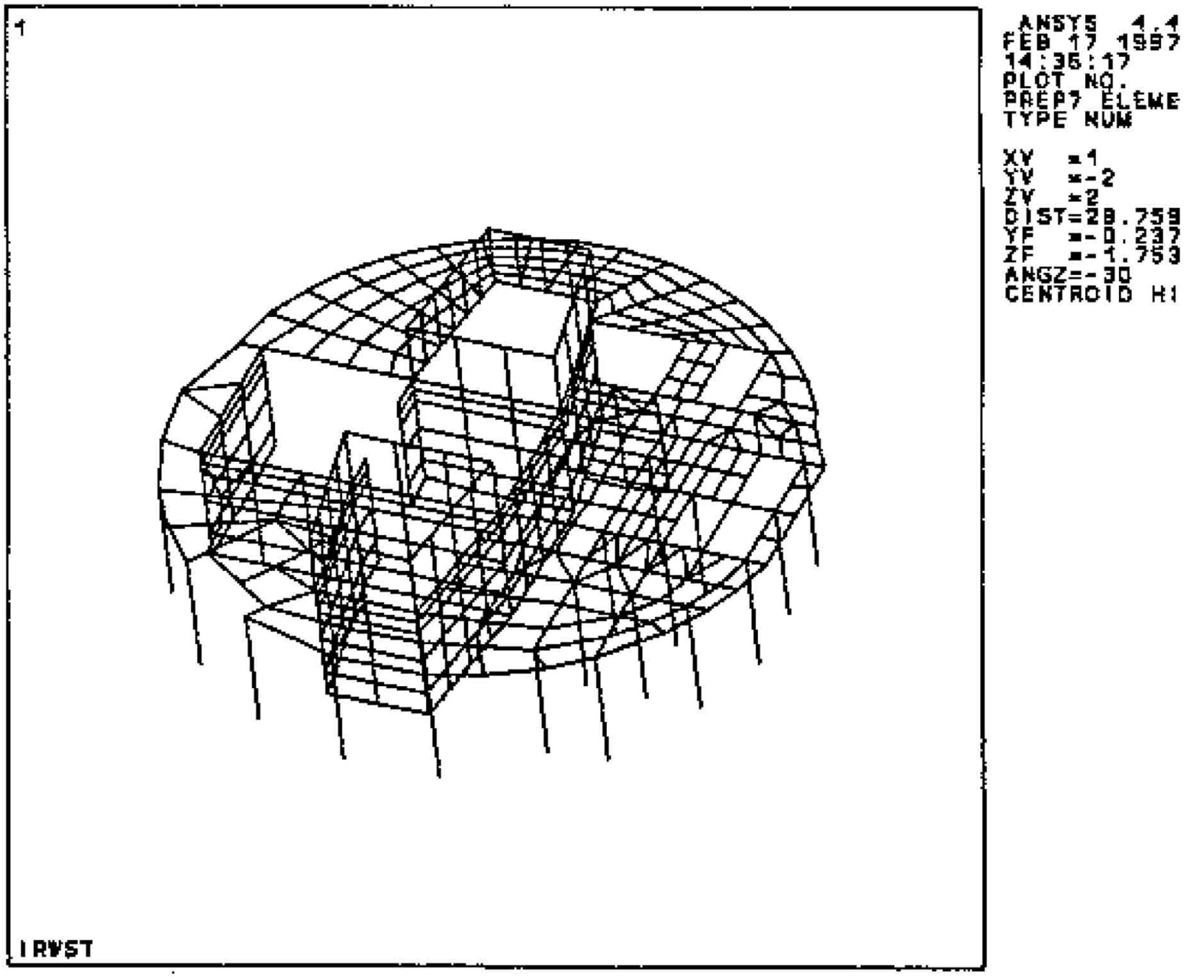


Figure 3.8.3-10 (Sheet 1 of 2)
IRWST Fluid Structure Finite Element Model
CIS Structural Model

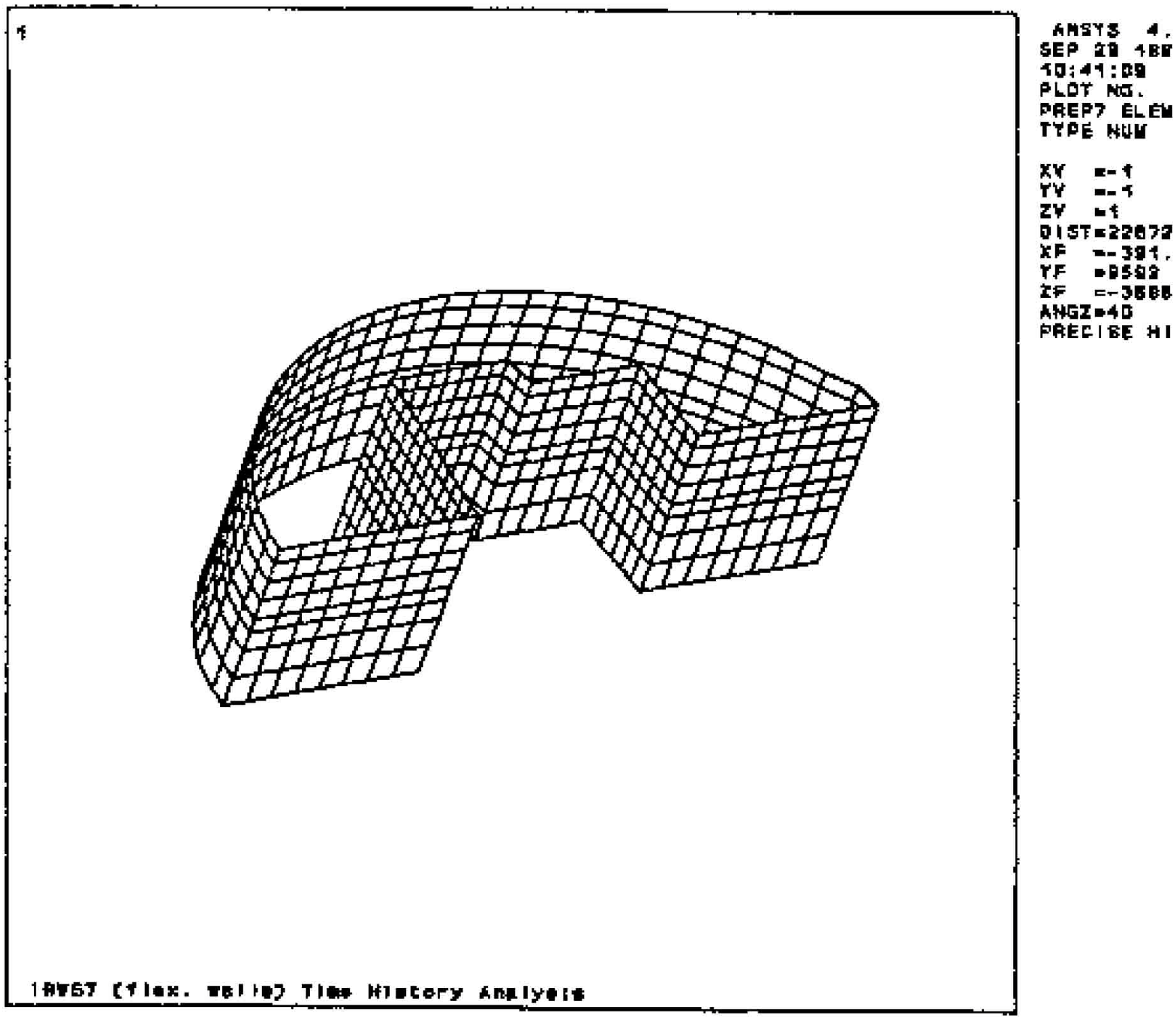


Figure 3.8.3-10 (Sheet 2 of 2)
IRWST Fluid Structure Finite Element Model
IRWST Structural Model

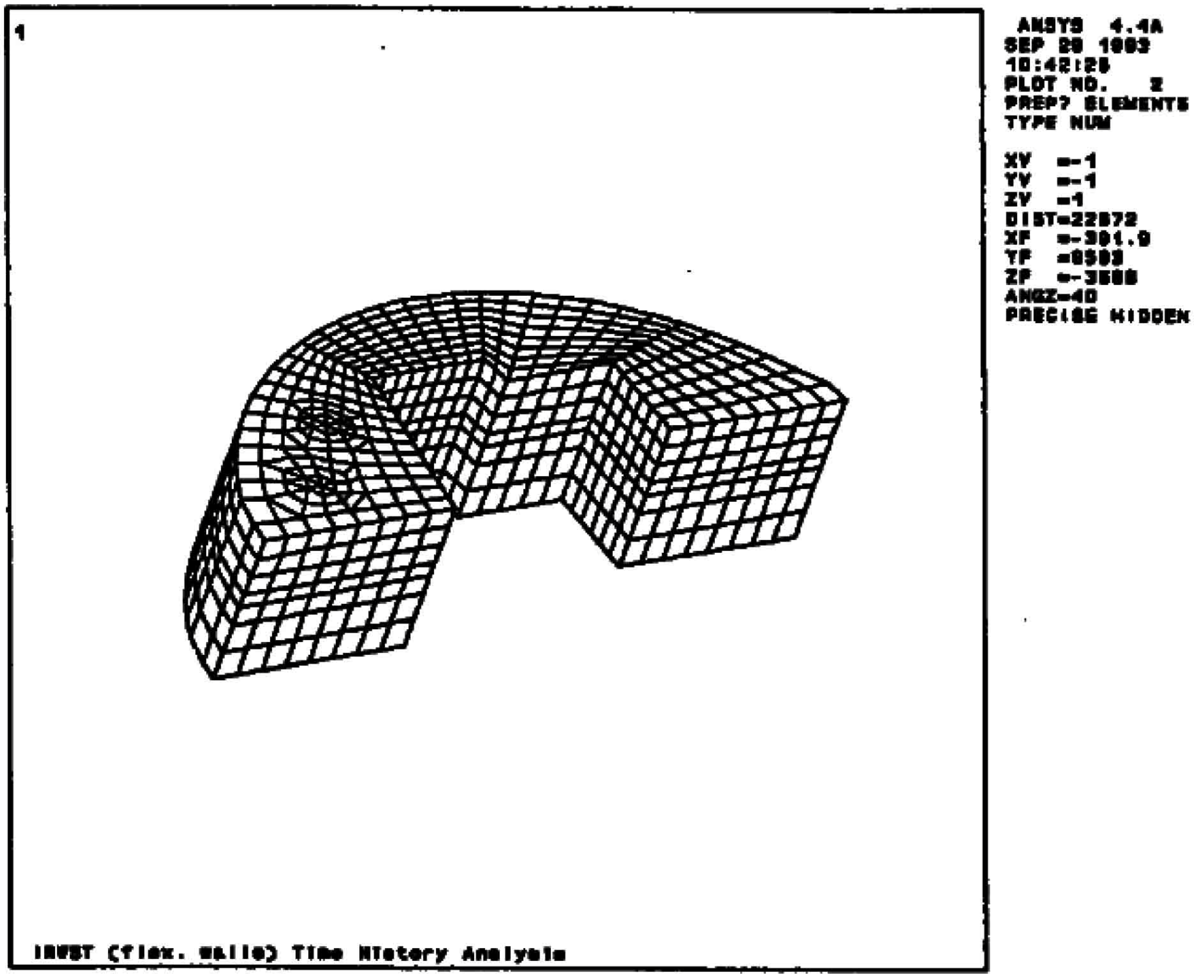


Figure 3.8.3-11
IRWST Fluid Structure Finite Element Model
Fluid Model

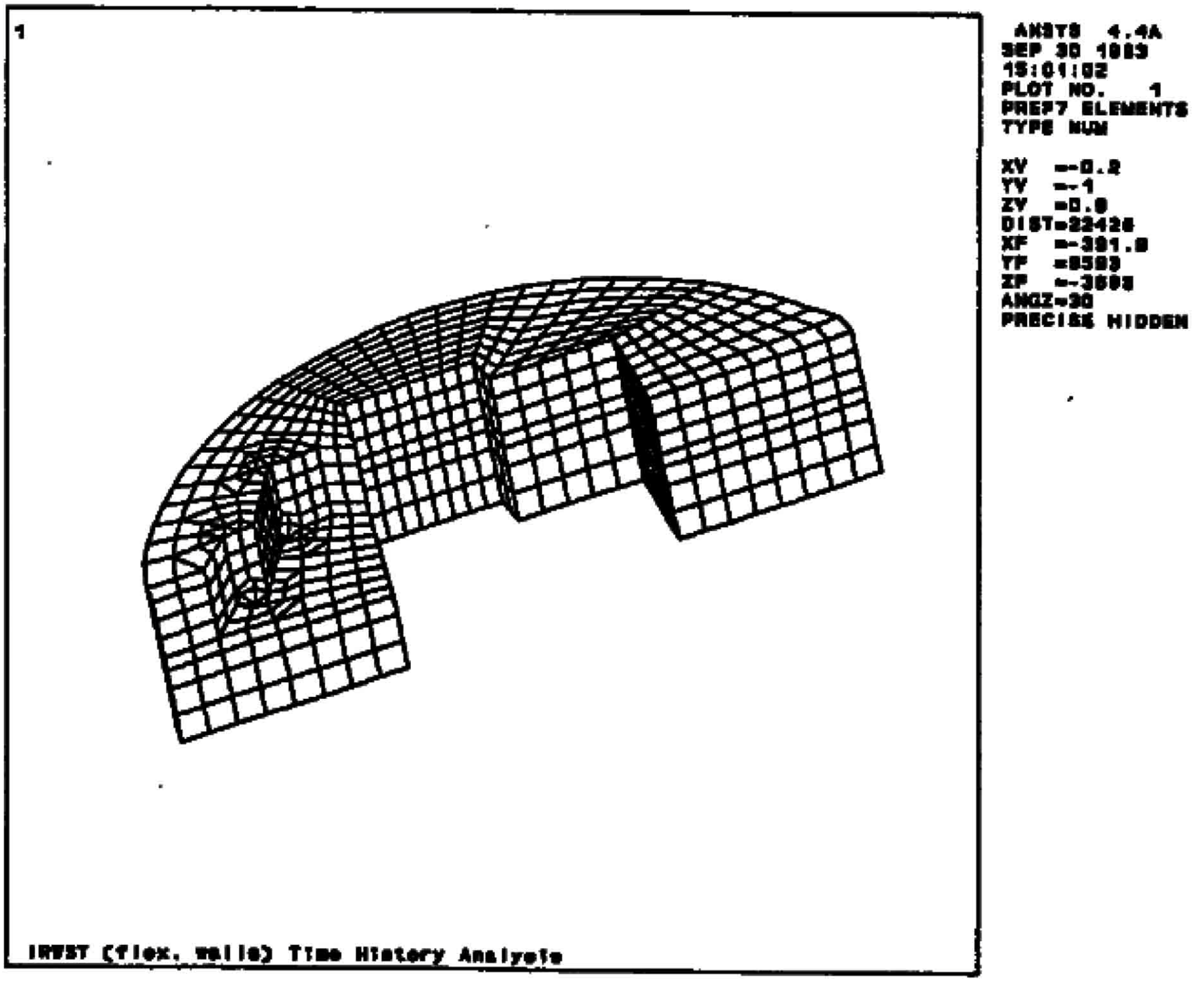


Figure 3.8.3-12
IRWST Fluid Structure Finite Element Model
Sparger Region Detail

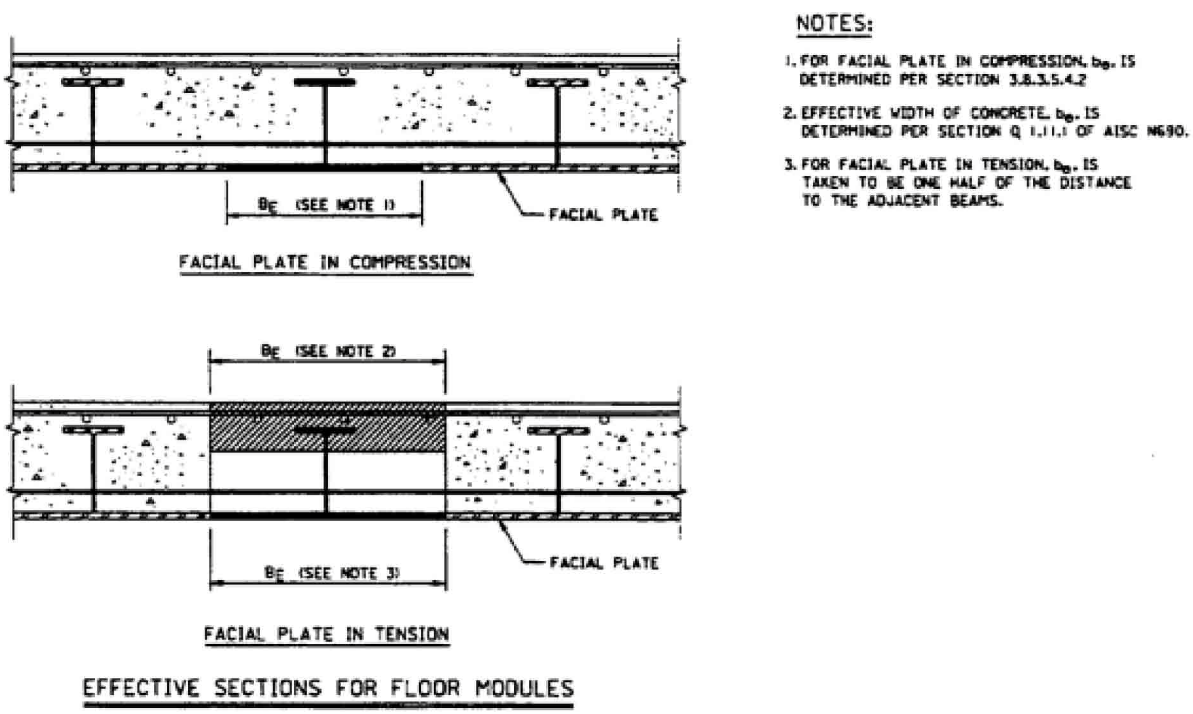


Figure 3.8.3-13
Effective Sections for Floor Modules

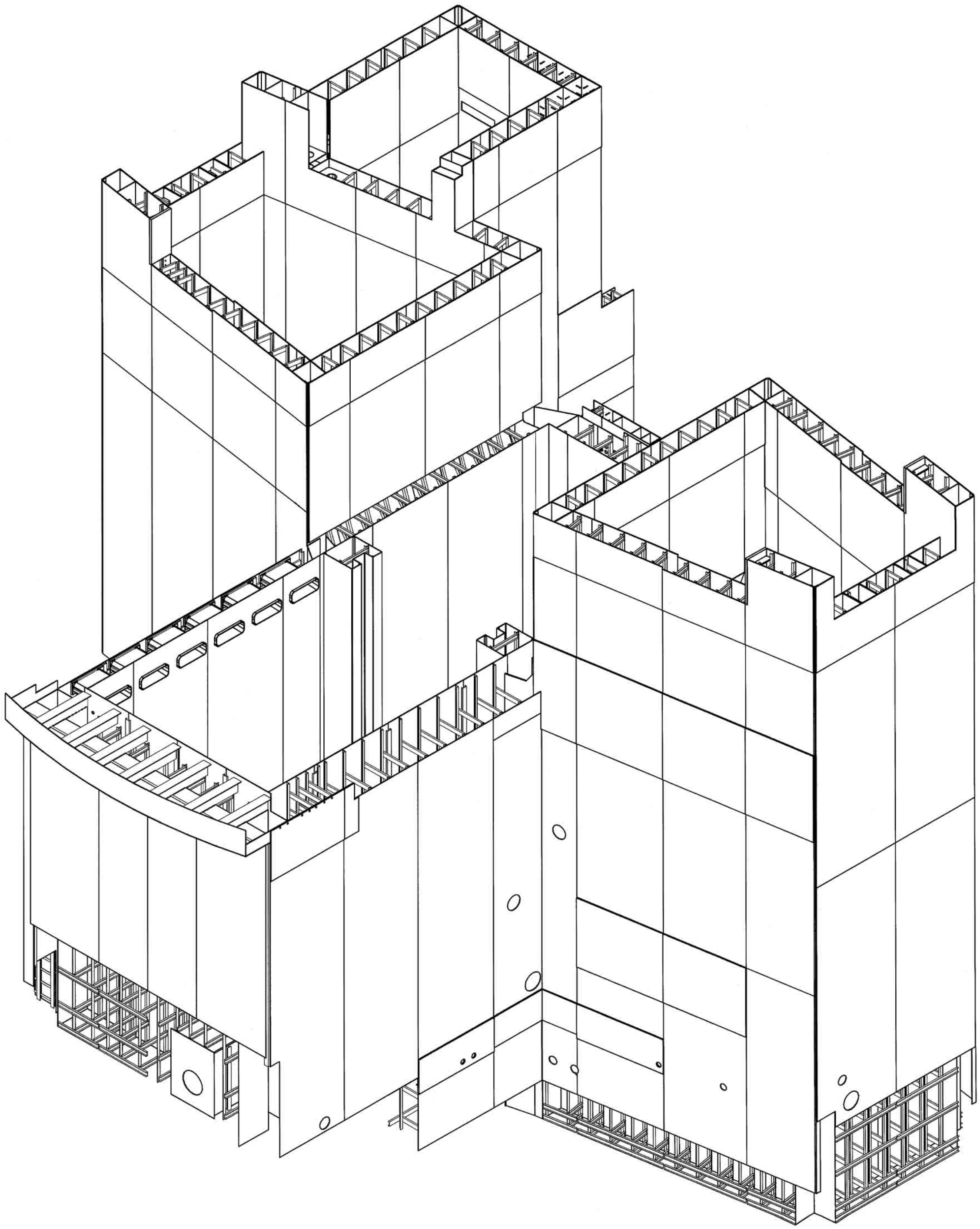


Figure 3.8.3-14 (Sheet 1 of 5)
[*CA-01 Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

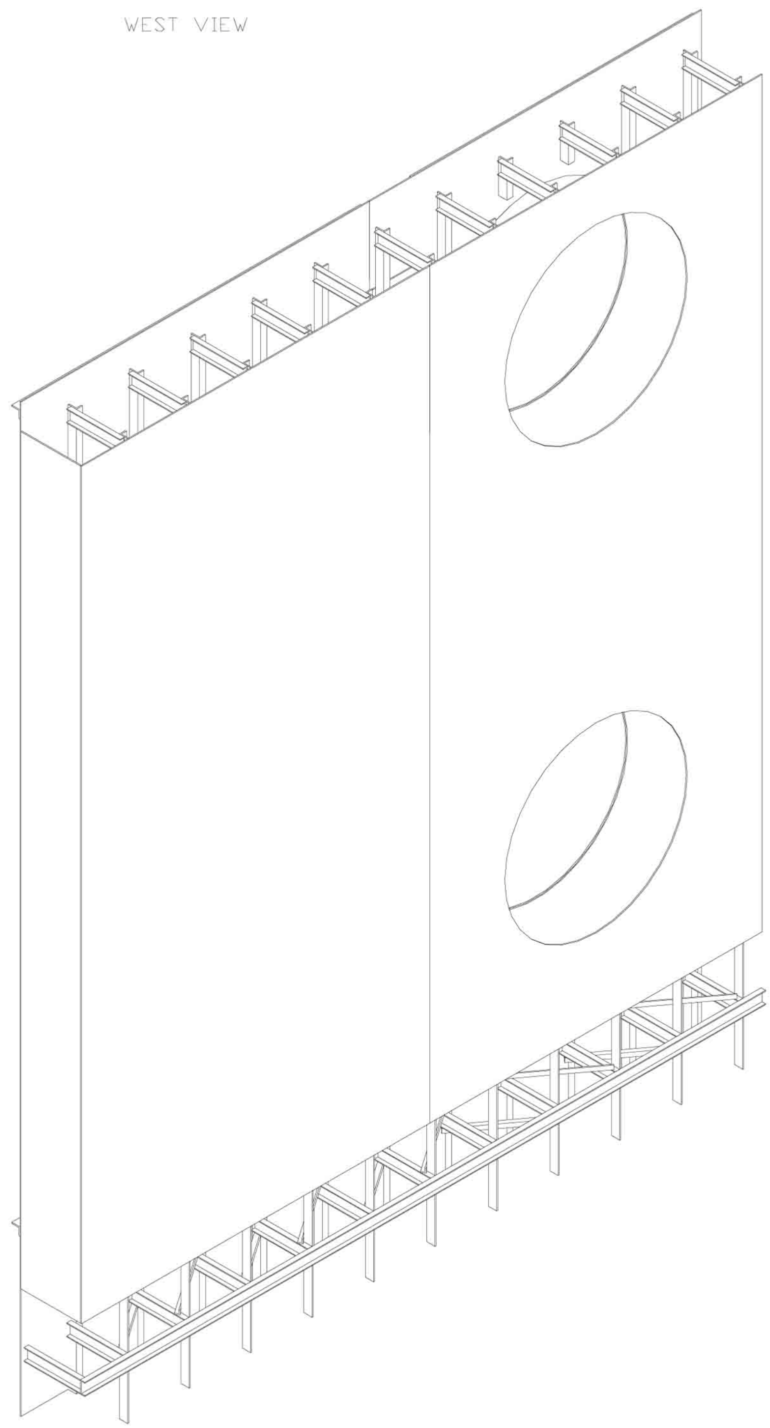
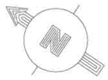


Figure 3.8.3-14 (Sheet 2 of 5)
[*CA-02 Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



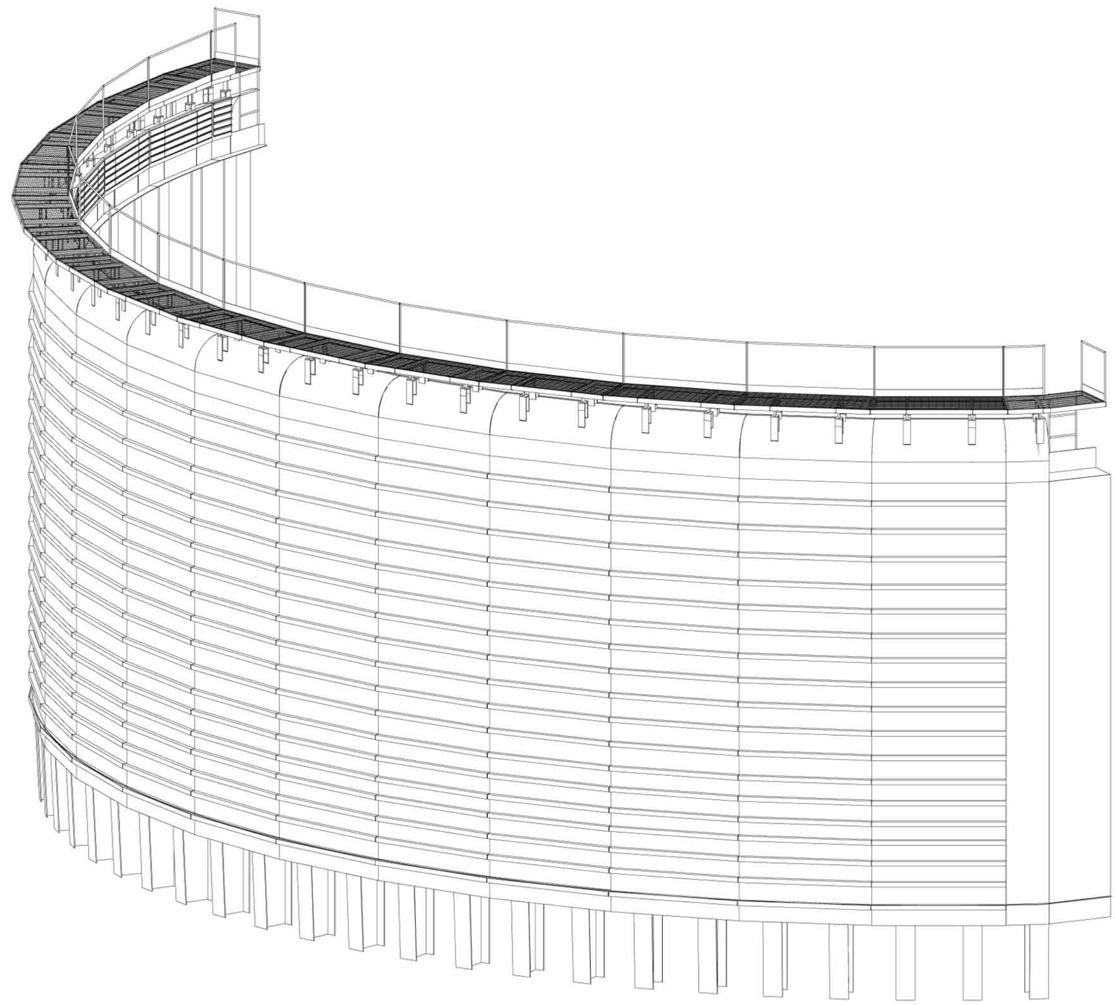
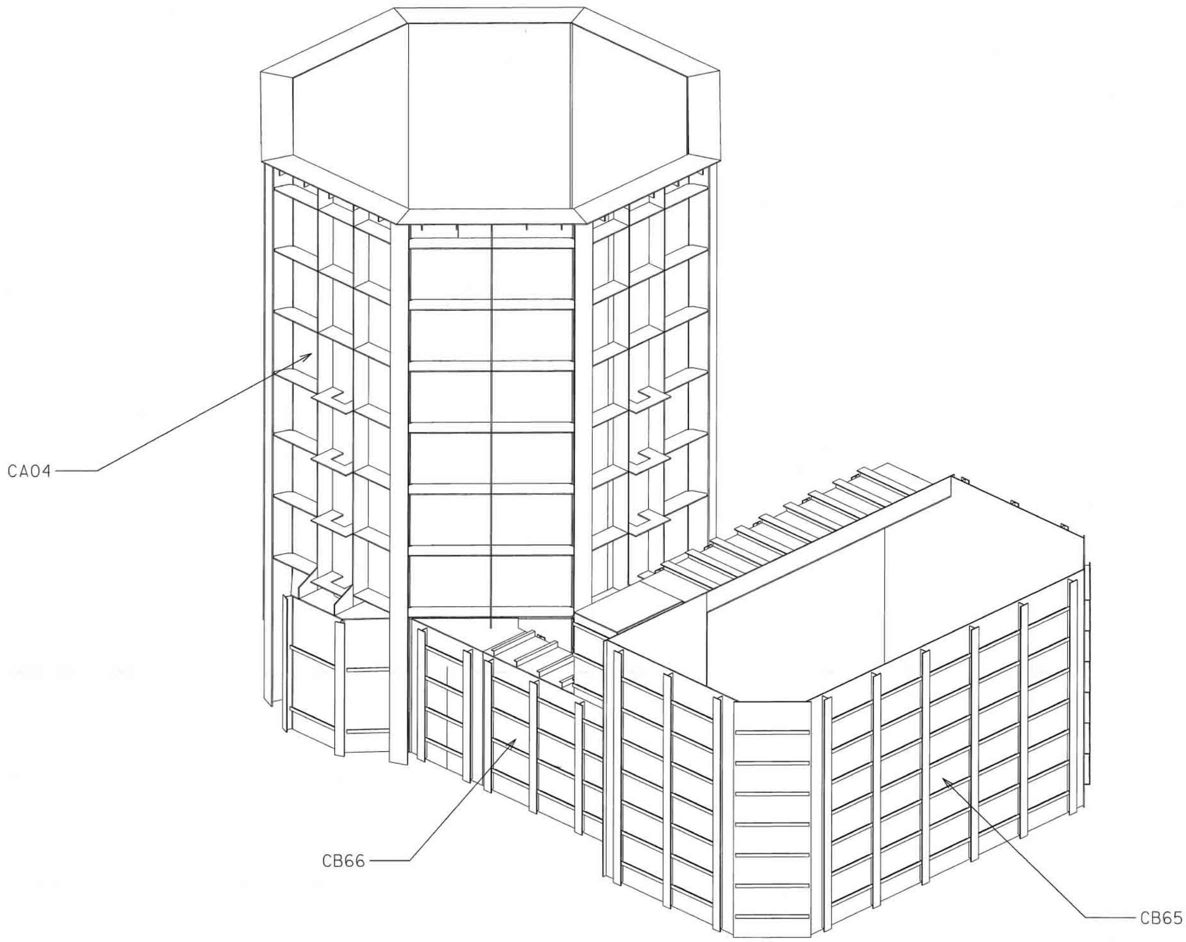


Figure 3.8.3-14 (Sheet 3 of 5)
[*CA-03 Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



ISO VIEW LOOKING SOUTH WEST

Figure 3.8.3-14 (Sheet 4 of 5)
[*CA-04 Structural Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

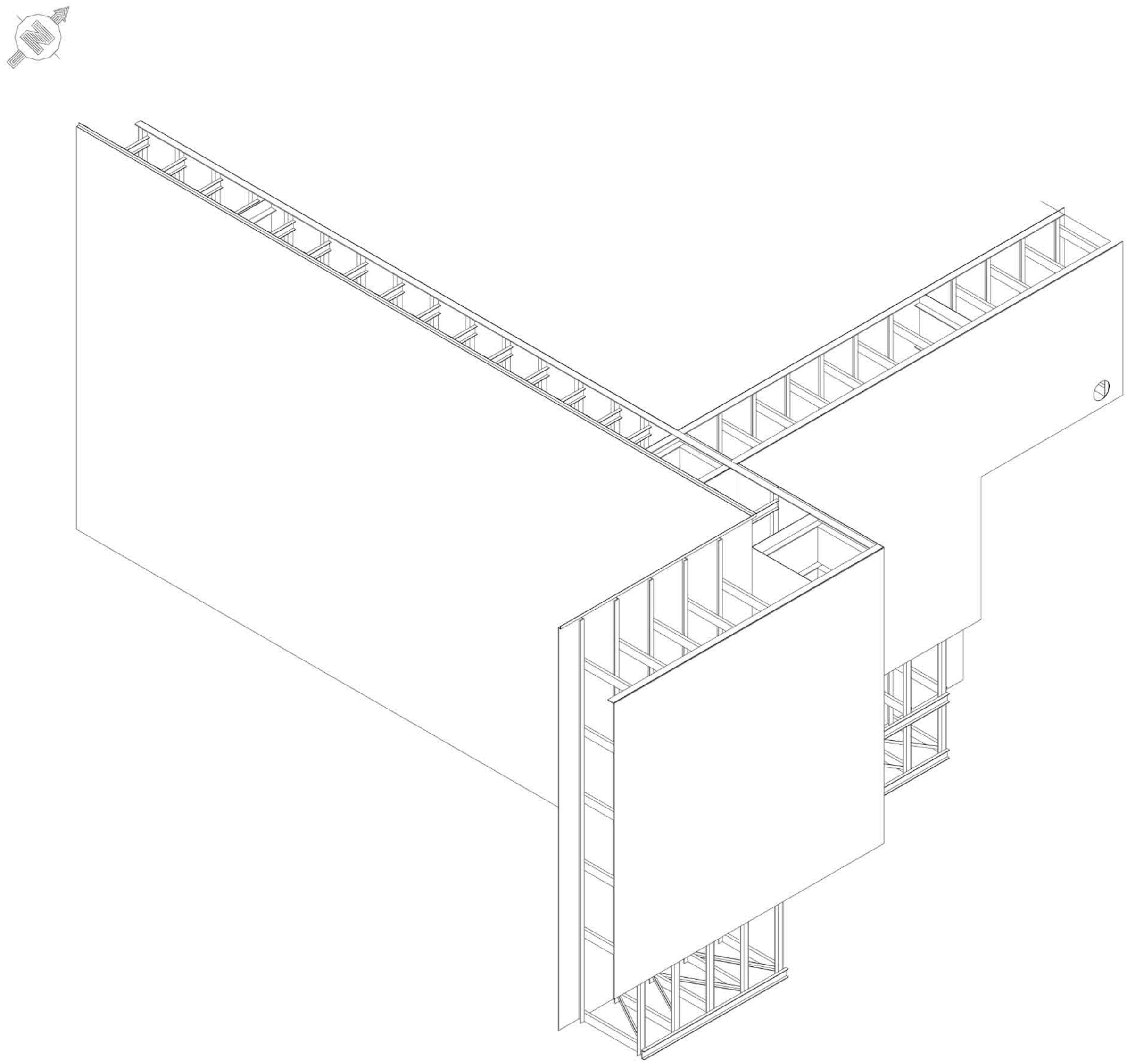
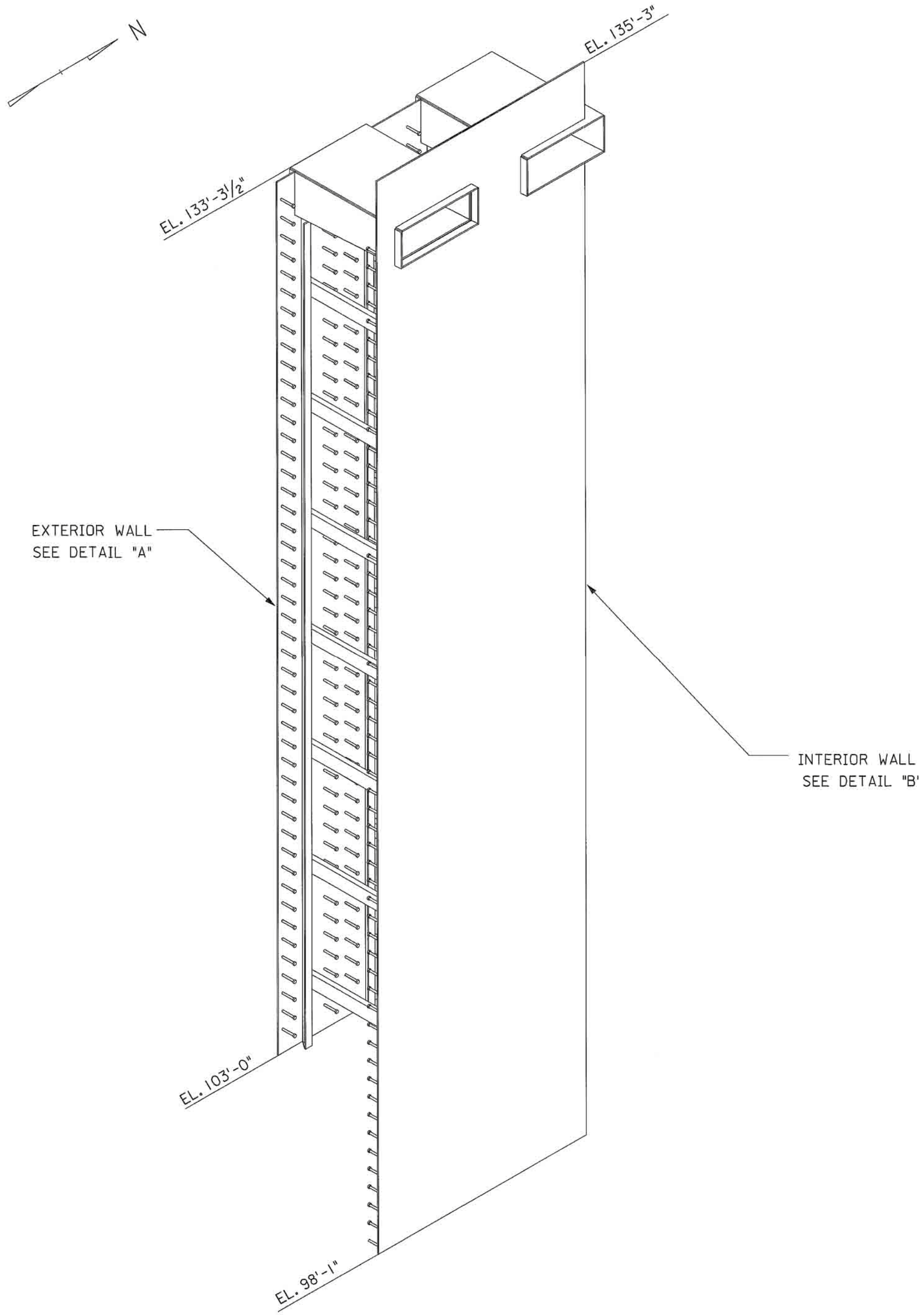


Figure 3.8.3-14 (Sheet 5 of 5)
[*CA-05 Module*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



Note: See Figure 3.8.3-8 for fabrication detail.

Figure 3.8.3-15 (Sheet 1 of 2)
[*Typical Submodule*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

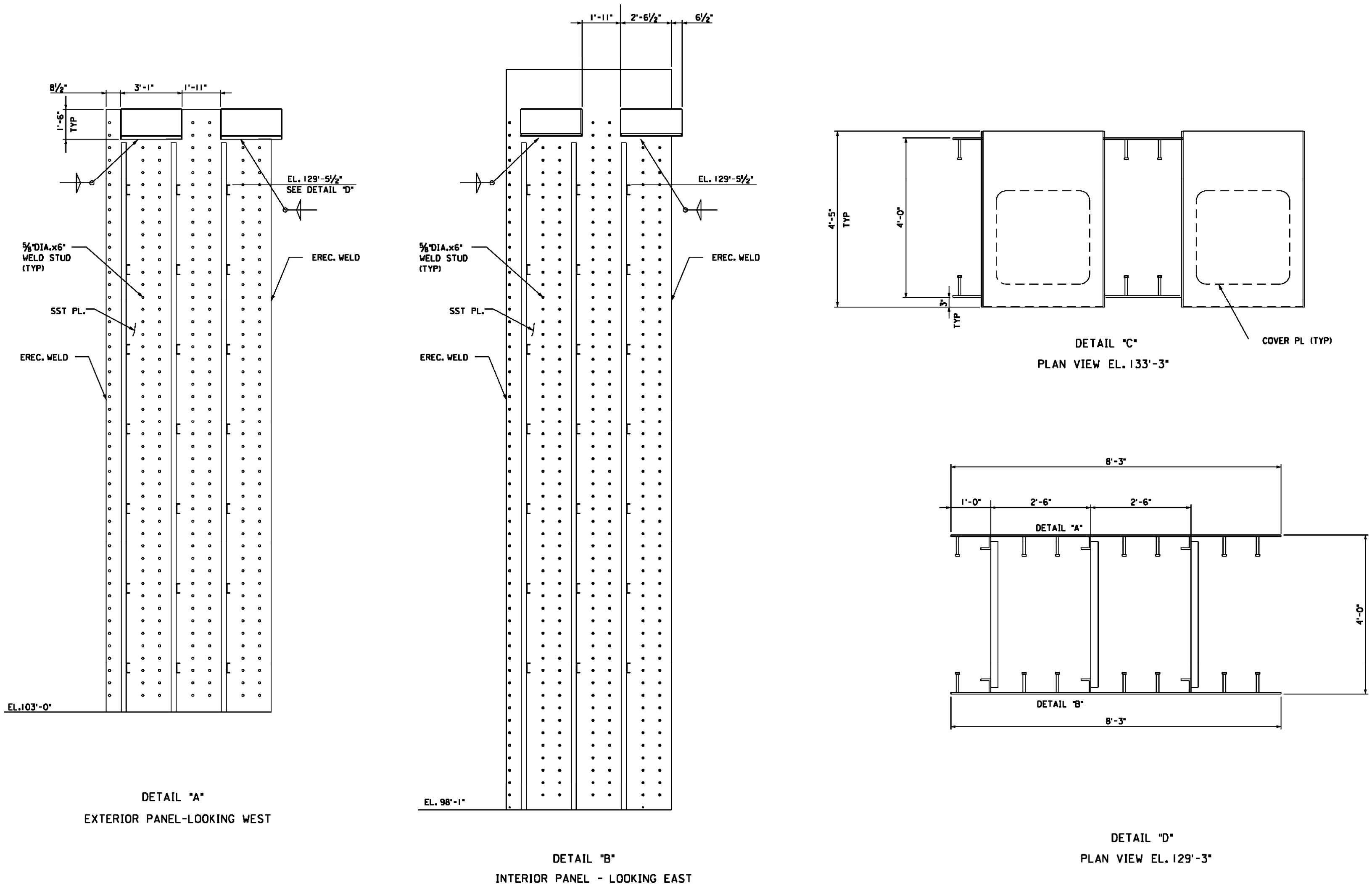


Figure 3.8.3-15 (Sheet 2 of 2)
[*Typical Submodule*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

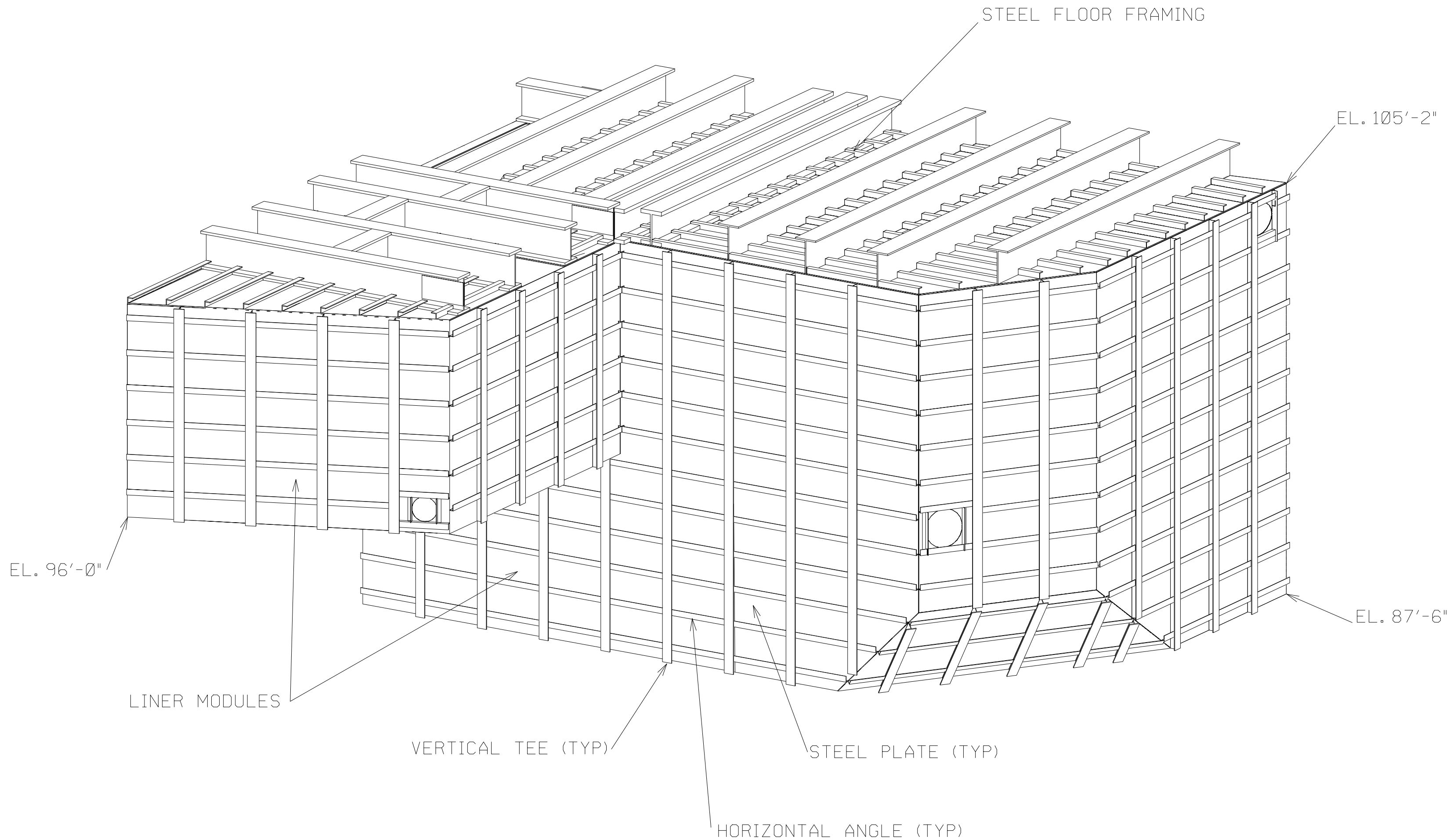
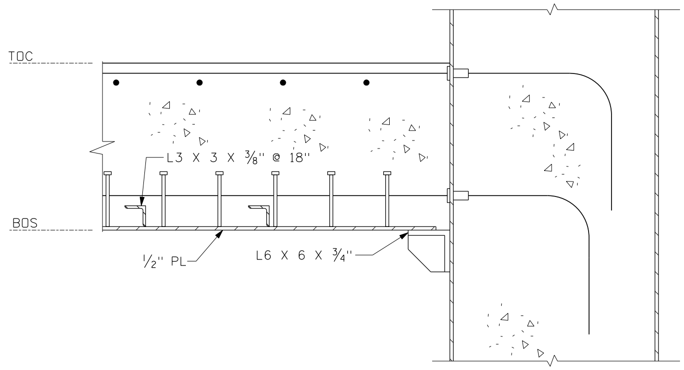
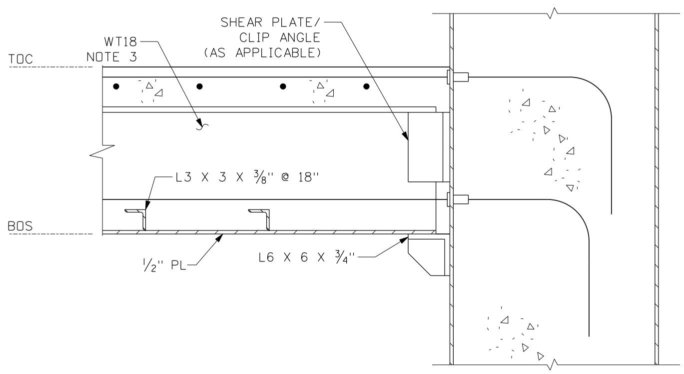


Figure 3.8.3-16
Liner Modules



SECTION BETWEEN BEAMS IN FLOOR
NOTES: 1, 2, 3, 4, 5, 6, 8 & 9



SECTION AT BEAMS IN FLOOR
NOTES: 1, 2, 3, 4, 5, 6, 7 & 8

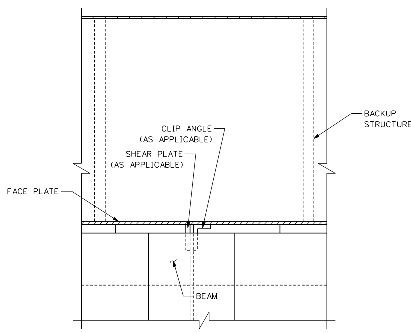
NOTES:

1. DETAILS SHOWN ARE REPRESENTATIVE OF FLOOR MODULES AT EL 107'-2" AND 135'-3" AND THE CONNECTIONS OF FLOOR MODULES TO WALL MODULES INSIDE CONTAINMENT. REFER TO SUBSECTION 3.8.3.1.3 AND OTHER NOTES FOR ADDITIONAL INFORMATION ABOUT DESIGN DETAILS AND VARIATIONS FOR FLOOR MODULE TO WALL MODULE CONNECTIONS.
2. REINFORCEMENT USE, SIZE, AND SPACING IN THE FLOOR MODULE CONCRETE SATISFY THE REQUIREMENTS IN ACI 349. WHERE USED, THE REINFORCEMENT SIZE RANGE IS #7 TO #11. THE DESIGN OF THE STANDARD HOOKS IN THE WALL MODULES AND THE COUPLERS CONNECTING THE FLOOR REINFORCEMENT TO THE HOOKS IN THE WALL MODULES SATISFIES THE REQUIREMENTS OF ACI 349.
3. DESIGN OF THE PLATES, BEAMS, AND STIFFENERS IN THE FLOOR, INCLUDING PLATE SIZE AND SPACING, AND TYPE, SIZE, AND SPACING OF STRUCTURAL SHAPES VARIES AND SATISFIES THE REQUIREMENTS OF AISC N690.
4. THE REINFORCEMENT AND FLOOR DESIGN ELEMENTS SHOWN ARE FOR LOCATIONS AWAY FROM OPENINGS, PENETRATIONS, AND OTHER OBSTRUCTIONS.
5. IN SOME LOCATIONS, REINFORCEMENT AT THE CONNECTION IS NOT REQUIRED BECAUSE THE LOADS ARE TRANSFERRED THROUGH THE STEEL SECTION DIRECTLY TO THE BACKUP STRUCTURES OR THE SEAT ANGLE. THE DESIGN OF THE FLOORS SATISFIES THE APPLICABLE REQUIREMENTS IN ACI 349 AND AISC N690.
6. THE DETAIL DESIGN, LOCATION, AND ATTACHMENT OF THE FLOOR AND BEAM SUPPORTS ARE DESIGNED TO THE REQUIREMENTS OF AISC N690. SUPPORT CONFIGURATIONS, INCLUDING THE USE OF PLATES, STRUCTURAL SHAPES, AND STIFFENERS, ARE BASED ON LOADING AND LOCAL GEOMETRY CONSIDERATIONS.
7. THE DESIGNS OF THE CONNECTIONS AND BACKUP STRUCTURES WITHIN THE MODULES WALL, INCLUDING PLATE SIZE, AND SPACING, AND TYPE, SIZE, AND SPACING OF STRUCTURAL SHAPES VARY AND SATISFY THE REQUIREMENTS OF AISC N690.
8. THE THICKNESS OF THE ADJACENT WALL IS BASED ON THE WALL DESIGN REQUIREMENTS AND LOCATION.
9. REFER TO SUBSECTION 3.8.3.5.4 FOR INFORMATION ABOUT THE SHEAR STUDS IN THE FLOOR MODULE.

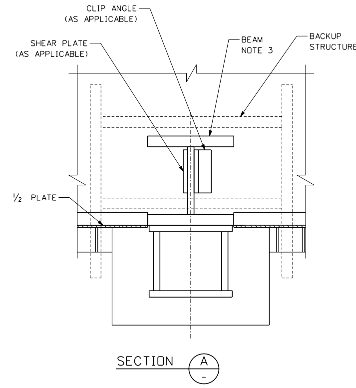
SEE SUBSECTION 3.8.3.5.8.1 FOR INFORMATION ON TIER 2* DESIGNATION.

Figure 3.8.3-17 (Sheet 1 of 2)
[*Structural Modules – Design Details Standard Floor Connection*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

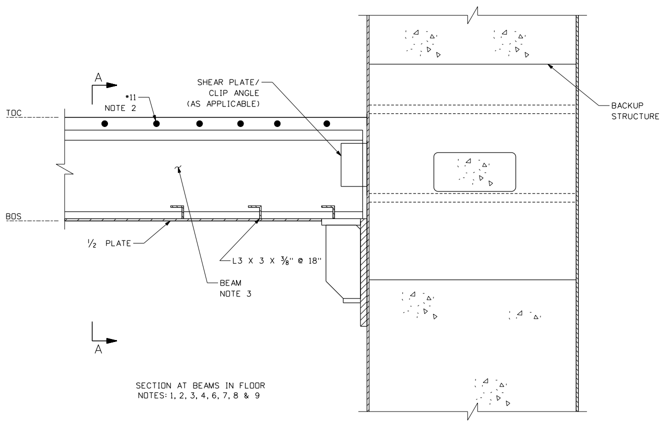


TOP VIEW AT BEAMS IN FLOOR
NOTE: 7

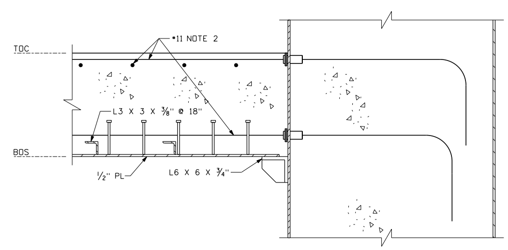


END VIEW AT BEAMS IN FLOOR
NOTES: 3, 6, 7 & 9

NOTES:
1. DETAILS SHOWN ARE REPRESENTATIVE OF FLOOR MODULES AT EL 107'-2" AND 135'-3" AND THE CONNECTIONS OF FLOOR MODULES TO WALL MODULES INSIDE CONTAINMENT. REFER TO SUBSECTION 3.8.3.1.3 AND OTHER NOTES FOR ADDITIONAL INFORMATION ABOUT DESIGN DETAILS AND VARIATIONS FOR FLOOR MODULE TO WALL MODULE CONNECTIONS.
2. REINFORCEMENT USE, SIZE, AND SPACING IN THE FLOOR MODULE CONCRETE SATISFY THE REQUIREMENTS IN ACI 349. WHERE USED, THE REINFORCEMENT SIZE RANGE IS #7 TO #11. THE DESIGN OF THE STANDARD HOOKS IN THE WALL MODULES AND THE COUPLERS CONNECTING THE FLOOR REINFORCEMENT TO THE HOOKS IN THE WALL MODULES SATISFIES THE REQUIREMENTS OF ACI 349.
3. DESIGN OF THE PLATES, BEAMS, AND STIFFENERS IN THE FLOOR, INCLUDING PLATE SIZE AND SPACING, AND TYPE, SIZE, AND SPACING OF STRUCTURAL SHAPES VARIES AND SATISFIES THE REQUIREMENTS OF AISC N690.
4. THE REINFORCEMENT AND FLOOR DESIGN ELEMENTS SHOWN ARE FOR LOCATIONS AWAY FROM OPENINGS, PENETRATIONS, AND OTHER OBSTRUCTIONS.
5. IN SOME LOCATIONS, REINFORCEMENT AT THE CONNECTION IS NOT REQUIRED BECAUSE THE LOADS ARE TRANSFERRED THROUGH THE STEEL SECTION DIRECTLY TO THE BACKUP STRUCTURES OR THE SEAT ANGLE. THE DESIGN OF THE FLOORS SATISFIES THE APPLICABLE REQUIREMENTS IN ACI 349 AND AISC N690.
6. THE DETAIL DESIGN, LOCATION, AND ATTACHMENT OF THE FLOOR AND BEAM SUPPORTS ARE DESIGNED TO THE REQUIREMENTS OF AISC N690. SUPPORT CONFIGURATIONS, INCLUDING THE USE OF PLATES, STRUCTURAL SHAPES, AND STIFFENERS, ARE BASED ON LOADING AND LOCAL GEOMETRY CONSIDERATIONS.
7. THE DESIGNS OF THE CONNECTIONS AND BACKUP STRUCTURES WITHIN THE MODULES WALL, INCLUDING PLATE SIZE AND SPACING, AND TYPE, SIZE, AND SPACING OF STRUCTURAL SHAPES VARY AND SATISFY THE REQUIREMENTS OF AISC N690.
8. THE THICKNESS OF THE ADJACENT WALL IS BASED ON THE WALL DESIGN REQUIREMENTS AND LOCATION.
9. THE BEAM SEAT SHOWN IS NOT USED AT ALL BEAM LOCATIONS. THE SEAT ANGLE SHOWN ON SHEET 1 IS USED AT OTHER LOCATIONS.
10. REFER TO SUBSECTION 3.8.3.5.4 FOR INFORMATION ABOUT THE SHEAR STUDS IN THE FLOOR MODULE.



SECTION AT BEAMS IN FLOOR
NOTES: 1, 2, 3, 4, 6, 7, 8 & 9



SECTION BETWEEN BEAMS IN FLOOR
NOTES: 1, 2, 3, 4, 5, 6, 8 & 10

SEE SUBSECTION 3.8.3.5.8.1 FOR INFORMATION ON TIER 2* DESIGNATION.

Figure 3.8.3-17 (Sheet 2 of 2)
[*Structural Modules – Design Details Heavily Loaded Floor Connection*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

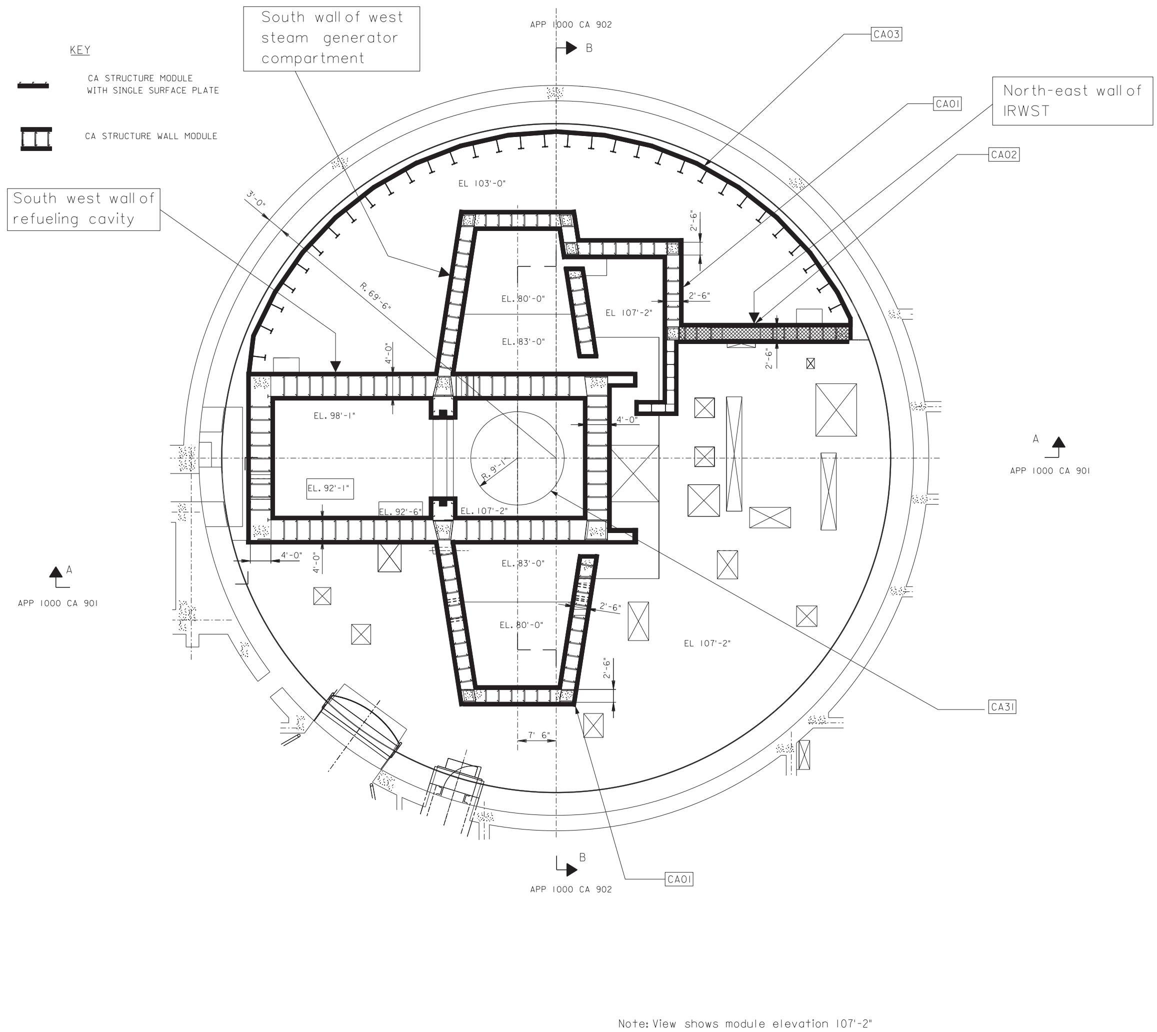
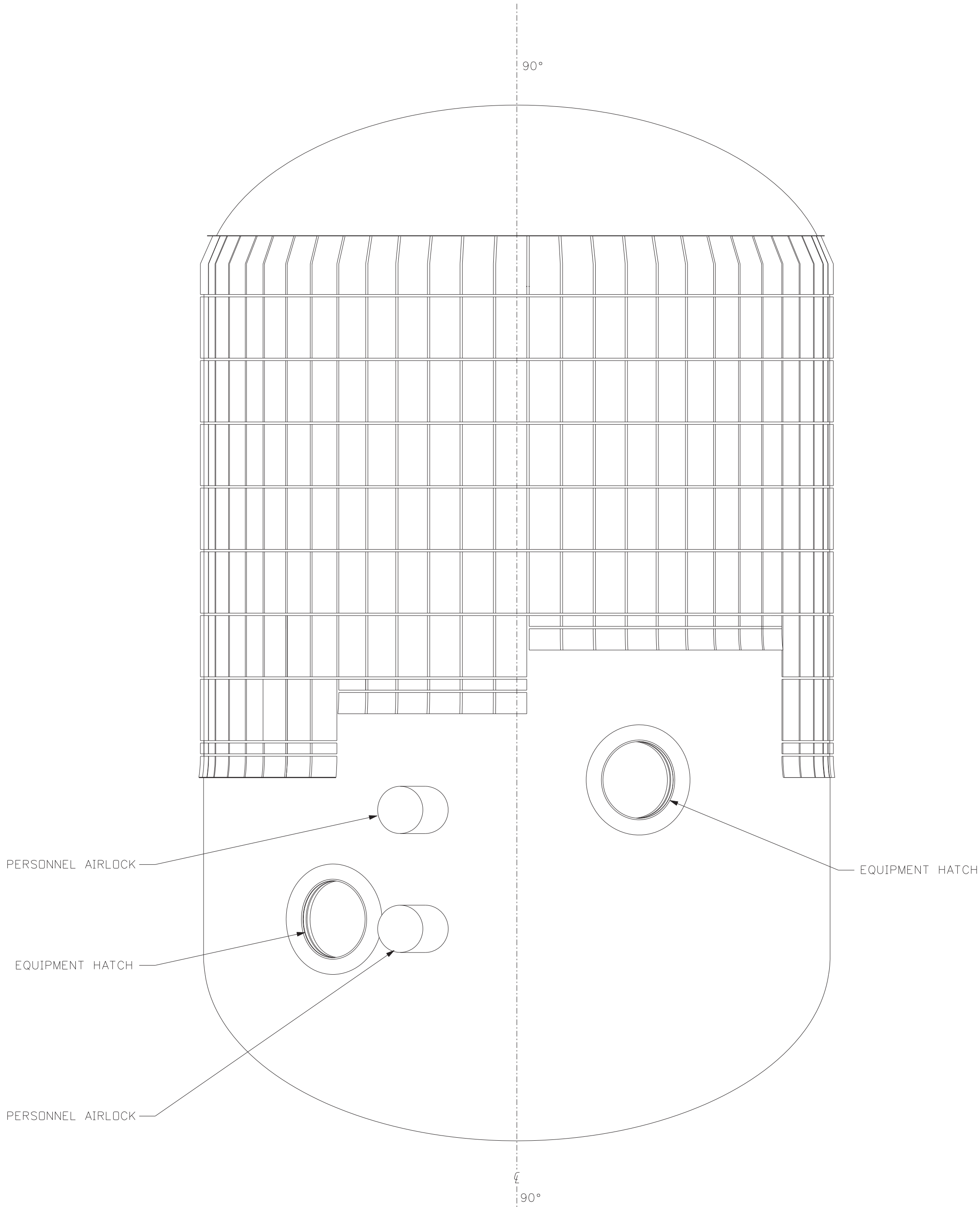


Figure 3.8.3-18
[*Location of Structural Wall Modules*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.



NOTE: COURSE LAYOUT, PLATE/INSERT PLATE GEOMETRY, WELD SEAMS FOR THE CONTAINMENT, HATCHES, AIRLOCKS, AND THE NUMBER AND CONFIGURATION OF THE CONTAINMENT AIR BAFFLE PANELS ARE PROVIDED FOR ILLUSTRATIVE PURPOSES ONLY.

**Figure 3.8.4-1 (Sheet 1 of 4)
Containment Air Baffle
General Arrangement**

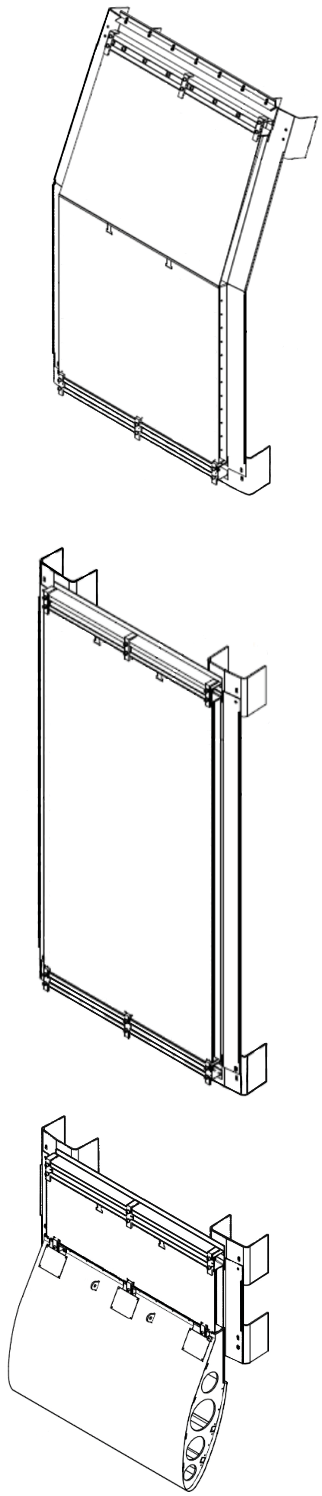


Figure 3.8.4-1 (Sheet 2 of 4)
Containment Air Baffle
Panel Types

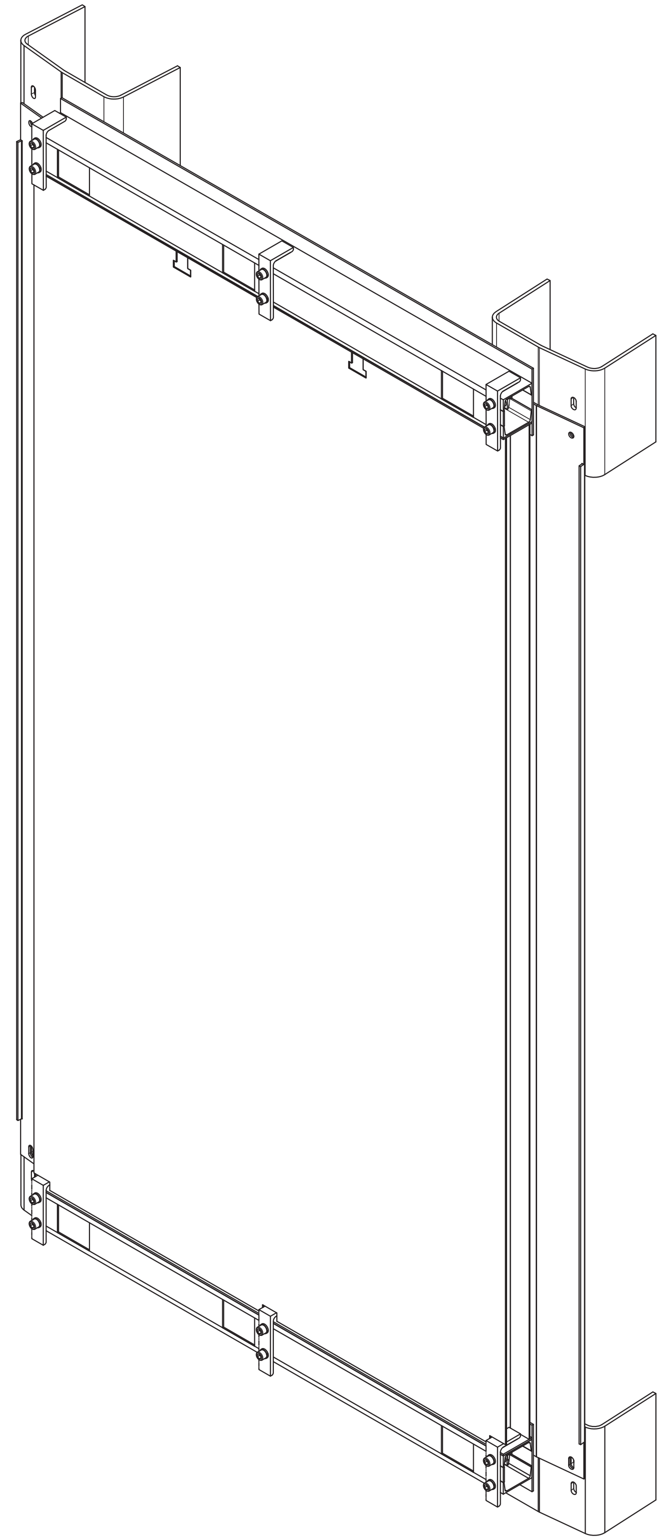


Figure 3.8.4-1 (Sheet 3 of 4)
Containment Air Baffle
Typical Panel on Cylinder

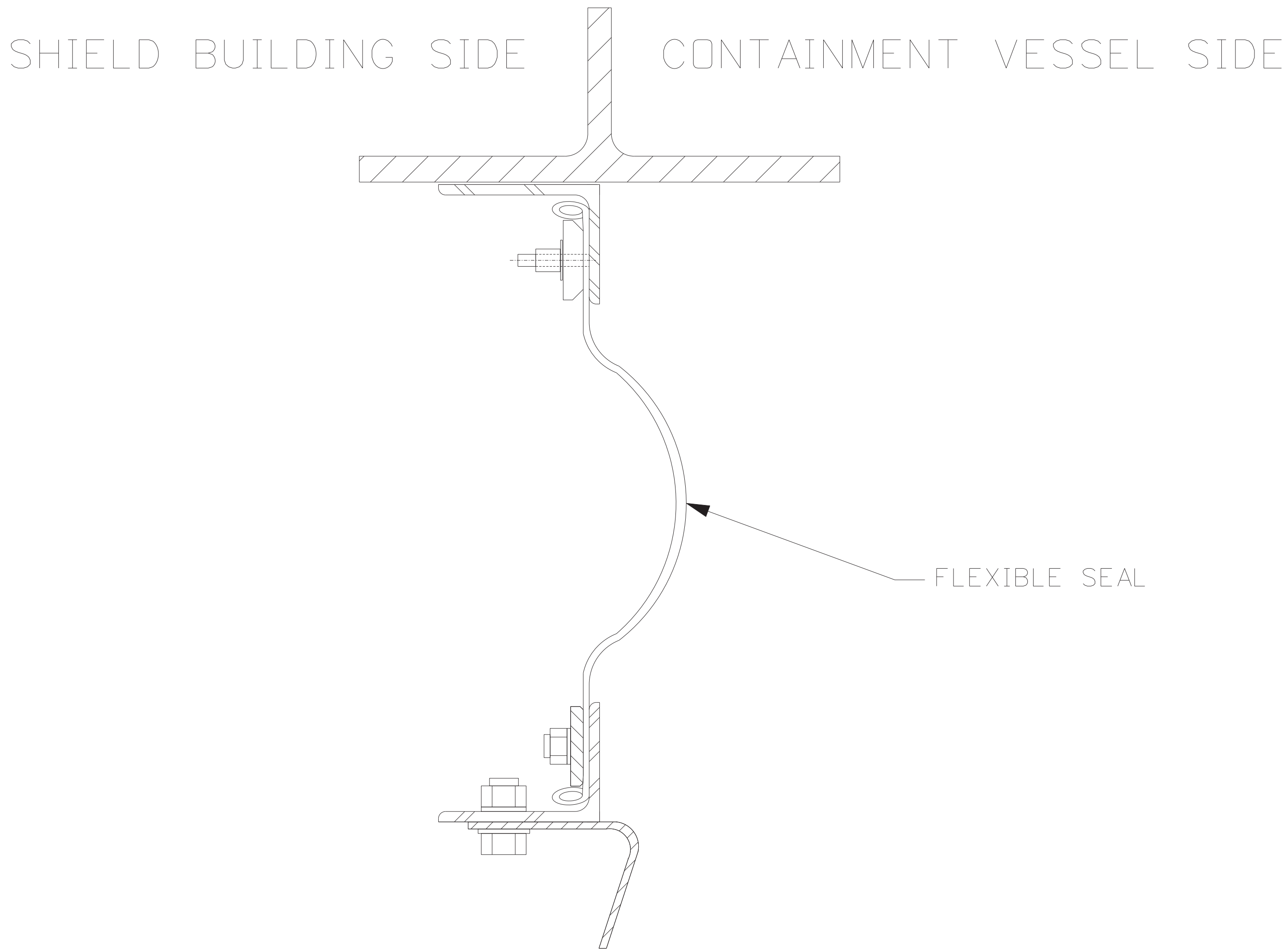


Figure 3.8.4-1 (Sheet 4 of 4)
Containment Air Baffle
Flexible Seal

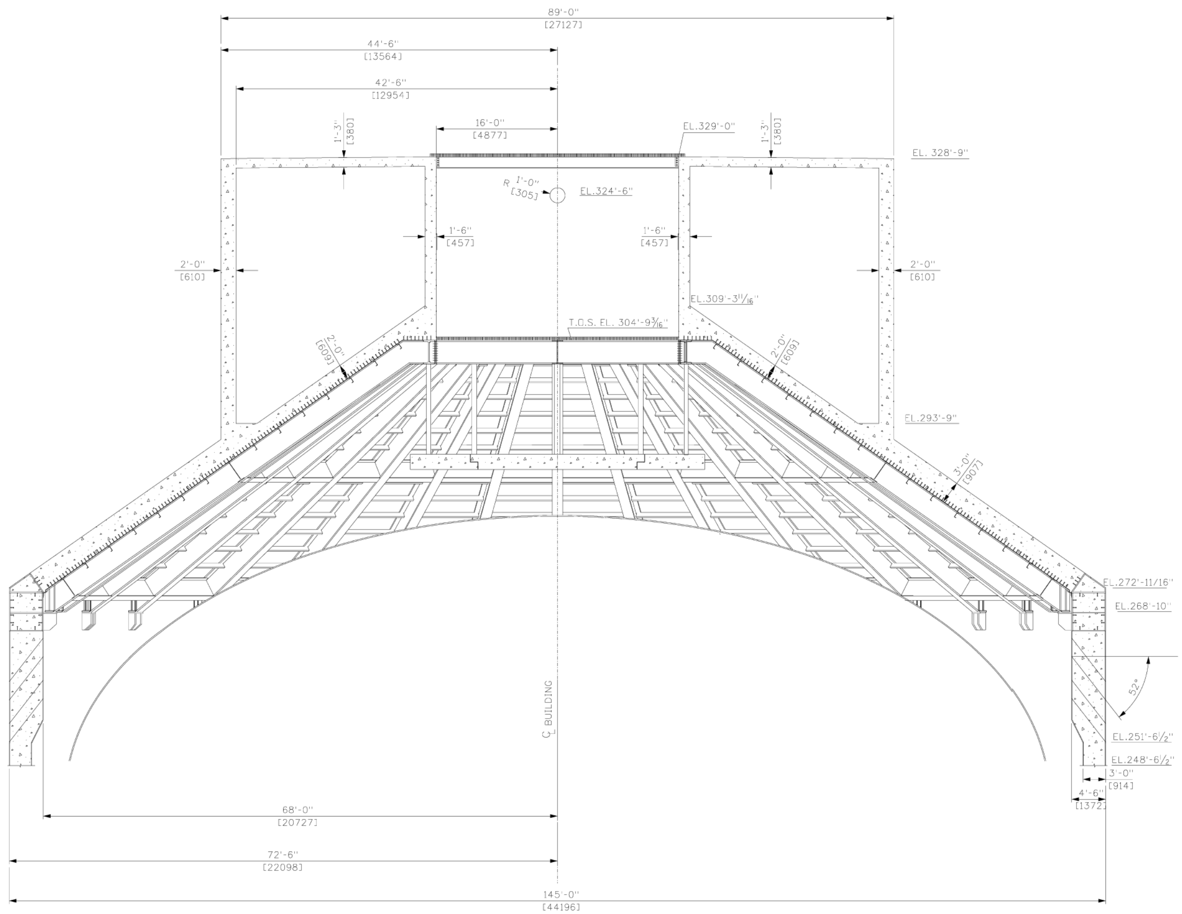


Figure 3.8.4-2
[*Passive Containment Cooling Tank*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

Figure 3.8.4-3 Not Used

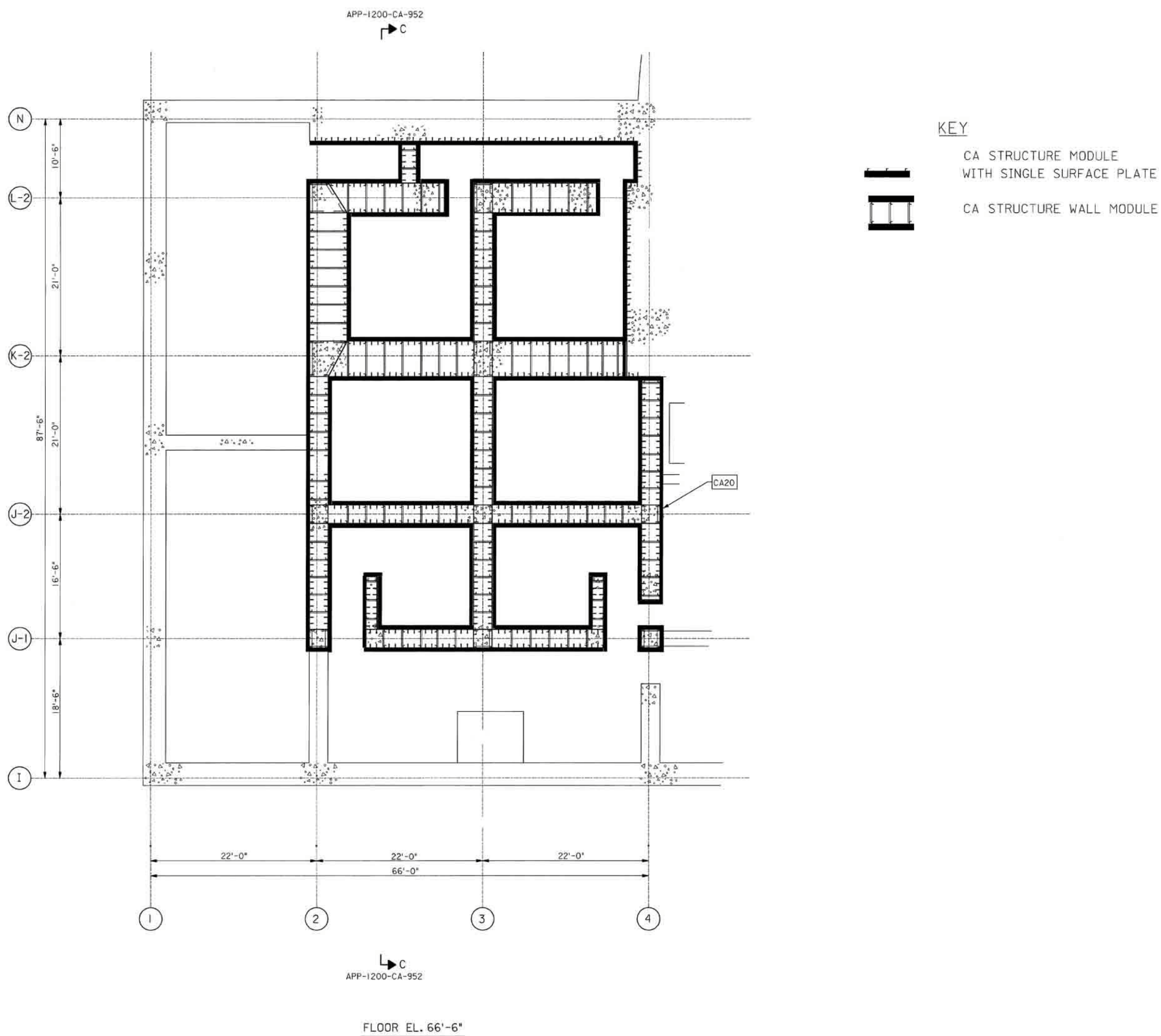


Figure 3.8.4-4 (Sheet 1 of 5)
[*Structural Modules in Auxiliary Building*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

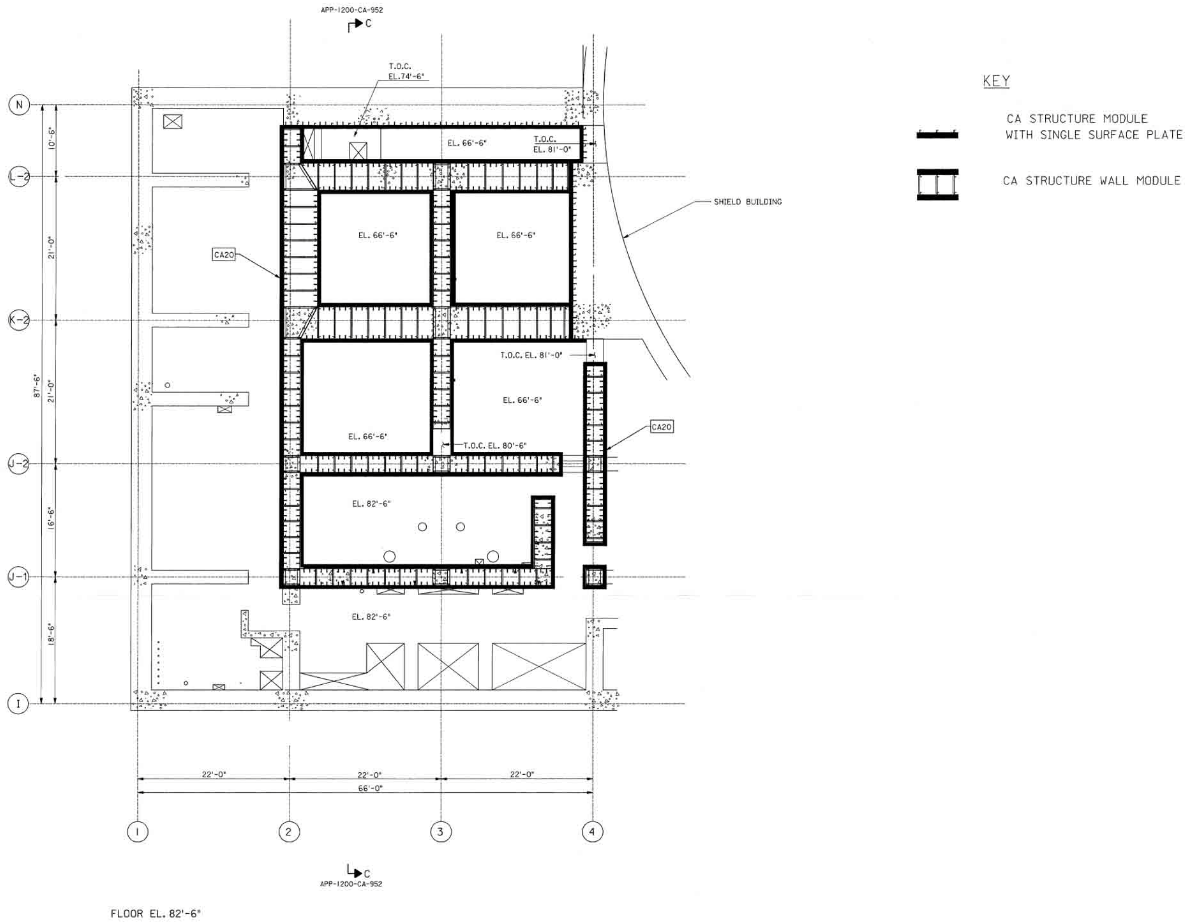


Figure 3.8.4-4 (Sheet 2 of 5)
[*Structural Modules in Auxiliary Building*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

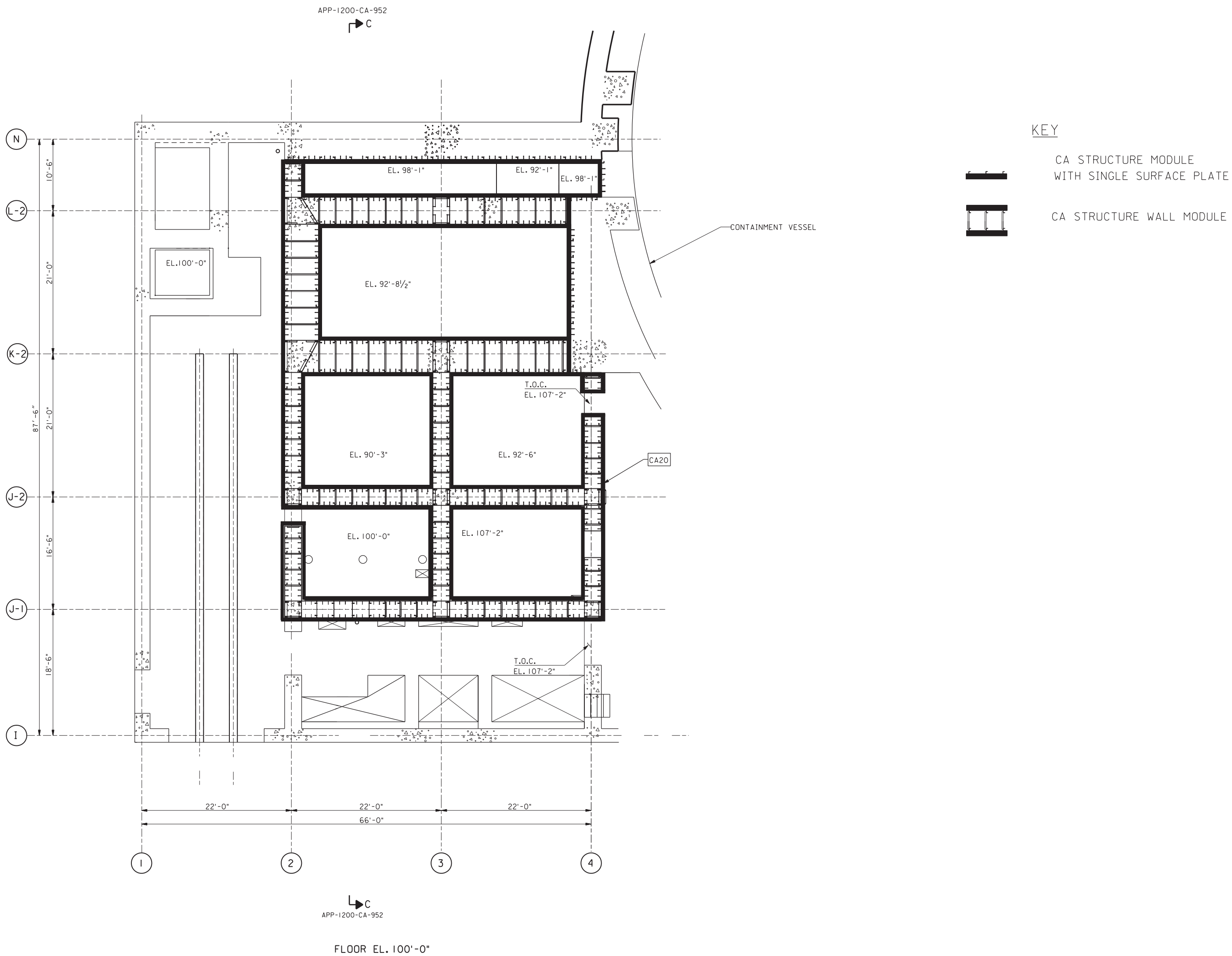


Figure 3.8.4-4 (Sheet 3 of 5)
[*Structural Modules in Auxiliary Building*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

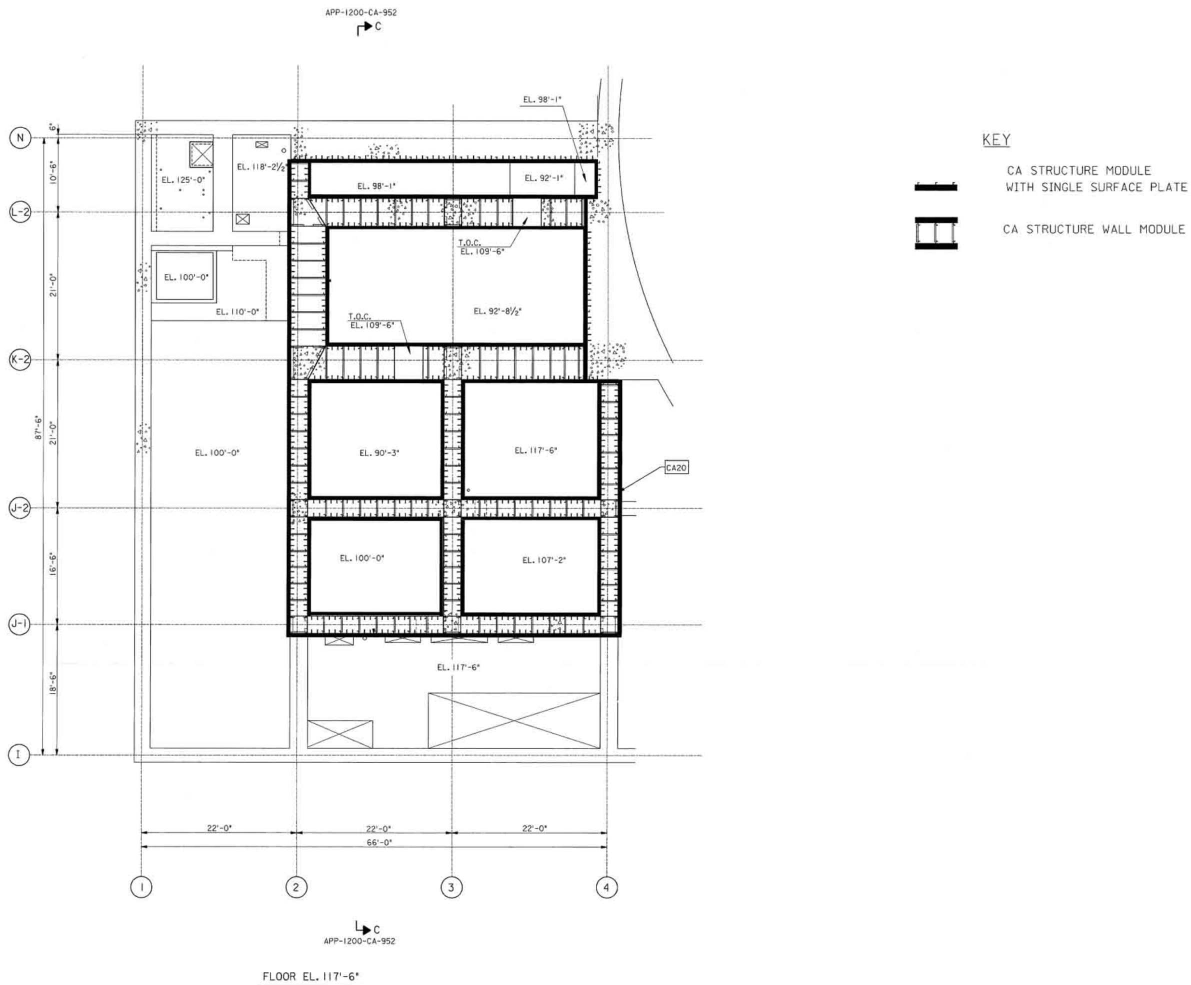


Figure 3.8.4-4 (Sheet 4 of 5)
[*Structural Modules in Auxiliary Building*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

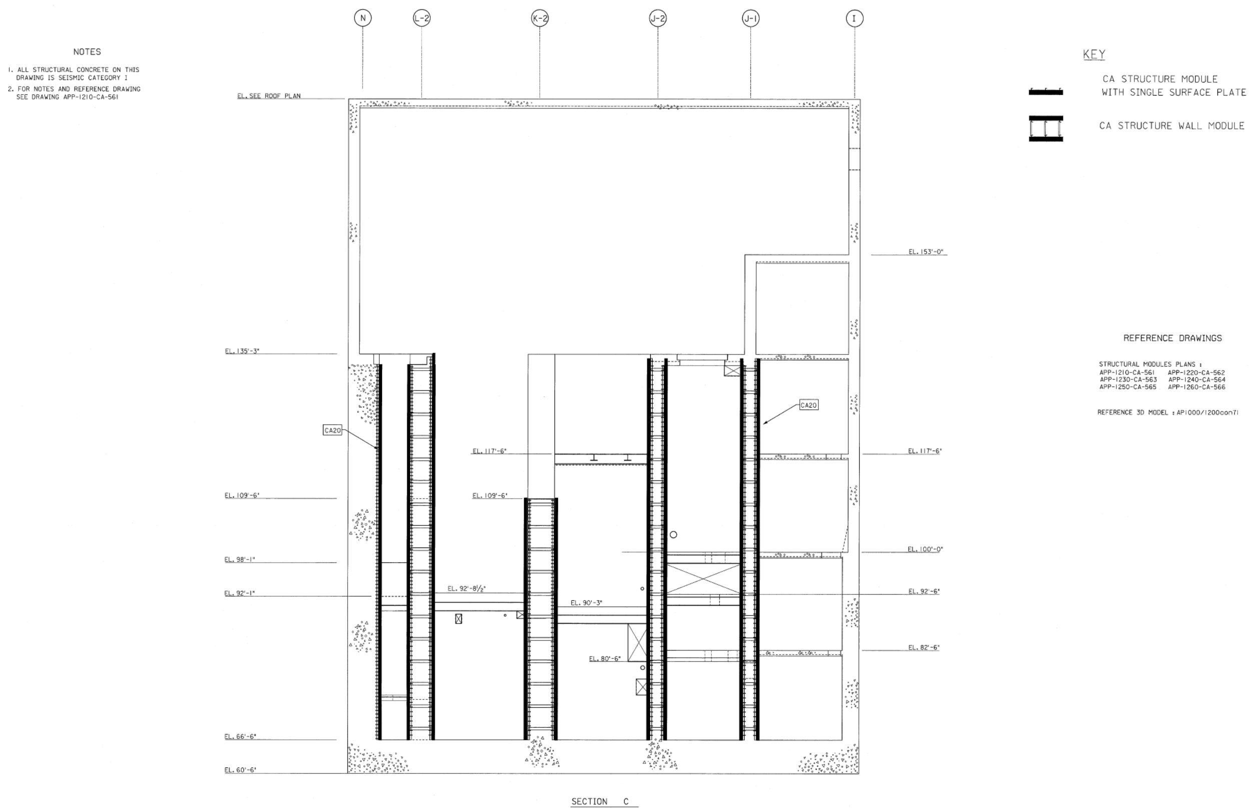


Figure 3.8.4-4 (Sheet 5 of 5)
[*Structural Modules in Auxiliary Building*]*

*In accordance with the departure evaluation process specified in License Condition 2.D.(13), NRC Staff approval may be required prior to implementing a change in this information.

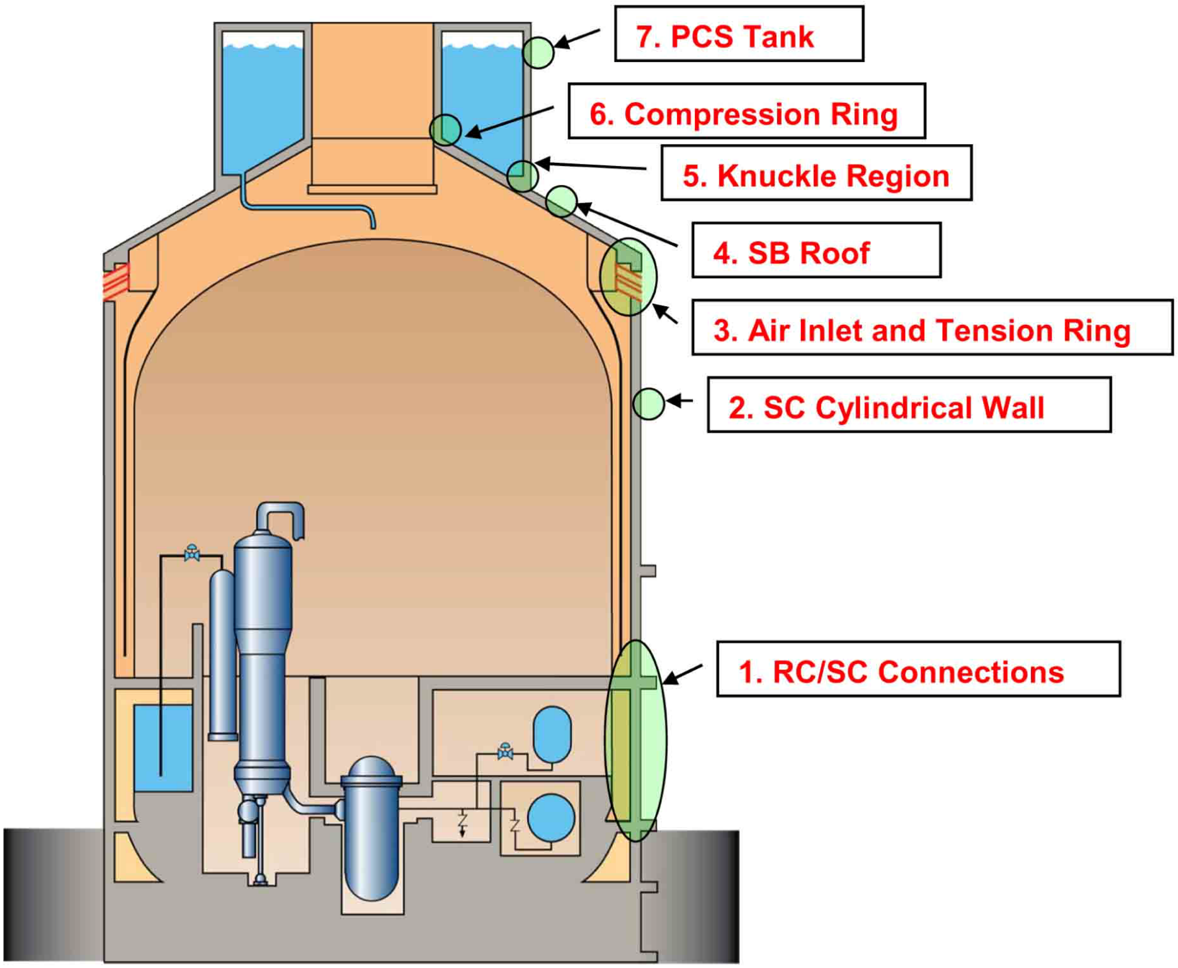


Figure 3.8.4-5
Shield Building Structure Key Areas

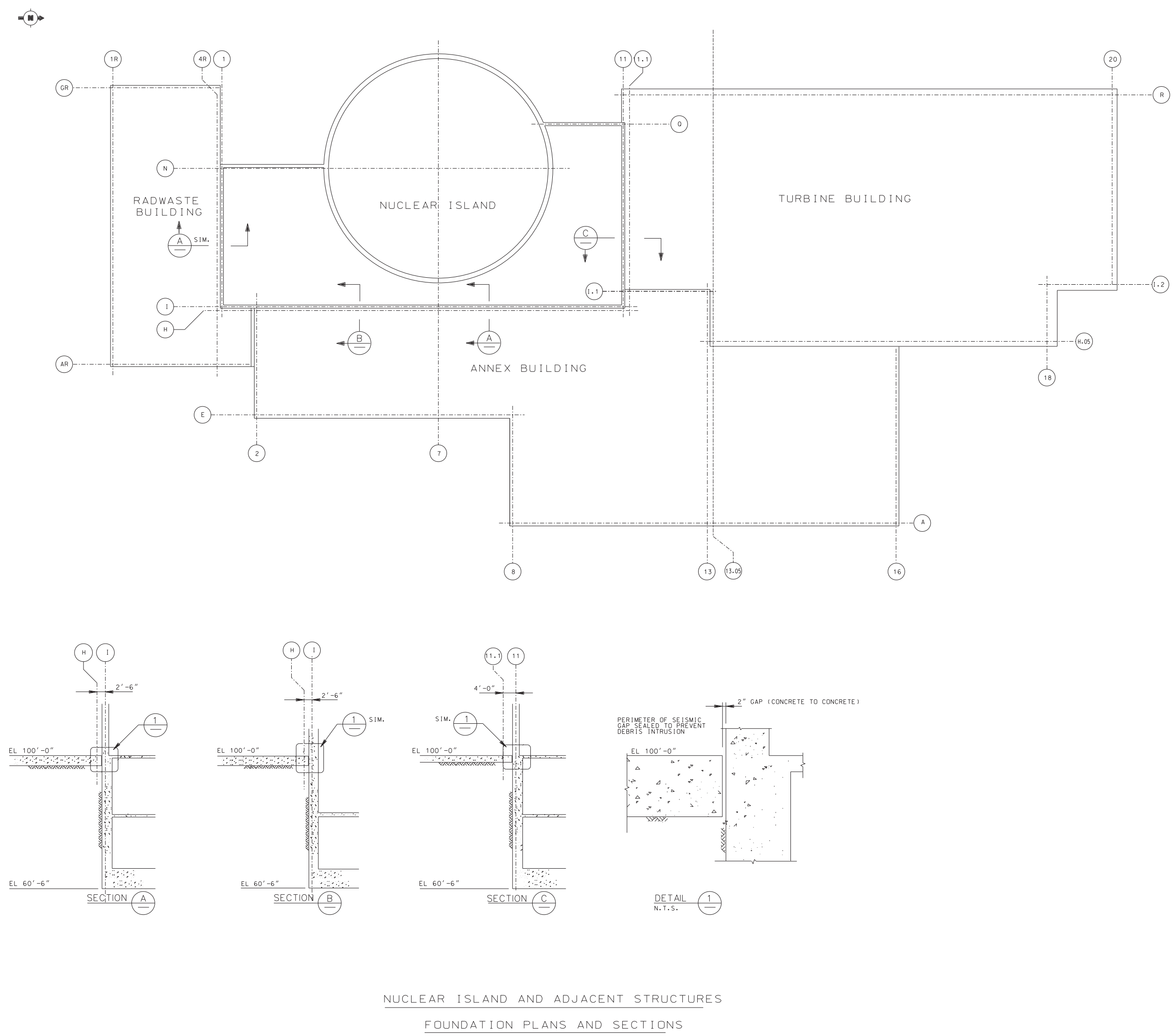


Figure 3.8.5-1
Foundation Plan

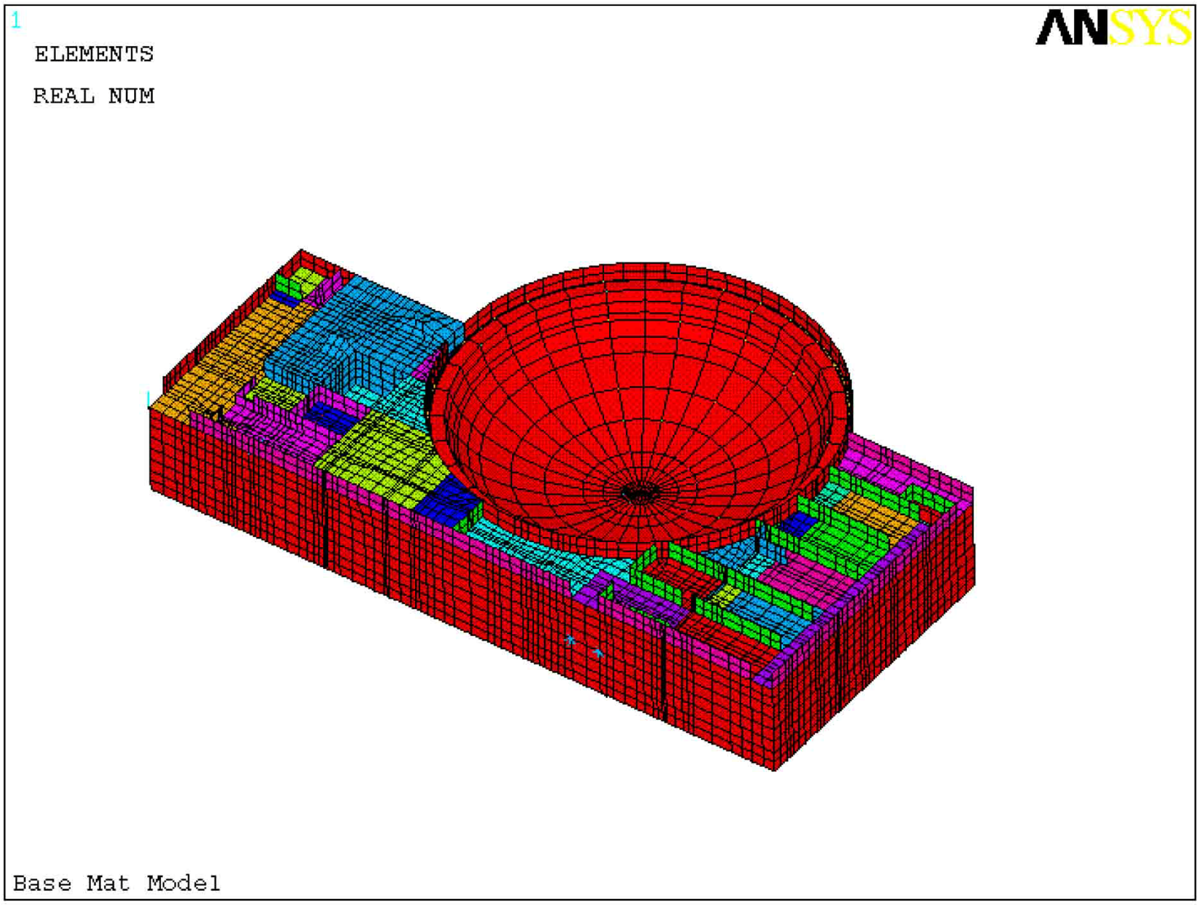


Figure 3.8.5-2
Isometric View of Finite Element Model

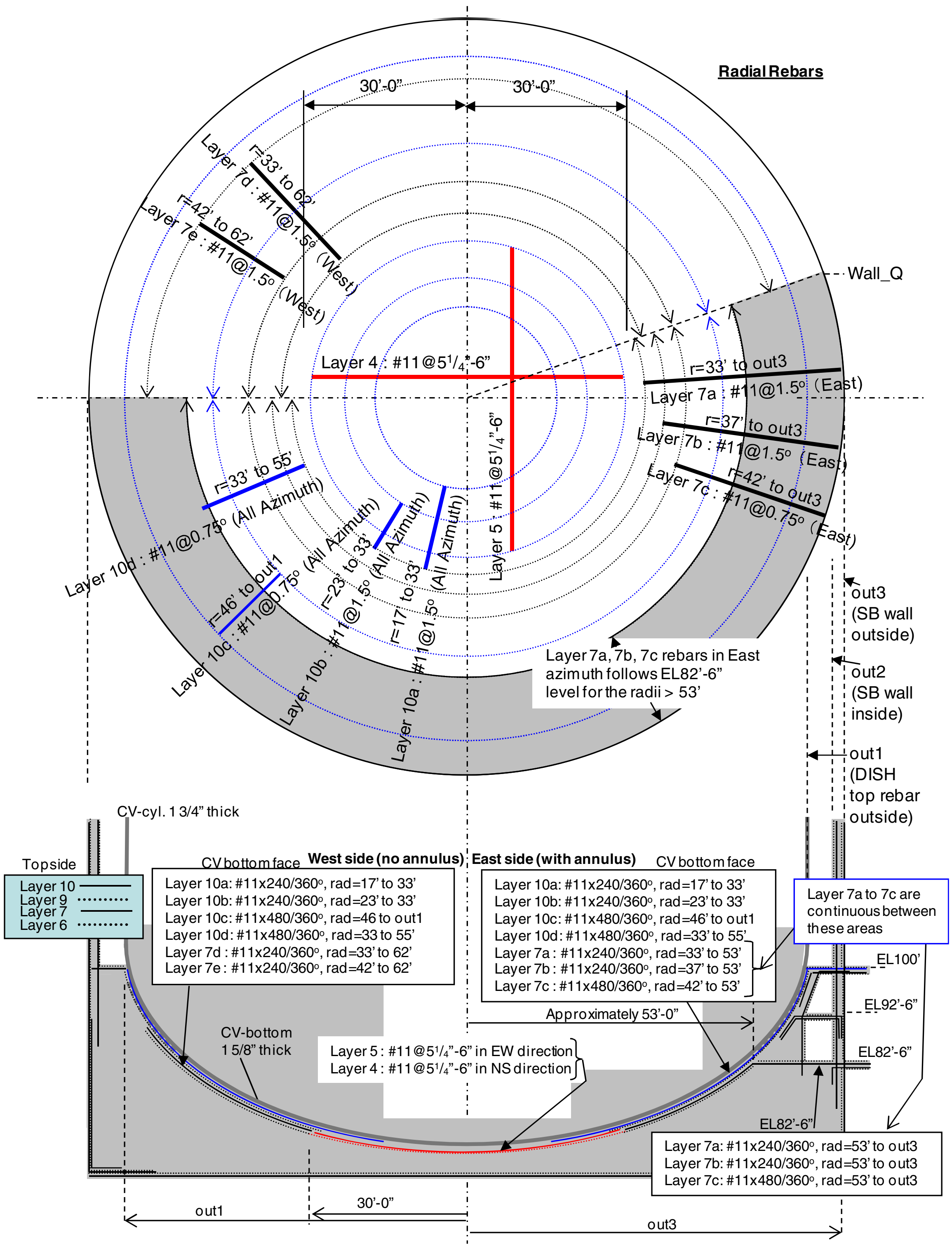


Figure 3.8.5-3 (Sheet 1 of 7)
Radial Reinforcement, Top Side of DISH

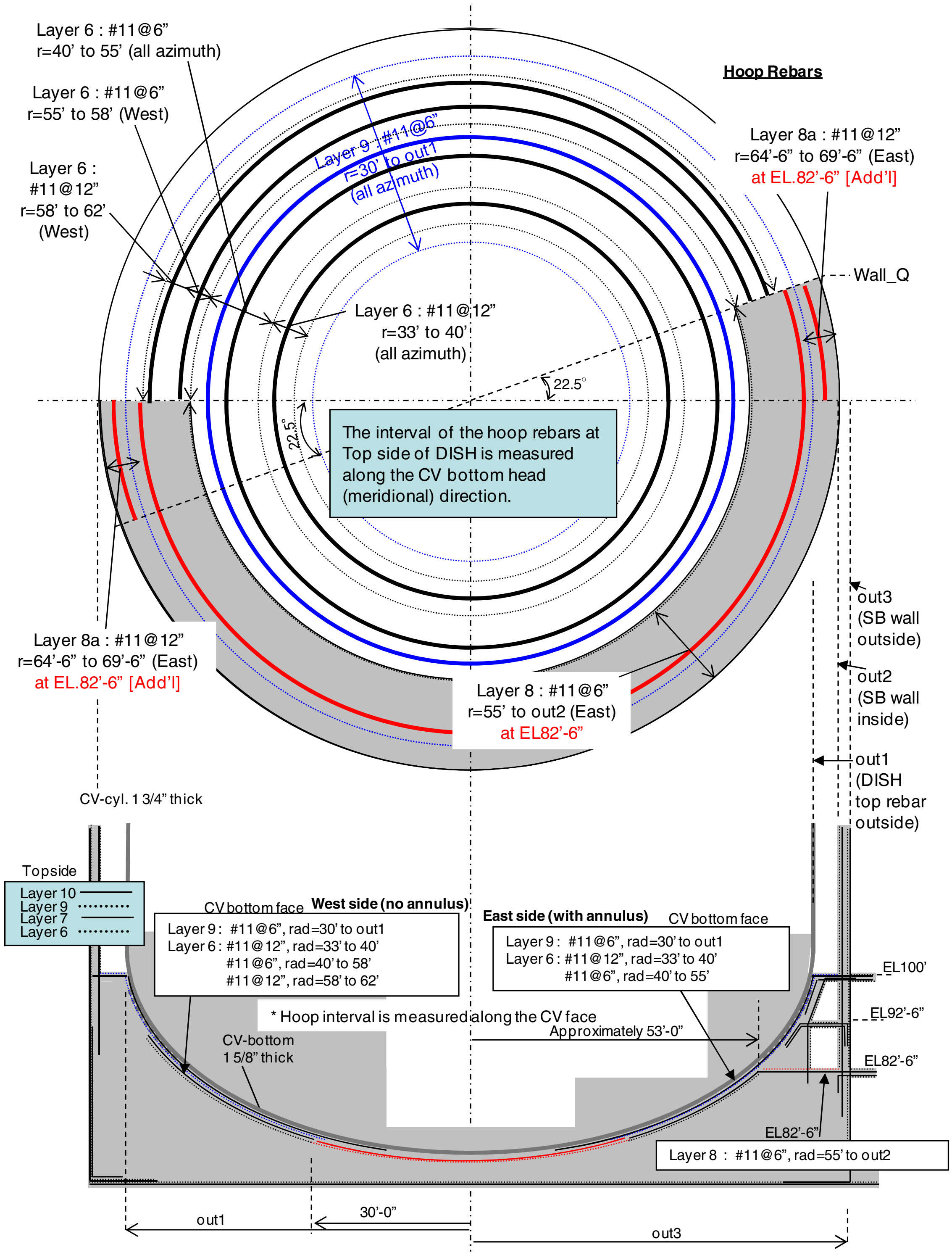
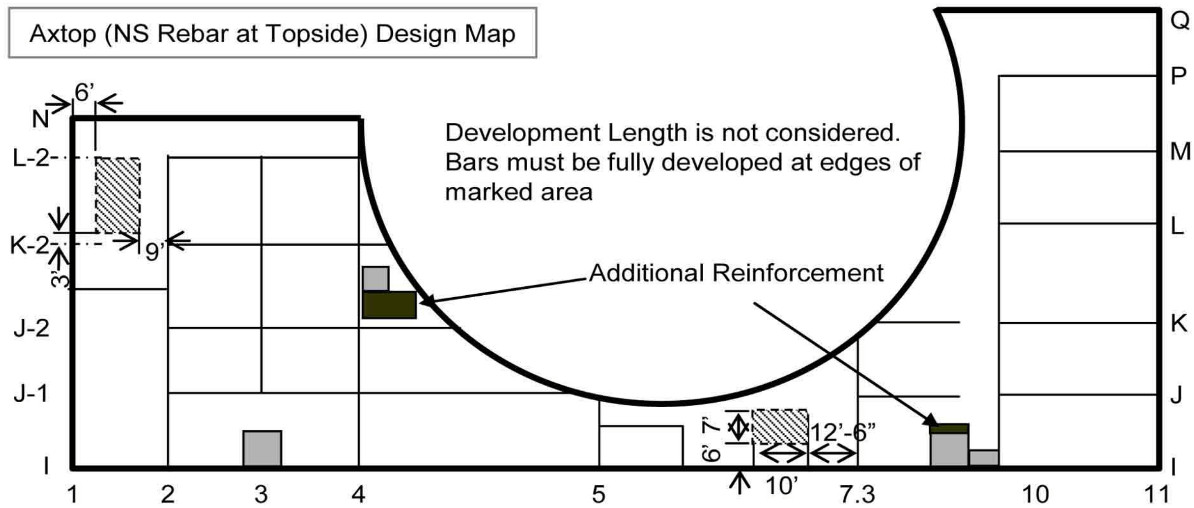
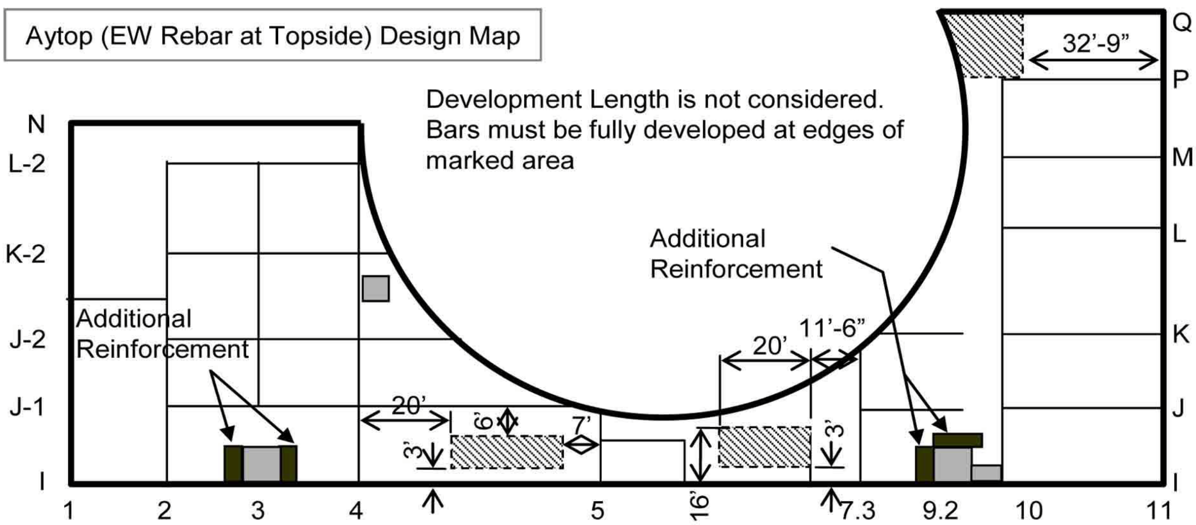


Figure 3.8.5-3 (Sheet 2 of 7)
Circumferential Reinforcement, Top Side of DISH



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #11@12"
Location	Layer 4		Top side of the 2' & 3' thick Pits

Figure 3.8.5-3 (Sheet 3 of 7)
Longitudinal Reinforcement Map, Top Side in NS Direction



Rebar arrangement	: #14@12”　　: #14@12”+#9@12”	: #11@12”
Location	Layer 5	Top side of the 2’ & 3’ thick Pits

Figure 3.8.5-3 (Sheet 4 of 7)
Longitudinal Reinforcement Map,
Top Side in EW Direction

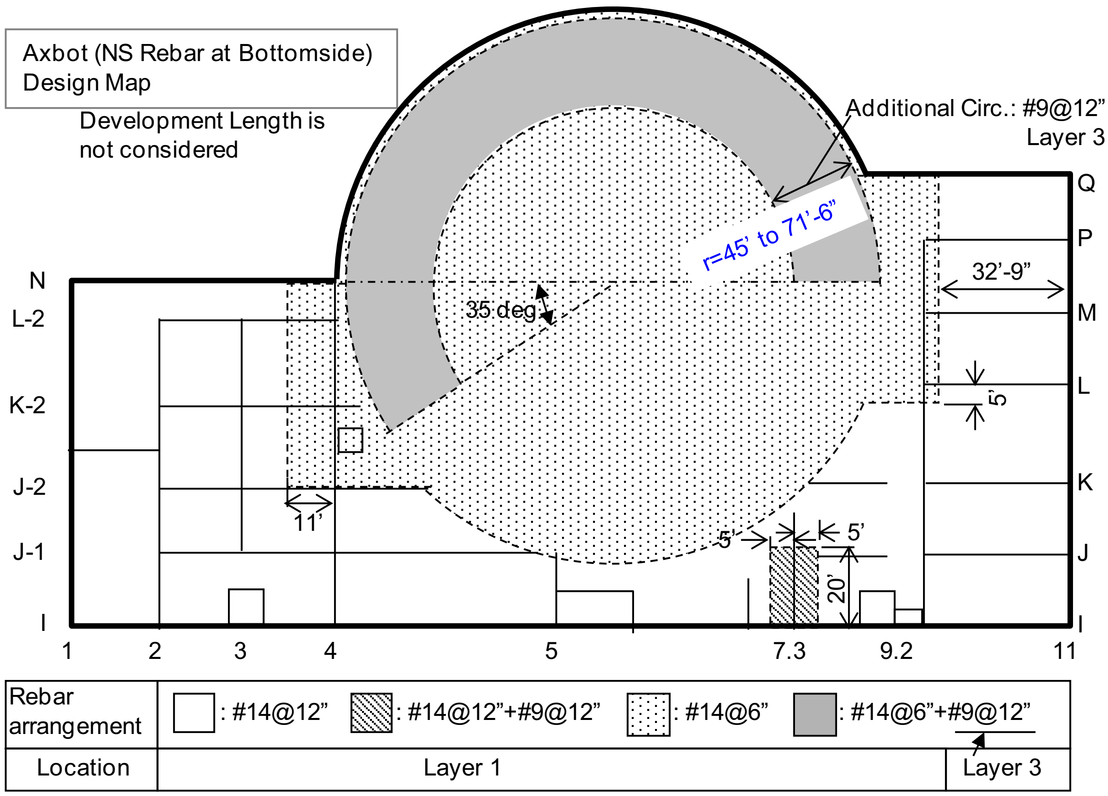
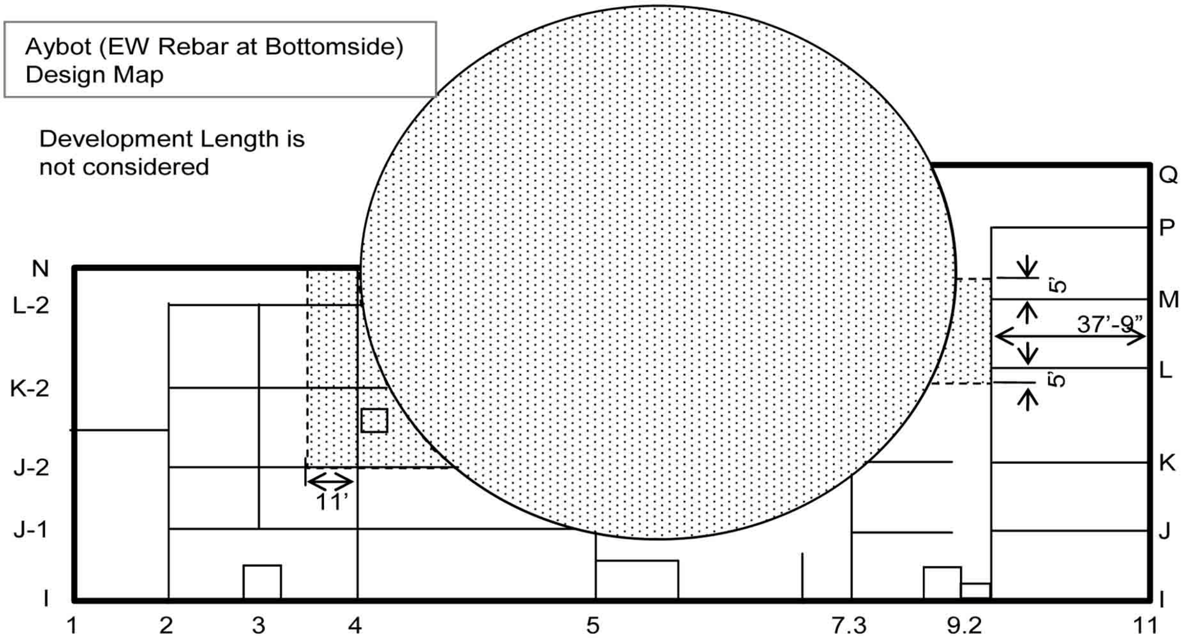


Figure 3.8.5-3 (Sheet 5 of 7)
Longitudinal Reinforcement,
Bottom Side of DISH and 6′ Basemat (NS)



Rebar arrangement	: #14@12" ; : #14@12"+#9@12" ; : #14@6"
Location	Layer 2

Figure 3.8.5-3 (Sheet 6 of 7)
Longitudinal Reinforcement,
Bottom Side of DISH and 6′ Basemat (EW)

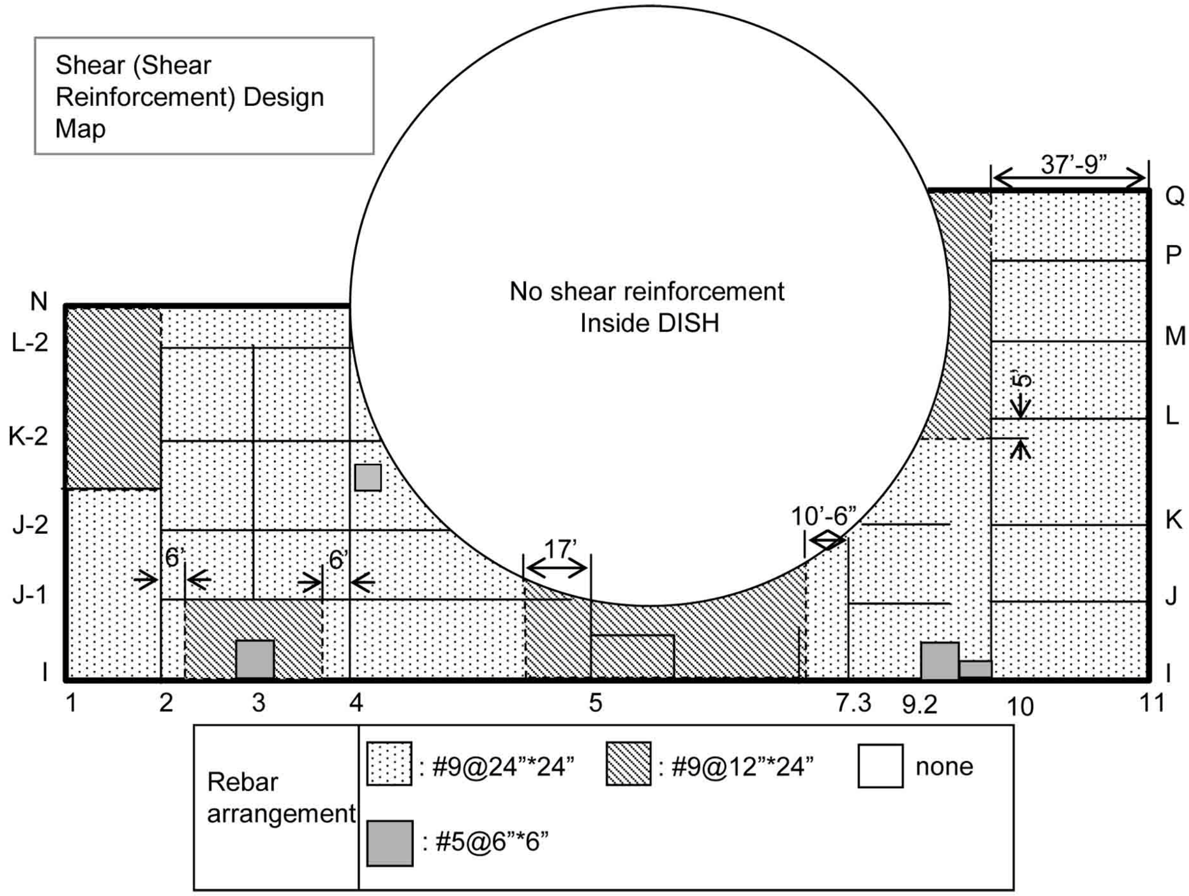


Figure 3.8.5-3 (Sheet 7 of 7)
Shear Reinforcement Map