3.8 Design of Category I Structures

This design of Category I structures section provides information on the Seismic Category I structures of the U.S. EPR, including the Reactor Containment Building (RCB), Reactor Building (RB) internal structures, other Seismic Category I structures, and their foundations. *[Figure 3B-1 illustrates key dimensions of the Nuclear Island (NI) Common Basemat Structure and other Seismic Category I structures.]**

The RB is located in the central portion of the NI Common Basemat Structure and houses the reactor coolant system (RCS). The RB consists of two concrete shell structures, which are the inner RCB and the outer Reactor Shield Building (RSB). The NI Common Basemat Structure foundation basemat supports both structures. The RCB houses the RB internal structures. These structures are Seismic Category I.

The RB is surrounded by Safeguard Buildings (SB) 1, 2, 3, and 4, and the Fuel Building (FB). The NI Common Basemat Structure foundation basemat supports each of these buildings, which are safety-related Seismic Category I structures. The main steam system and main feedwater system valve rooms are located within SBs 1 and 4. The vent stack is a safety-related Seismic Category I structure supported on the Fuel Building roof.

Two Emergency Power Generation Buildings (EPGB) are located separately from the NI Common Basemat Structure. Each EPGB contains two emergency diesel generators and supporting equipment, and has its own independent foundation basemat. These structures are Seismic Category I.

Four Essential Service Water Buildings (ESWB), which house Essential Service Water Cooling Towers (ESWCT) and Essential Service Water Pump Buildings (ESWPB) are located separately from the NI Common Basemat Structure and are supported by independent foundation basemats. These structures are Seismic Category I.

Seismic Category I building structures are analyzed and designed in accordance with Sections 3.7 and 3.8. The geometry of the buildings is provided in Appendix 3B, the regulatory compliance criteria are provided in Section 3.1 and additional loading definitions are provided in Section 3.3 through 3.6. The NI Common Basemat Structures, ESWB and EPGB, are reinforced concrete. As such, the effects of concrete cracking is addressed in the analysis by reducing the out-of-plane bending stiffness for walls, slabs, columns, and beams by 50 percent (except RCB). Analysis of the RCB, a post-tensioned structure, is based on full section properties. Analytical studies indicate that consideration of additional concrete cracking in in-plane bending and in-plane shear leads to approximately the same or lower seismic base shear demand and thus confirmed the validity of the approach for determining force and moment magnitudes as well as the distributions throughout the structure. The foundation mats of the buildings are modeled with full section properties.

Safety-related buried conduit, duct banks, pipes and pipe ducts are installed Seismic Category I to support and protect safety-related distribution systems outside of the NI Common Basemat Structure.

The dimensional arrangement drawings for Seismic Category I structures are provided in Appendix 3B.

3.8.1 Concrete Containment

The RCB is part of the RB system as illustrated in Figure 3B-1. The RCB controls the release of airborne radioactivity following postulated design basis accidents (DBA) and provides radiation shielding for the reactor core and the RCS. The RCB is a post-tensioned concrete pressure vessel and is located inside the reinforced concrete RSB described in Section 3.8.4. This section addresses the concrete elements of the RCB. Section 3.8.2 addresses steel sub-elements of the RCB (e.g., the equipment hatch and other penetrations). Section 6.2 describes the functional aspects of the containment system (e.g., heat removal, containment isolation, combustible gas control and leakage testing).

3.8.1.1 Description of the Containment

Figure 3.8-1—Reactor Building Plan at Elevation -50 Feet, Figure 3.8-2—Reactor Building Plan at Elevation -20 Feet, Figure 3.8-4—Reactor Building Plan at Elevation +5 Feet, Figure 3.8-5—Reactor Building Plan at Elevation +17 feet, Figure 3.8-6—Reactor Building Plan at Elevation +29 feet, Figure 3.8-7—Reactor Building Plan at Elevation +45 feet, Figure 3.8-8—Reactor Building Plan at Elevation +64 feet, Figure 3.8-9—Reactor Building Plan at Elevation +79 feet, Figure 3.8-10—Reactor Building Plan at Elevation +94 feet, Figure 3.8-11—Reactor Building Section A-A, and Figure 3.8-13—Reactor Building Section C-C show plan and section views of the RCB. See Sections 3.8.3 and 3.8.4 for additional figures showing structures adjacent to the RCB.

The RCB is located inside the reinforced concrete RSB. The RSB protects the containment structure from external hazards (e.g., wind loads, tornado and hurricane loads, aircraft hazard, explosion pressure wave and missiles). An annular space, designated as the RB annulus, is provided between the RCB and the RSB to prevent interaction of the two structures when subjected to extreme postulated design basis and beyond design basis loading conditions.

The RCB houses the RB internal structures. To prevent adverse interactions inside the RCB, the RB internal structures are physically independent of the RCB, except at the supporting foundation basemat. No structural connections are provided between the RCB and the RB internal structures. The RCB also provides structural support for the polar crane.

The NI Common Basemat Structure foundation basemat supports the RCB, and provides the only physical contact of the RCB with other structures. See Section 3.8.5 for a description of the NI Common Basemat Structure foundation basemat.

The RCB is a Seismic Category I, post-tensioned reinforced concrete shell structure consisting of an upright cylinder capped with a spherical dome. The dimensions of the RCB are approximately 162 feet outside diameter, by 153 feet inside diameter, by 218 feet high. The RCB is concentric with, and completely enclosed by, the RSB. No soil loadings are applied to the containment structure, and waterproofing materials are not required around the exterior surface of containment. A leak-tight steel liner plate covers the entire inner surface of the RCB, including the basemat (GDC 16).

*[The RCB is a concrete containment structure with a steel liner designed in accordance with the ASME Code, Section III, Division 2 (Reference 1)]** (GDC 16). The RCB accommodates the calculated pressure and temperature conditions resulting from a loss of coolant accident (LOCA) without exceeding the design leakage rate and with sufficient margin (GDC 50). The RCB is designed for an internal pressure of 62 psig and a maximum temperature of 309.2°F. The RCB is also designed for a negative internal pressure of -3 psig.

The equipment hatch and two airlocks provide access to the RB. A third opening provides access to the lower containment during construction. Section 3.8.2 provides a description of these sub-assemblies. The equipment hatch [ ] is located at [ ] and opens to the operating level of the RB internal structures. A personnel airlock is located at [ ] at the heavy load operating floor level and connects to a secure stair tower that serves various levels of the RCB. A construction access is located at [ ] An emergency airlock is located at [ ] and opens to the operating floor level from [ ]

The equipment hatch allows the entry of heavy components (e.g., the reactor pressure vessel, steam generators, reactor coolant pumps, and pressurizer) into the RB. The size of the hatch accommodates the entry of the reactor pressure vessel during construction and the entry of a replacement steam generator or pressurizer in one piece.

*[The steel liner plate is part of the concrete containment system and is designed in accordance with ASME Code, Section III, Division 2 (Reference 1).]** The liner plate serves as a leak-tight membrane to prevent the uncontrolled release of radioactive materials to the environment (GDC 16). The steel liner plate is approximately 0.25 inch thick.

RCB penetrations are described in Section 3.8.2.1.

3.8.1.1.1 Concrete Wall and Dome Shells and Connection to Foundation

The RCB wall is 4 feet, 3 inches thick, and the dome is 3 feet, 3 $^{3}/_{8}$ inches thick. The NI Common Basemat Structure foundation basemat supporting the containment structure is approximately 10 feet thick under the liner plate. Additional concrete is provided over the liner plate inside of containment to support the RB internal structures.

The wall and dome shells of the RCB are post-tensioned with hoop, vertical, and gamma tendons. Reinforcing steel bars are provided in the concrete containment walls and dome for crack control and strength to accommodate seismic and other loads.

Three buttresses run vertically and project outward from the outside surface of the cylindrical containment wall. These buttresses serve as the anchorage locations for terminating the horizontal hoop tendons. The anchorage surfaces of the buttresses are normal to the tangent line of the anchored hoop tendons. The buttresses are located at azimuths 0°, 112°, and 230°. Appendix 3E provides details of the design and reinforcement for the containment cylinder wall and buttresses.

A tendon gallery is provided under the circumference of the cylindrical containment wall below the NI Common Basemat Structure foundation basemat. This gallery provides access for installing and maintaining the lower terminations of the vertical wall tendons. Appendix 3E provides details of the design and reinforcement where the RCB wall intersects with the NI Common Basemat Structure foundation basemat.

A ring girder is provided around the top perimeter of the cylindrical containment wall where it transitions into the spherical dome. The ring girder is a thickened area of concrete that stiffens the containment vessel at the transition area. This serves as the termination point for the upper end of vertical tendons and the ends of the horizontal gamma tendons.

The concrete shell is thickened around the equipment hatch opening to provide a reinforced area where the concrete is removed for the opening. Appendix 3E provides details of the design and reinforcement in the equipment hatch area. Horizontal and vertical tendons are routed around penetrations through the containment wall. The two airlocks and the construction opening are located in the thickened buttresses.

Structural anchorages embedded in the containment wall support the polar crane. Structural members are welded to these embedments for supporting the polar crane rails.

3.8.1.1.2 Post-Tensioning System

Tendons are provided both horizontally and vertically in the cylindrical portion of the RCB. Tendons are provided in two orthogonal directions in the plan view of the containment dome. Layouts of the tendons vary to accommodate penetrations through the RCB wall.

The Freyssinet C-range post-tensioning system is the tendon system used for post-tensioning the concrete RCB. The Freyssinet 55C15 tendon system is made up of 55 seven-wire strands in each tendon. Section 3.8.1.6.3 describes the material properties of the tendon system. With the exception of the three greased test tendons of each type (vertical, gamma, and horizontal hoop) provided for force monitoring, the other tendons are grouted in place after tensioning.

A total of 119 horizontal hoop tendons are provided around the cylindrical shell of the RCB. The tendons terminate at the three vertical buttresses provided around the outside of the containment wall. Terminations alternate so that each buttress has a horizontal tendon terminating every third hoop (i.e., each hoop tendon extends the full circumference of the building).

A total of 47 vertical tendons are provided around the cylindrical shell of the RCB. The vertical tendons terminate at the top of the ring girder that is provided at the transition of the wall to the spherical dome roof. A total of 104 gamma tendons are also provided vertically up through the containment wall where they then wrap over the dome and terminate at the ring girder on the opposite side of the wall. The gamma tendons are separated into two groups that are placed 90° apart in the RBC dome. The bottom of both the vertical tendons and the gamma tendons terminate at the tendon gallery.

The U.S. EPR design is based on the use of Alternative B of RG 1.90, Revision 1 for monitoring deformations under pressure. Membrane compression will be maintained and the maximum stress in the tensile reinforcing will be limited to one-half the yield strength of the reinforcing steel (0.5fy), under the peak expected pressure for inservice inspection (ISI) tests.

Additional information on layout and design of the tendons is provided in Appendix 3E for the RCB cylindrical wall, and buttress areas. The minimum required post tensioning force to offset the structural integrity test (SIT) pressure loading is 801k/ft hoop force, 401k/ft vertical force, and 548k/ft in both directions for the dome.

Figure 3.8-18—Finite Element Model of Reactor Containment Building Tendon Layout in Cylindrical Wall and Figure 3.8-19—Finite Element Model of Reactor Containment Building Tendon Layout in Dome show the finite element model (FEM) of the tendon layout.

3.8.1.1.3 Liner Plate System

A carbon steel liner plate covers the entire inside surface of the RCB, excluding penetrations. The steel liner is 0.25 inch thick and is thickened locally around penetrations, large brackets, and at major attachments. Except for the bottom horizontal surface, angle and channel steel sections anchor the liner plate to the concrete containment structure. The in-containment refueling water storage tank (IRWST), including the containment sumps, are lined with 0.25 inch thick stainless steel liner plates that serve as additional corrosion protection for the underlying carbon steel liner. See Section 3.8.3 for a description of the IRWST.

Steel shapes reinforce the plate both longitudinally and laterally to provide rigidity during prefabrication, erection, and concrete placement. The steel shapes are welded to the liner plate and are fully embedded in the concrete to provide a rigid connection to the inside surface of the RCB concrete. The concrete foundation of the RB internal structures is poured on top of the liner plate at the basemat surface, embedding the lower region of the liner plate in the foundation. The liner plate is not used as a strength element to carry design basis loads; however, the liner supports the weight of wet concrete during the construction of the RCB.

*[Section CC-3810 of ASME Section III, Division 2 prescribes the criteria for design of liner anchorage system.]** The U.S. EPR liner anchorage system is designed using an energy approach described in BC-TOP-01, Revision 1 (Reference 68), which addresses ASME criteria. The methodology considers the variation in liner yield strength analytically by converting liner strain to stress and membrane forces assuming the plate remains elastic. In addition, the variation of liner plate thickness is accounted for by considering a thicker panel (+16 percent) with outward curvature being adjacent to a nominal plate with inward curvature (refer to Figure 2 through 4 of Reference 68). The inward curvature is evaluated as no more than 1/8 inch during fabrication and erection of the liner plate as given in Reference 68. The weld offset is mitigated through quality control in accordance with ASME Section III Division 2 CC-4523.2. The effects of concrete voids behind the liner are mitigated by the construction method employed. Lower concrete modulus is mitigated due to the code required over strength and the extensive performance testing required of the concrete mix. The variation of anchorage spacing is mitigated by quality control during the fabrication process. The anchorage system is designed with a safety factor so that the local crushing of the concrete is limited and a means of stress redistribution to obtain a maximum load capacity. The structural discontinuities areas, such as pipe penetration and openings, are designed as special regions.

Section 3.8.2 contains a description of the penetrations through the containment liner, including the equipment hatch, airlocks, piping penetration sleeves, electrical penetration sleeves, and the fuel transfer tube penetration sleeve.

No load transfer attachments are used at the bottom portion of the liner plate to transfer loads from the concrete RB internal structures into the lower portion of the NI Common Basemat Structure foundation basemat. RB internal structure lateral reaction loads are transferred through the liner plate. This is achieved by lateral bearing on the haunch wall at the bottom of the RB internal structures foundation where it is embedded in concrete above the NI Common Basemat Structure foundation basemat.

Structural attachments to the containment walls and dome include various pipe, HVAC, electrical, and equipment support brackets, as well as the polar crane rail supports. The liner plate is continuously welded to embedded plate areas and areas with thickened plates so that a continuous leak-tight barrier is maintained.

3.8.1.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used in the design, fabrication, construction, testing, and in-service inspection of the RCB (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50).

3.8.1.2.1 Codes and Standards

- ACI 117-90/117R-90, Specification for Tolerances for Concrete Construction and Materials (Reference 6).
- ACI 301-05, Specifications for Structural Concrete for Buildings (Reference 7).
- ACI 304R-00, Guide for Measuring, Mixing, Transporting, and Placing Concrete (Reference 8).
- ACI 305.1-06, Specification for Hot-Weather Concreting (Reference 9).
- ACI 306.1-90, Standard Specification for Cold-Weather Concreting (Reference 10).
- ACI 347-04, Guide to Form Work for Concrete (Reference 11).
- ACI 349-01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (exception described in Sections 3.8.4.4 and 3.8.4.5) (Reference 12).
- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of Condition A strength reduction factors even when supplemental reinforcement is provided (Reference 63).
- ACI SP-2 (99), Manual of Concrete Inspection (Reference 13).

- ANSI/AWS D1.4-2005, Structural Welding Code - Reinforcing Steel (Reference 19).
- ASME Code.
  - Section II - Material Specifications.
  - Section III, Division 2 - Code for Concrete Reactor Vessels and Containments (Reference 1).
  - Section V - Nondestructive Examination.
  - Section VIII - Pressure Vessels.
  - Section IX - Welding and Brazing Qualifications.
  - Section XI – Rules for Inservice Inspection of Nuclear Power Plant Components.
- Acceptable ASME Code cases per RG 1.84, Revision 33, August 2005.
- ASME NOG-1-04, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder) (Reference 21).

3.8.1.2.2 Specifications

Industry standards (e.g., those published by the ASTM) are used to specify material properties, testing procedures, fabrication, and construction methods. Section 3.8.1.6 lists the applicable standards used.

Structural specifications cover the areas related to the design of the RCB. These specifications emphasize the important points of the industry standards for the RCB and reduce the options that would otherwise be permitted by the industry standards. These specifications cover the following areas:

- Concrete material properties.
- Mixing, placing, and curing of concrete.
- Reinforcing steel and splices.
- Post-tensioning system.
- Liner plate system.

3.8.1.2.3 Design Criteria

The design of pressure retaining components of the RCB complies with:

- Article CC-2000 of the ASME Code, Section III, Division 2.

- Article CC-3000 of the ASME Code, Section III, Division 2 (GDC 1, GDC 2, and GDC 16).
- ASME Code, Section XI, Subsection IWL, Requirements for Class CC Concrete Components of Light-Water Cooled Plants.
- ASME Code, Section XI, Subsection IWE, Requirements for Class MC and Metallic Liners of Class CC Concrete Components of Light-Water Cooled Power Plants.

3.8.1.2.4 Regulations

- 10 CFR 50 – Licensing of Production and Utilization Facilities.
- 10 CFR 50, Appendix A – General Design Criteria for Nuclear Power Plants (GDC 1, 2, 4, 16, and 50).
- 10 CFR 50, Appendix J – Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors.
- 10 CFR 100 – Reactor Site Criteria.

3.8.1.2.5 NRC Regulatory Guides

Regulatory Guides applicable to the design and construction of the RCB:

- RG 1.7, Revision 3.
- RG1.35.1, July 1990.
- RG 1.84, Revision 33.
- RG 1.90, Revision 1.
- RG 1.94, Revision 1.
- RG 1.107, Revision 1.
- RG 1.136, Revision 3 (exception described in 3.8.1.3).
- RG 1.199, November 2003 (exception described in 3.8.1.4).
- RG 1.216, August 2010.

3.8.1.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the expected loads over a broad range of site conditions. *[Loads and load combinations for the RCB are in accordance with the requirements of Article CC-3000 of the ASME Code, Section III, Division 2,]** Code for Concrete Containments and ACI Standard 359, and RG 1.136 (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). RG 1.136 endorses the 2001 Edition of the ASME Code with the 2003 addenda (including exceptions taken in RG 1.136). The U.S. EPR standard plant design is based on the 2004 Edition of the Code, inclusive of the exceptions taken in RG 1.136. Design loads and loading combinations for the concrete RCB are described in Sections 3.8.1.3.1 and 3.8.1.3.2.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard plant design envelope for the RCB, or perform additional analyses to verify structural adequacy.

3.8.1.3.1 Design Loads

The concrete RCB is designed for the following loads:

Service Loads

- Normal Loads – Normal loads are those loads encountered during normal plant operation and shutdown (GDC 4). This load category includes:
  - Dead Loads (D) – Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.
  - Live Loads (L) – Live loads include any normal loads that vary with intensity or point of application, including moveable equipment. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, varied from zero to full value, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied as appropriate for the loading condition.
  - Soil Loads or Lateral Earth Pressure (H) – There are no soil or lateral earth pressure loads on the RCB because it is surrounded by other Seismic Category I structures that shield it from these loads.
  - Hydrostatic Loads (F) – Hydrostatic loads due to water stored in pools and tanks are considered in the design of RB internal structures that exert reaction loads on the RCB and NI Common Basemat Structure foundation basemat. Hydrodynamic loads resulting from seismic excitation of fluids are included as a component of the safe shutdown earthquake (SSE) load. There are no hydrostatic loads from groundwater or external floods on the RCB because it is surrounded by other Seismic Category I structures that subsequently provide a shield. Buoyancy loads are addressed in Section 3.8.5 for foundation design.
  - Thermal Loads (T_o) – Thermal loads consist of thermally induced forces and moments resulting from normal plant operation and environmental conditions. Thermal loads and their effect are based on the critical transient or steady-state condition. Thermal expansion loads due to axial restraint, as well as loads resulting from thermal gradients, are considered.

The ambient air temperatures listed below are for normal operation. Normal operation temperatures are given as a maximum value during summer and a minimum value during winter.

RB internal ambient temperatures:

- During normal operation:
  Equipment Area: 131°F (maximum), 59°F (minimum).
  Service Area: 86°F (maximum), 59°F (minimum).

- During normal shutdown: 86°F (maximum), 59°F (minimum).

RB annulus internal ambient temperatures:

- During normal operation: 113°F (maximum), 45°F (minimum).

– Pipe Reactions (R_o) – Pipe reactions are those loads applied by piping system supports during normal operating or shutdown conditions based on the critical transient or steady state conditions. The dead weight of the piping and its contents are not included. Appropriate dynamic load factors are used when applying transient loads, such as water hammers.

– Post-Tension Loads (J) – Post-tension loads are those loads developed from applying strain on the containment tendons.

– Relief Valve Loads (G) – Relief valve loads are those loads resulting from the actuation of a relief valve or other high-energy device.

– Pressure Variant Loads (P_v) – Pressure variant loads are those external pressure loads resulting from pressure variation either from inside or outside of containment.

– Construction Loads – Construction loads are those loads to which the structure may be subjected during construction of the plant. Construction loads will be applied to evaluate partially completed structures, temporary structures, and their respective individual members. *[Design load requirements during construction for buildings and other structures will be developed in accordance with Table CC-3230-1 of the Section III, Division 2, of the ASME Code]** and with SEI/ASCE 37-02. The magnitude and location of construction loads will be applied to generate the maximum load effects of dead, live, construction, environmental, and lateral earth pressure loads. Consideration will be given to the loads and load effects of construction methods, equipment operation, and sequence of construction.

– Test Loads – Test loads are those loads that are applied during structural integrity testing or leak-rate testing. This load category includes:

- Test Pressure Loads (P_t) – Test pressure loads are those loads resulting from the pressure exerted on the RCB during the SIT at 1.15 times the design pressure and during the leak-rate test at 1.0 times the DBA pressure.

- Test Thermal Loads (T_t) – Test thermal loads include thermal effects and loads experienced by the RCB during the structural integrity and leak-rate tests.

Factored Loads

- Severe Environmental Loads – Severe environmental loads are those loads that could be encountered infrequently during the life of the plant (GDC 2). This load category includes:
  - Wind Loads (W) – There are no wind loads applicable on the RCB because it is surrounded by other Seismic Category I structures that subsequently provide a shield.
  - There are no operating basis earthquake (OBE) loads applicable to the overall RCB design for the U.S. EPR because an OBE level of one-third the SSE has been selected. See Section 3.7.1 for a description of the OBE.
- Extreme Environmental Loads – Extreme environmental loads are those that are credible but are highly improbable (GDC 2). This load category includes:
  - SSE (E') – SSE loads are those loads generated by an earthquake with a peak horizontal ground acceleration of 0.30g. Seismic loads in the vertical direction and two orthogonal horizontal directions are considered to act simultaneous. Section 3.7 provides a description of how SSE loads are determined and combined. SSE loads are considered due to applied inertia loads, including dead loads, live loads, and hydrodynamic loads (i.e., water in storage pools and tanks).
  - Tornado and Hurricane Loads (W_t) – Loads generated by the design basis tornado and hurricane are described in Section 3.3 and Section 3.5. This load category includes:
    - Tornado and Hurricane Wind Pressure (W_w) – Tornado and Hurricane wind pressure is not applicable because the RCB is protected from wind forces by the RSB.
    - Tornado and Hurricane Created Differential Pressure (W_p) – The RSB is designed as an enclosed, unvented structure, which does not allow tornado differential pressure forces to affect the RCB (W_p) = 0 for hurricane.
    - Tornado and Hurricane Generated Missiles (W_m) – Tornado and Hurricane-generated missile loads are not applicable because the RSB serves as a barrier to protect the RCB from missile strikes.
- Abnormal Loads – Abnormal loads are those loads generated by a postulated high-energy pipe break accident. This event is classified as a DBA (GDC 4 and GDC 50). These loadings include an appropriate dynamic load factor to account for the dynamic nature of the load, unless a time-history analysis is performed to justify otherwise. Abnormal loads include the following loads:

– Internal Flooding Loads (F_a) – Loads resulting from the internal flooding of containment during or following a postulated DBA.

– Buoyant Force (F_b) – Fluid forces acting vertically on a partially or fully submerged body as a result of the design basis maximum flood. Section 3.8.5 describes application of buoyant force loads to the NI Common Basemat Structure foundation basemat.

– Pressure Load (P_a) – Pressure equivalent static load within or across a compartment or building generated by a postulated pipe break.

– Thermal Load (T_a) – Thermal loads generated by the postulated pipe break (including thermal load T_o).

– Accident Pipe Reactions (R_a) – Pipe reactions generated by the postulated pipe break (including pipe reaction load R_o).

– Pipe Break Loads (R_r) – Local loads following a postulated pipe break. Unless a time-history analysis is performed to justify otherwise, these loadings include an appropriate dynamic load factor to account for the dynamic nature of the load. This load category includes:

  - Pipe Break Reaction Loads (R_{rr}) – R_{rr} is defined as the equivalent static load on the structure generated by the reaction of the high-energy pipe during the postulated break.

  - Pipe Break Jet Impingement Loads (R_{rj}) – R_{rj} is defined as the jet impingement equivalent static load on the structure generated by the postulated break.

  - Pipe Break Missile Impact Loads (R_{rm}) – R_{rm} is defined as the missile impact equivalent static load on the structure generated by or during the postulated break, such as pipe whipping.

Other Loads

Other loads refer to postulated events or conditions that are not included in the design basis (GDC 4). These loading conditions and effects are evaluated without regard to the bounding conditions under which SSC are required to perform design basis functions. This load category includes:

- Aircraft Hazard (A) – Aircraft hazard refers to loads on a structure resulting from the impact of an aircraft. The evaluation of this loading condition is considered as part of the plant safeguards and security measures. Aircraft hazard loads are not applicable on the RCB because it is surrounded by other Seismic Category I structures that provide a shield.

- Explosion Pressure Wave (B) – Explosion pressure wave refers to loads on a structure resulting from an explosion in the vicinity of the structure. The evaluation of this loading condition is considered as part of the plant safeguard and security measures. Explosion pressure wave loads are not applicable on the RCB because it is surrounded by other Seismic Category I structures that provide a shield.

- Combustible Gas (C) – Combustible gas loads are pressure loads that result from a fuel-clad metal-water reaction followed by an uncontrolled hydrogen burn during a post-accident condition in a reactor containment (Refer to Section 6.2.5).

Missile Loads other Than Hurricane- or Tornado-Generated Missiles

There are no missile loads on the RCB resulting from activities of nearby military installations, turbine failures, or other causes. The RCB is surrounded by other Seismic Category I structures that shield it from missiles.

3.8.1.3.2 Design Load Combinations

Loading combinations used for the design of the RCB, including its steel liner plate, are in accordance with guidance provided in NUREG-0800, Standard Review Plan, Section 3.8.1 (Reference 3) (GDC1, GDC 2, GDC 4, GDC 16, and GDC 50).

The NI Common Basemat Structure is a monolithic concrete structure. However, various portions of the structure have different classifications (i.e., RCB, RB internal structures, and other Seismic Category I structures) and correspondingly different design requirements, as shown in Figure 3.8-118. In some instances, the load combinations identified in NUREG-0800 do not include certain independent loadings which should be considered to account for potential structure-to-structure effects (i.e., the effect on one structure resulting from loadings applied to a separate, but monolithically connected, structure). To account for potential structure-to-structure effects, the NUREG-0800 loading combinations are adjusted by including the necessary additional independent loadings. The independent loadings added to the load combinations include hydrostatic load (F), buoyant force (F_b), and soil load/lateral earth pressure (H). The load factors for hydrostatic load (F) and buoyant force (F_b) are matched to that of the dead load (D) for each loading combination, while the load factor for soil load/lateral earth pressure (H) is matched to that of the live load (L). Section 3.8.1.3.1 provides details regarding all loads considered for the design of the RCB.

The following guidance is used for applying load combinations for the design of the RCB:

- The live load (L) is applicable after construction of containment. Construction loadings, temporary or otherwise, may also be considered as live loads and included within appropriate loading combinations.

- Unless a time-history analysis is performed to justify otherwise, the maximum values of load combinations including the loads P_a, T_a, R_a, R_{rr}, R_{rj}, R_{rm}, or G are used, including an appropriate dynamic load factor.

- For concrete members, U_S is defined as the required section strength for service loads based on the allowable stresses defined in Subarticle CC-3430 of the ASME Code, Section III, Division 2, with additional guidance provided by NUREG-0800.

- For concrete members, U_F is defined as the required section strength for factored loads based on the allowable stresses defined in Subarticle CC-3420 of the ASME Code, Section III, Division 2, with additional guidance provided by NUREG-0800.

- The following requirements are met for the design of concrete components for factored load conditions:

  – Primary forces must not bring the local section to a general yield state with respect to any component of section membrane strain or section flexural curvature. General yield state is the point beyond which additional section deformation occurs without an increase in section forces.

  – Under combined primary and secondary forces on a section, the development of a general yield state with respect to those membrane strains or flexural curvatures that correspond to secondary stress components is acceptable, and is subject to rebar strain limits specified in Subarticle CC 3420 of the ASME Code, Section III, Division 2. The concept of a general yield state is not applicable to strains associated with radial shear stress.

- Primary and secondary forces are as defined in Subarticle CC-3130 of the ASME Code, Section III, Division 2.

- Limitations on maximum concrete temperatures as defined in Subarticle CC-3440 of the ASME Code, Section III, Division 2 are observed.

- Loads and loading combinations encompass the soil cases described in Section 3.7.1, using the design criteria described in Section 3.7.1 and Section 3.7.2.

The following load combinations define the design limits for the Seismic Category I concrete RCB. These load combinations define the design limits for the Seismic Category I steel liner plate for the RCB, except that load factors are considered to be 1.0.

- Service load combinations (test loads).

  $U_S = D + L + H + F + F_b + J + P_t + T_t$

- Service load combinations (construction loads).

  $U_S = D + L + H + F + F_b + T_o + J + W$

- Service load combinations (normal loads).

  $U_S = D + L + H + F + F_b + T_o + R_o + J + G + P_v$

- Factored load combinations (severe environmental loads).

  $U_F = D + 1.3L + 1.3H + F + F_b + T_o + R_o + J + G + P_v + 1.5W$

- Factored load combinations (extreme environmental loads).

  $U_F = D + L + H + F + F_b + T_o + R_o + J + G + P_v + E'$

  $U_F = D + L + H + F + F_b + T_o + R_o + J + G + P_v + W_t$

- Factored load combinations (abnormal loads).

  $U_F = D + L + H + F + F_b + J + G + 1.5P_a + T_a + R_a$

  $U_F = D + L + H + F + F_b + J + G + P_a + T_a + 1.25R_a$

  $U_F = D + L + H + F + F_b + J + 1.25G + 1.25P_a + T_a + R_a$

- Factored load combinations (abnormal or severe environmental loads).

  $U_F = D + L + H + F + F_b + J + G + 1.25W + 1.25P_a + T_a + R_a$

  $U_F = D + L + H + F + F_b + T_o + J + G + F_a + W$

- Factored load combinations (abnormal or extreme environmental loads).

  $U_F = D + L + H + F + F_b + J + G + E' + P_a + T_a + R_a + R_r$

  $U_F = D + J + P_g1 + P_g2$

3.8.1.4 Design and Analysis Procedures

*[The analysis and design of the post-tensioned RCB comply with the requirements of Article CC-3300 of the ASME Code, Section III, Division 2]** and RG 1.136 (GDC 1 and GDC 16).

Computer programs perform many of the computations required for the RCB analysis and design. In many cases, classical methods and manual techniques are also used for the analysis of localized areas of the containment structure and its subassemblies. Manual calculations are generally used for:

- Initial proportioning of the dome, wall, and base slab and determining tendon layout.

- Evaluation of the effects of locally applied loads, such as crane loads and pipe reaction loads.
- Preparation of input for the computer analyses.
- Design of the liner plate and its anchorage to the concrete containment shell.

The analysis and design methods incorporate several phases. Overall analysis and design are performed for structures using computer models of the NI Common Basemat Structure, Seismic Category I structures. Then, localized design evaluations account for local loadings and discontinuities in structures (e.g., openings and local changes in member cross-sections). Results from the local analyses are combined with the overall global analysis results to produce the final design.

An ultimate capacity analysis is performed, as described in Section 3.8.1.4.11, to determine the ultimate internal pressure load capability of the containment for use in probabilistic risk assessment and severe accident analyses. The ultimate capacity analysis evaluates the concrete containment structure (including the liner plate), as well as large containment penetrations, such as the equipment hatch and airlocks.

Combustible gas loads are pressure loads that result from a fuel-clad metal-water reaction followed by an uncontrolled hydrogen burn during a post-accident condition in a reactor containment (Section 6.2.5). Combustible gas loads are evaluated per the requirements of RG 1.216 and RG 1.136. RG 1.136, Regulatory Position C.5 provides the loads and load combinations acceptable for analysis and design of containment when exposed to the loading conditions associated with combustible gas. The principal combustible gas for the U.S. EPR is hydrogen. The U.S. EPR design does not include an inerting gas system. Containment maximum pressure is 75 psig based on pressure load time histories due to the hydrogen released by assuming 100 percent fuel-clad reaction with reactor coolant followed by hydrogen burning. RG 1.136, Regulatory Position C.5 and RG 1.7 specify a minimum pressure of 45 psig combined with dead load (D) as a minimum design condition. U.S. EPR calculated maximum pressure is greater than the regulatory required minimum pressure. ANSYS computer code was used to perform a structural analysis of the RCB to calculate maximum liner strain. The elastic model of containment described in Section 3.8.3.4.1 is employed. The elements associated with the liner plate, containment wall, ring girder, dome, foundation, and RBIS foundation are isolated from the overall static model. Additionally, a nonlinear model created from a six- degree slice of the RCB liner, wall, ring girder, and dome, which implements axisymmetric boundary conditions, is also analyzed. This nonlinear model allows for concrete cracking and the tensile capability of the reinforcing bars. A separate analysis is performed to determine the effects of the pressure load on containment penetrations. These analyses consider dead loads, pre-stressing loads, and the internal pressure load from the hydrogen burn event, and considered degradation of material properties due to the higher temperature resulting from hydrogen burn. RCB liner strains calculated for the pressure time histories during this hydrogen burn are within strain limits described by RG 1.7 and ASME Code Section III, Division 2, Subarticle CC-3720.

Gaps are provided between the RCB and adjoining interior and exterior structures to accommodate deformation during pressurization and as a result of seismic movements.

Appendix 3E provides details of the design and reinforcement for the containment wall to foundation connection.

Appendix 3E provides details of the design and reinforcement for the containment cylinder wall and buttresses.

The following sections provide details of design and analysis of the RCB.

3.8.1.4.1 Computer Programs

The containment structure is included in an overall model developed for analysis of the NI Common Basemat Structure, which includes the RCB with the RB internal structures, the RSB, the SBs, the FB, and the NI Common Basemat Structure foundation basemat. The RCB is modeled and analyzed using the ANSYS computer program. ANSYS is a validated and verified, quality-controlled computer program that has been used for a number of years in the nuclear power industry. Refer to Chapter 17 for a description of the quality assurance program for the U.S. EPR design certification.

The ANSYS model is used to analyze the RCB for the loads defined in Section 3.8.1.3.1. The results from these load case analyses are combined and factored using the loading combinations defined in Section 3.8.1.3.2. The design of the RCB shell wall and dome is generally controlled by load combinations containing the +62/-3 psig design internal pressure load and SSE seismic loads.

The overall NI Common Basemat Structure analysis is performed using the ANSYS finite element computer program. The RCB is modeled in combination with the other structures of the NI Common Basemat Structure and basemat using a mesh of finite elements. The element mesh for the RCB consists of the dome and cylindrical shell wall, which interconnects with the overall NI Common Basemat Structure foundation basemat. No other structures physically connect to the containment structure; therefore, the foundation basemat is the only interfacing structure in the model. Section 3.8.5 describes the modeling of the NI Common Basemat Structure foundation basemat.

ANSYS SOLID45 solid elements are primarily used to model the RCB concrete dome and cylindrical shell wall. SOLID45 is a three-dimensional, eight-node brick element that is suitable for moderately thick shell structures. It can also provide out-of-plane shear forces and has an elastic-plastic capability. Four and five layers of SOLID45 elements are used to model through the thickness of the dome and cylindrical shell wall, respectively. ANSYS SOLID95, a twenty node brick element, and ANSYS SOLID92, a ten node tetrahedral element, are also used to model the RCB. The buttresses, ring girder, and thickened areas around the base of the containment structure are included in the ANSYS model. Soft elements are used to represent the large openings for the equipment hatch, two airlocks, and construction opening.

Post-tensioning tendon forces are included in the RCB structural analysis. Forces for each post-tensioning tendon in the RCB shell and dome are calculated along their routing lengths for the appropriate nodal locations in the RCB model. This is accomplished by using a second ANSYS model developed for this task. The tendons are modeled using ANSYS LINK8 type elements, which are two-node three-dimensional truss type elements. These elements follow the routings used in the structure and are given the material properties of the tendons. Forces are applied to these links by imposing strains along the lengths of the modeled tendons, tensioning losses are explicitly included in these calculations. The calculated reactions forces from the tendon model are then applied as forces to the RCB model. ANSYS BEAM44 elements, which are two-node three dimensional beam type elements, are used to model the end anchorages for the tendons.

The steel liner causes discontinuity between the NI foundation basemat and the base of the RCB interior structure. This transition is modeled using multi-point constraints to allow sub-modeling of the interior structure and interface, as needed. For static modeling considerations, the individual companion nodes are coupled together. The strength of the liner is not relied upon to carry structural loadings.

The FEM used for analysis of the RCB is shown in Figure 3.8-14—Finite Element Model of Reactor Containment Building, Figure 3.8-15—Finite Element Model of Reactor Containment Building Dome Concrete, Figure 3.8-16—Finite Element Model of Reactor Containment Building Basemat Concrete, Figure 3.8-17—Finite Element Model of Reactor Containment Building Interface with Concrete Interior Structures, Figure 3.8-18, and Figure 3.8-19. The FEM is based on the layout and dimensions shown in the figures listed in Section 3.8.1.1.

Additional descriptions of the RCB computer model are provided in Appendix 3E.

3.8.1.4.2 Assumptions on Boundary Conditions

The RCB is modeled integral with the NI Common Basemat Structure foundation basemat in the overall ANSYS model. Section 3.8.5 provides information on the design of the NI Common Basemat Structure foundation basemat and interface conditions between the soil and foundation. Soil spring parameters are described in Section 3.8.5 and the soil conditions are described in Section 2.5.

3.8.1.4.3 Axisymmetric and Nonaxisymmetric Loads

The RCB is modeled in its entirety as a three-dimensional structure. The loads described in Section 3.8.1.3.1 are applied in the locations and directions appropriate for each load. Overall pressure is applied uniformly to the interior surface of the containment structure. Pressure variant loads potentially present in the annulus are applied uniformly to the exterior surface of the structure.

Localized loads, such as penetration dead loads, hydrostatic pool water loads, live loads, and pipe rupture loads are applied to specific portions of the structural model as appropriate. Post-tension loads are applied to each tendon in its specific location. Seismic loads are applied in each possible direction and combination for the two horizontal and one vertical load directions using the methodology described in Section 3.7.

3.8.1.4.4 Transient and Localized Loads

Thermal and pressure loads resulting from a LOCA are applied to the RCB model as a non-linear load condition. The LOCA temperature peaks rapidly at the surface of the internal liner plate and builds up over time through the thickness of the concrete containment vessel. Accident pressure and temperature curves used in the analysis are presented in Figure 3.8-20—Accident Temperature versus Time (Reactor Containment Building) and Figure 3.8-21—Accident Pressure versus Time (Reactor Containment Building).

A heat transfer analysis was performed for the RCB accident temperature using the ANSYS computer code. Temperature gradients through the wall and dome were calculated with respect to time using the curve, and annulus temperature of 79°F (26°C) and the thermal properties in Table 3.8-1—Thermal Properties for Heat Transfer Analysis-Reactor Containment Building.

Structural forces were computed, with time, based on the heat transfer analysis using the ANSYS computer code. Figure 3.8-22—Temperature Gradient Through Cylinder Wall, Figure 3.8-23—Temperature Gradient Through Dome, and Figure 3.8-24—Temperature Gradient Through Basemat provide the generic results of this analysis. These results and those of the accident pressure analysis were reviewed in detail to establish critical time points for the development of load cases to be used in the structural analysis. Forces and moments at times 0 second, 1.39 hours, 24 hours and 100 hours were selected as critical for cylinder, dome, and basemat forces and moments. Additional internal pressure was added to the RCB due to the heating of the liner plate.

The RCB, including the steel liner, is designed to resist the effects of impulse loads and dynamic effects. Structural members designed to resist impulse loads and dynamic effects in the abnormal, extreme environmental, and abnormal and extreme environmental categories are allowed to exceed yield strain and displacement values. The allowable stresses applicable to the determination of section strength are as specified in Subsections CC-3400 and CC-3700 of the ASME Code, Section III, Division 2. In determining tensile yield strength of reinforcing steel (i.e., f_y) the dynamic effect of the loading may be considered. The applicable design assumptions in Subsection CC-3930 of the ASME Code, Section III, Division 2 are used in calculating the effects of impact or impulse.

The ductility limits used in design for impact load do not exceed two-thirds the ductility determined at failure. The ductility limits used in design for impulse load do not exceed one-third the ductility determined at failure. See Section 3.8.5 for a description of additional requirements for missile barrier design and ductility requirements applicable to the design of the RCB.

3.8.1.4.5 Creep, Shrinkage, and Cracking of Concrete

Conservative values of concrete creep and shrinkage are used in the design of the RCB. Moments, forces, and shears are obtained on the basis of uncracked section properties in the static analysis. However, in sizing the reinforcing steel required, the concrete is not relied upon for resisting tension. Thermal moments are modified by mesh refinement and cracked-section analysis using analytical techniques. The ANSYS computer code and the RCB model thermal stress evaluation, based on results from the heat transfer analysis, were used to evaluate cracking due to accident thermal loading. The material properties, specifically E (Young's modulus), for the finite elements, were redefined as bilinear. This approximation allows the moment of inertia of a wall section to reduce in proportion to the amount of cracking developed due to the thermal loading. The threshold tensile value for cracking, maximum tension in the concrete, is taken as $4\sqrt{f'_c}$. Elements are not allowed to heal once cracked. Results from this analysis are used to factor the thermal moments from the RCB static analysis for the design of concrete sections.

Section 3.8.1.6.1 describes methods used to confirm that concrete properties satisfy design requirements.

3.8.1.4.6 Dynamic Soil Pressure

Soil loads are not applicable to the design of the RCB because the building is completely surrounded by other structures above the NI Common Basemat Structure foundation basemat.

3.8.1.4.7 Tangential Shear

The design and analysis procedures for tangential shear are in accordance with the ASME Code Section III, Division 2 and RG 1.136.

Tangential shear is resisted by the vertical reinforcement and the horizontal hoop reinforcement in the RCB wall.

3.8.1.4.8 Variation in Physical Material Properties

In the design and analysis of the RCB, consideration is given to the effects of possible variations in the physical properties of materials on the analytical results. The properties used for analysis purposes were established based on past engineering experience with similar construction and materials. Values used are delineated in Table 3.8-2—Material Properties – Reactor Containment Building, Table 3.8-3—Tendon Frictional Losses, and Table 3.8-4—Thermal Properties – Reactor Containment Building. Additional reviews of materials and their effects on the analysis and design of the RCB will be included in design specification development and materials selection.

Losses due to elastic shortening, concrete creep and shrinkage, and relaxation of the post-tensioning cables were accounted for in the analysis. Table 3.8-5—Tendon Losses and Effective Forces with Time summarizes the losses and delineates the final wire stresses.

When designing the structure under full service and factored load conditions, allowable stress levels are used based on the minimum strength of the concrete and reinforcing materials used in construction of the containment to account for variations in physical properties. The containment is designed for the range of soil properties described in Section 3.7.1.

3.8.1.4.9 Penetrations

Large penetrations through the concrete RCB include the equipment hatch, two airlocks, and a construction opening, which are described in Section 3.8.1.1. The two airlocks are located in the containment buttresses, with one positioned at azimuth 0° and one positioned at azimuth 230°. The construction opening, which is a temporary opening permanently sealed using a metal pressure closure cap after construction, is also located at azimuth 230°. The equipment hatch is located in the cylindrical shell portion of containment at azimuth 150° between the buttress locations. The containment shell is thickened in the region surrounding the equipment hatch.

Submodels with refined element meshes and tendon configurations are used to analyze the containment vessel in the areas around the equipment hatch and in the buttress at azimuth 230° that contains the penetrations for an airlock and the construction opening. Displacements and loadings obtained from the full containment model are applied to the equipment hatch and buttress at azimuth 230° submodel to more accurately represent results in the regions around the large openings for the various loading conditions. The modulus of elasticity of the solid elements at the openings in the full containment model is reduced to one percent to consider the effect of the openings; however, the openings are explicitly included in the submodel. The modification of material properties at those solid elements was done based on the satisfactory match of displacement and stress contours between the full containment model and the equipment hatch and buttress sub-models.

Small penetration openings through the concrete RCB are defined as those having a diameter of less than approximately 6 feet. These are not considered to have a specific effect on the overall design of the RCB and are not included in the overall computer model of containment.

Appendix 3E provides details of the design and reinforcement in the equipment hatch area.

Section 3.8.2 provides design details of the steel portion of containment penetrations.

3.8.1.4.10 Steel Liner Plate and Anchors

*[The design of the steel liner plate is in accordance with Subarticle CC-3600 of the ASME Code, Section III, Division 2.]** The steel liner plate is not considered as a structural strength member when performing containment design basis analyses. The steel liner plate is designed to withstand the effects of imposed loads and to accommodate deformation of the concrete containment without jeopardizing leak-tight integrity (GDC 16). The steel liner plate is anchored to the concrete containment in a manner that does not preclude local flexural deformation between anchor points. Calculated strains and stresses for the steel liner plate do not exceed the values given in Table CC-3720-1 of the ASME Code, Section III, Division 2. Strains associated with construction-related liner deformations may be excluded when calculating liner strains for service and factored load combinations as allowed by the code. The liner is anchored to the concrete containment around the outside perimeter of the sides of the embedded portion between elevation -25 feet, 7 inches and elevation -7 feet, 6.5 inches. Anchors are not provided on the inside surface of the liner. Overturning moments and sliding forces of the RB internal structures relative to the liner plate are resisted by the appropriate structural dead weight and lateral bearing.

The steel liner plate anchorage system is designed to accommodate design loads and deformations without loss of structural or leak-tight integrity (GDC 16). The steel liner plate anchorage system is designed so that a progressive failure of the anchorage system is prevented in the event of a defective or missing anchor. The steel liner plate is anchored to the concrete so that the liner strains do not exceed the strain allowable given in Paragraph CC-3720 of the ASME Code, Section III, Division 2. The anchor size and spacing is designed so that the response of the steel liner plate is predictable for applicable loads and load combinations. The anchorage system is designed to accommodate the design in-plane shear loads and deformations exerted by the steel liner plate and normal loads applied to the liner surface. The allowable force and displacement capacity for the steel liner plate anchors does not exceed the values given in Table CC-3730-1 of the ASME Code, Section III, Division 2. The load combinations specified in Section 3.8.1.3.2 are applicable to the steel liner plate anchors. Mechanical and displacement-limited loads are as defined in Subparagraph CC-3730(a) of the ASME Code, Section III, Division 2. *[Concrete anchors are designed in accordance with ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1),]** and with the guidelines of RG 1.199. The use of Appendix D to ACI 349-06 is an exception to RG 1.199, which endorses Appendix B to ACI 349-01 for concrete anchorage design. Use of Appendix D to ACI 349-06 (with exception stated in Section 3.8.1.2.1) is acceptable as it results in an equivalent or conservative anchorage design when compared to that of Appendix B to ACI 349-01.

Steel liner plate penetration assemblies, including nozzles, reinforcing plates, and penetration anchors are designed to accommodate design loads and deformations without loss of structural or leak-tight integrity (GDC 16). Effects such as temperature, concrete creep, and shrinkage are considered. Temporary and permanent brackets and attachments to the steel liner plate are designed to resist the design loads without loss of the liner integrity due to excessive deformation or load from the brackets or attachments.

Design of the steel liner plate and anchorage system is based on minimum strengths for the materials that are specified for fabrication of the steel components and their interface with the concrete containment. Deviations in the geometry of the liner plate due to fabrication and erection tolerances are considered in the design.

The materials of the liner and its stiffening and anchorage components that are exposed to the internal environment of containment are selected, designed, and detailed to withstand the effects of imposed loads and thermal conditions during design basis conditions.

3.8.1.4.11 Containment Ultimate Capacity

The Ultimate Pressure Capacity Deterministic Analyses for the RCB is performed in accordance with RG 1.136, RG 1.216 and guidance provided in SRP 3.8.1.II.4.K (Rev. 2)

Analysis results for the various containment elements are summarized in Table 3.8-6. These results are based on ANSYS non-linear finite element containment model with nominal stress-strain elasto-plastic materials properties under accident temperature and with cracked concrete section behavior.

The Ultimate Nominal Pressure Capacities for the cylinder and dome sections are calculated using the two degree slice FEM with simulated axisymmetric boundary conditions. The ultimate conditions in these cases are 0.8 percent strain level in tendon areas located away from discontinuities (according to SRP 3.8.1.II.4.K). The simplified cross-checking hand calculation confirms the FEM results.

The Ultimate Nominal Pressure Capacities for the ring and gusset sections are evaluated using the same FEM as above with non-linear analysis run until the first 0.8 percent strain level in the rebars in the critical sections.

Non-Linear 3D FEM is used for the hatch Ultimate Nominal Pressure Capacities evaluation. The non-linear steel properties for hatch, flanges, and sleeves are based on elastic-perfectly plastic model with bilinear kinematic hardening according to Von Mises yield criteria. Geometric nonlinearity is accounted for in the large displacement (stability) calculation. The results of calculations are summarized in Table 3.8-6.

The equipment hatch is a spherical shell. The stability analysis is performed in accordance with NE-3133.4. The allowable pressure for buckling is 85.67 psig. In accordance with NE-3222, the compressive allowable stress is increased by 150 percent for ASME Service Level D, which gives an ultimate capacity buckling pressure of 128.5 psig.

Since the hatch performs a leak tightness role, the allowable strain criteria in accordance with ASME Code, Section III, Div. 2, Subsection CC, Article CC-3720 is conservatively used for the hatch ultimate pressure capacity evaluation. These allowable strains are: membrane strain of ε_C=0.5%, ε_T=0.3% and combined membrane + bending strain of ε_C=1.4%, ε_T=1%.

The estimated Ultimate Pressure Capacities are determined from the principal strain levels, which approach ultimate in the protruding sleeves while remaining below yield in the hatch and flange areas. Under ultimate internal pressure that exceeds 2.0 times the design pressure, the sealing strip between the clamps remains in compression and remains leak tight. The radial ribs on the sleeve serve as buckling stiffeners for the hatch sleeve and are designed to carry axial force that exceeds 2.5 times the design pressure. The hatch cover and protruding sleeve buckle at greater than 2.0 times the design pressure.

An ultimate pressure capacity evaluation has been performed for the other major containment penetrations including the construction opening closure, the containment dedicated spare penetration, the personnel airlocks, the fuel transfer tube, and the main steam and feedwater line penetrations.

The ultimate capacity is evaluated using the design basis accident temperature and the following criteria.

1. Structural Capacity- A pressure 2.5 times the containment design pressure (2.5 x 62 psig = 155 psig) is applied to the penetration. The resulting strain levels are compared against the ASME Subsection CC factored strain allowable values in

Table CC-3720-1. The 2.5 times design pressure is considered adequate to demonstrate sufficient margin exists above the design pressure for the ultimate capacity evaluation.

2. Stability (or buckling) - A stability analysis is performed to determine the buckling pressure in accordance with ASME Subsection NE, paragraph NE-3222, where one-third of the basic compressive allowable stress is considered or the buckling pressure is determined in accordance with NE-3133. ASME Level D allowable buckling pressures are determined. Strain values are determined from application of the allowable buckling pressure in an analysis with non-linear material properties and evaluated against the ASME Subsection CC factored strain allowable values in Table CC-3720-1.

3. Potential Leak Paths - The sealing mechanisms and strain levels in the metallic components at the ultimate capacity pressure are evaluated to demonstrate that no containment leak paths are created.

The minimum ratio of the ultimate capacity pressure (Pu) to the design pressure (Pd) and the controlling mode/location is presented in Table 3.8-6.

Construction Opening Closure

The structural capacity of the construction opening closure is determined by finite element analysis techniques. The construction opening closure is a spherical shell. The stability analysis is performed in accordance with NE-3133.4. The allowable pressure for buckling is 79 psig. The compressive allowable stress is increased by 150 percent for Service Level D. Therefore, the ultimate capacity buckling pressure is 118.5 psig.

The construction opening closure is a welded cap. The calculated strain values do not exceed the factored allowable strain values in ASME Table CC-3720-1. Therefore, the leaktight integrity of the penetration is maintained at the evaluated pressures.

Containment Dedicated Spare Penetration

The capacity of the containment dedicated spare penetration sleeve is bounded by the main steam line penetration. The penetration closure capacity is bound by the construction opening closure as described in Section 3.8.2.4.1. Therefore, the ultimate capacity of the containment dedicated spare penetration does not govern the ultimate capacity of the U.S. EPR containment.

Personnel Airlocks

The structural capacity of the personnel airlocks is determined by finite element analysis techniques. The personnel airlocks consist of a complex geometry. The stability analysis is performed by a rigorous analysis in accordance with NE-3222.1(a)(1).

The basic allowable pressure for buckling is controlled by the capacity of the airlock door and is 79.6 psig. The compressive allowable stress is increased by 150 percent for Service Level D. Therefore, the ultimate capacity buckling pressure determined is 119.4 psig.

The airlock leak tight integrity is maintained by limiting the strains of the metallic parts to less than the factored allowable strain values in ASME Table CC-3720-1. The airlock seals are positive seating with the containment internal pressure. The airlock seals remain compressed with the strain limits considered for the metal components in the vicinity of the airlock door seals. Therefore, the leak tight integrity of the penetration is maintained at the containment ultimate capacity pressures.

Fuel Transfer Tube

The structural capacity of the fuel transfer tube is determined by finite element analysis techniques. The stability analysis of the fuel transfer tube is performed by a rigorous analysis in accordance with NE-3222.1(a)(1) and Code Case N-284-1. A non-linear finite element analysis is performed by incrementally applying pressure until the solution no longer converges. The allowable pressure for buckling is 230 psig, which is greater than 2.5 x Pd (155 psig). Therefore, the ultimate capacity results are reported at 2.5 Pd (155 psig).

The fuel transfer tube leak tight integrity is maintained by limiting the strains of the metallic parts to less than the factored allowable strain values in ASME Table CC-3720-1. The fuel transfer tube has a blind flange on the containment side which has positive seating with the containment internal pressure. The fuel transfer tube flange remains seated with the strain limits considered for the metal components in the vicinity of the blind flange. Therefore, the leak tight integrity of the penetration is maintained at the containment ultimate capacity pressures.

Main Steam and Feedwater Line Penetrations

The structural capacity of the main steam and feedwater line penetrations is determined by finite element analysis techniques. Buckling is not a failure mechanism for the main steam and feedwater line penetrations because the penetrations act as short columns with a slenderness ratio (kl/r) less than 89 (structural steel).

The main steam and feedwater line penetrations leak tight integrity is maintained by limiting the strains of the metallic parts to less than the factored allowable strain values in ASME Table CC-3720-1. Therefore, the leak tight integrity of the penetration is maintained at the containment ultimate capacity pressure.

3.8.1.4.12 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.4, 2.5, 3.3, 3.5, 3.7, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections. A cross-reference between U.S. EPR FSAR sections and information required by SRP Section 3.8.4, Appendix C is provided in Table 3.8-17.

3.8.1.5 Structural Acceptance Criteria

*[The limits for RCB allowable stresses, strains, deformations and other design criteria are in accordance with the requirements of Subsection CC-3400 of the ASME Code, Section III, Division 2,]** RG 1.136, and RG 1.216 (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). This applies to the overall containment vessel and subassemblies and appurtenances that serve a pressure retaining function, except as noted in Section 3.8.2. Specifically, allowable concrete stresses for factored loadings are in accordance with Subsection CC-3420 and those for service loads are in accordance with Subsection CC-3430.

The limits for stresses and strains in the liner plate and its anchorage components are in accordance with ASME Code, Section III, Division 2, Tables CC-3720-1 and CC-3730-1.

*[Limits for allowable loads on concrete embedments and anchors are in accordance with Appendix D of ACI-349-06 (with exceptions stated in Section 3.8.1.2.1, Codes)]** and guidance given in RG 1.199 (with exception described in Section 3.8.1.4.10).

Section 3.8.1.6 describes minimum requirements for concrete, reinforcing, post-tensioning tendons, and the liner plate system for the RCB.

A SIT is performed as described in Section 3.8.1.7.1.

The RCB is stamped to signify compliance with the ASME Code Section III, Division 2.

An as-built report is prepared to summarize deviations from the approved design and confirm that the as-built RCB is capable of withstanding the design basis loads described in Section 3.8.1.3 without loss of structural integrity or safety-related functions.

3.8.1.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control program, and special construction techniques used in the fabrication and construction of the RCB. Materials and quality control satisfy the following requirements (GDC 1):

- *[ASME Code, Section III, Division 2, Code for Concrete Containments/ACI Standard 359, Articles CC-2000, CC-4000, CC-5000, CC-6000, and CC-9000.]**
- RG 1.107, Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures, Revision 1, February 1977.
- RG 1.136, Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments, Revision 3, March 2007.

Concrete and reinforcement forming and placement tolerance not specifically addressed in these references are in accordance with ACI 349-01 and ACI 117-90.

3.8.1.6.1 Concrete Materials

Concrete Mix Design

*[The concrete mix design for the RCB conforms to the requirements specified in Subarticle CC-2230 of the ASME Code, Section III, Division 2.]**

Structural concrete used in the construction of the RCB shell wall and dome has a minimum compressive strength (i.e., f'_c) of 7000 psi at 90 days.

Concrete mix design is determined based on field testing of trial mixtures with actual materials used. Testing evaluates:

- Ultimate concrete strength, as well as early strength in support of an aggressive construction schedule.
- Creep and shrinkage characteristics.
- Concrete workability and consistency.
- Required concrete admixtures.
- Heat of hydration and required temperature control for large or thick concrete pours.
- Special exposure requirements when identified on design drawings.
- Thermal properties, diffusivity and a conductivity per CRD C36 (Reference 69) and CRD C44 (Reference 70), respectively.

Cement

Cement used for the concrete RCB conforms to the requirements of ASTM C150 (Reference 47) (Type I, Type II, Type IV or Type V) or ASTM C595 (Reference 48) (Type IP, Type IP [MS], or Type IP [MH]).

Low-alkali cement, as defined in ASTM C150, is used in concrete with aggregates that are potentially reactive per ASTM C33.

Aggregates

Aggregates used for the RCB meet the requirements specified in ASME Code, Section III, Division 2, Paragraph CC-2222.

Aggregates conform to the requirements of ASTM C33 (Reference 22).

ASTM Standards C1260 and C1293 (References 71 and 72) shall be used in testing aggregates for potential alkali-silica reactivity (ASR).

Admixtures

Air-entraining admixtures conform to the requirements of ASTM C260 (Reference 23).

Chemical admixtures conform to the requirements of ASTM C494 (Reference 24) or ASTM C1017 (Reference 25).

Fly ash and other pozzolanic admixtures conform to the requirements of ASTM C618 (Reference 26).

Grout fluidizers conform to the requirements of ASTM C937 (Reference 27).

Ground-granulated blast furnace slag used as an admixture is in accordance with the requirements of ASTM C989 (Reference 28).

Silica fume used as an admixture conforms to the requirements of ASTM C1240 (Reference 29).

Admixtures used in concrete mixtures in accordance with ASTM C845 (Reference 30) expansive cement is compatible with the cement and produce no deleterious effects.

Mix Water

Mix water used for the RCB is in accordance with the requirements of ASME Code, Section III, Division 2, Paragraph CC-2223.

Placement

Conveying, inspection, placement, and testing of concrete are performed in accordance with the following codes and standards:

- ACI 301-05, Specifications for Structural Concrete for Buildings.

- ACI 304R-00, Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete.
- ACI 305.1-06, Specification for Hot-Weather Concreting.
- ACI 306.1-90, Standard Specification for Cold-Weather Concreting.
- ACI 347-04, Recommended Practice for Concrete Formwork.
- ACI SP-2 (99), Manual of Concrete Inspection.
- ASTM C94, Specification for Ready-Mixed Concrete (Reference 38).

3.8.1.6.2 Reinforcing Steel and Splice Materials

Materials

Conventional reinforcing is used in the concrete RCB, which conforms to ASTM A615 (Reference 31) or ASTM A706 (Reference 32), and the criteria described in the ASME Code, Section III, Division 2, Subarticle CC-2330.

[Welded splices and mechanical splices of reinforcing bars are used. Mechanical splices are threaded, swaged, or sleeved with ferrous filler metal. These devices are qualified and the qualifications are maintained in accordance with Subarticle CC-4333 of ASME Code.] [These devices also meet the provisions of ACI 349-01, Section 12.14.3.]**

Welding of reinforcement is as specified in approved splice details and is located as shown on approved reinforcing placement drawings. *[Welding conforms to ASME Code, Section III, Division 2, Subsection CC, as supplemented by RG 1.136, and ANSI/AWS D1.4.]**

Materials used for bar-to-bar sleeves for mechanical cadweld-type rebar splices in the RCB conform to ASTM A513, (Reference 33) ASTM A519 (Reference 34), or ASTM A576 (Reference 35). For bar splice sleeves attached to the liner plate or structural steel shapes, the sleeves are carbon steel in accordance with ASTM A513, ASTM A519, or ASTM A576 (Grades 1008 through 1030).

*[Materials for mechanical threaded, swaged, or sleeved splicing systems are established in accordance with the ASME Code, Section III, Division 2, Subarticle CB-4333.]**

Fabrication and Placement

*[Fabrication and placement of reinforcing bars for the RCB are in accordance with Subarticle CC-4300 of the ASME Code, Section III, Division 2.]**

3.8.1.6.3 Tendon System Materials

Tendons

The post-tensioning tendon system consists of load-carrying and non-load-carrying components. The load-carrying components include the post-tensioning wires that make up the tendons, and anchorage components composed of bearing plates, anchor heads, wedges, and shims. Non-load-carrying components include the tendon sheathing (including sheaths, conduits, trumpet assemblies, couplers, vent and drain nipples, and other appurtenances) and corrosion prevention materials.

*[Materials used for the RCB post-tensioning system (including post-tensioning steel, anchorage components, and non-load-carrying and accessory components) meet the requirements of Subarticle CC-2400 of the ASME Code, Section III, Division 2.]**

The Freyssinet C-range post-tensioning system has the following properties:

- ASTM A416 (Reference 36), Grade 270, low-relaxation tendon material.
- Tendon ultimate strength $F_{pu} = 270$ ksi
- Tendon minimum yield strength $F_{py} = (0.9)(270) = 243$ ksi
- Modulus of elasticity of tendon material $E_{ps} = 28{,}000$ ksi
- Number of strands per tendon $N_{strands} = 55$
- Total area of each tendon $A_p = 12.76$ in^2

The materials used for the anchorage components are compatible with the tendon system. Tendon raceways consist of corrugated steel ducts and rigid metal conduit. These components are non-structural and are sealed to prevent the intrusion of concrete during construction.

Grouting of Tendons

*[Cement grout for the grouted tendons in the prestressing system in the RCB is selected based on the testing and material requirements of the ASME Code, Section III, Division 2, as amended by RG 1.136, which endorses the Regulatory Positions of RG 1.107, Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures.]**

Greasing of Tendons

*[Grease for the greased test tendons in the prestressing system in the RCB is selected based on the testing and material requirements of the ASME Code, Section III, Division 2.]**

3.8.1.6.4 Liner Plate System and Penetration Sleeve Materials

The 0.25 inch thick liner plate is SA-516, Grade 55, 60, 65 or 70 material, which conforms to Subarticle CC-2500 of the ASME Code, Section III, Division 2 (GDC 16). Thickened liner plates are used at penetrations, brackets, and embedded assemblies.

Penetration assemblies and appurtenances that are either not backed by concrete or are embedded in concrete and surrounded by a compressible material to provide local flexibility conform to the material requirements of Subsection NE of the ASME Code, Section III, Division 1 (Reference 73) (GDC 16). Penetration sleeve materials are listed in Table 6.1-1.

Welding materials conform to the requirements of ASME Code, Section II. Welding activities meet the requirements of ASME Code, Sections III and IX.

Materials used for the carbon steel liner plate, carbon steel and low alloy steel attachments, and appurtenances subject to ASME Code Division 2 requirements, meet the fracture toughness requirements of Subsection CC-2520 of the ASME Code, Section III, Division 2.

*[Materials used in ASME Division 1 attachments and appurtenances meet the fracture toughness requirements of Subsection 2300 of the ASME Code, Section III, Division 1.]**

3.8.1.6.5 Steel Embedments

*[Steel embedment materials conform to the requirements of Subsection CC-2000 of the ASME Code, Section III, Division 2.]**

3.8.1.6.6 Corrosion Retarding Compounds

Corrosion retarding compounds used for the RCB are described in Section 6.1.2.

3.8.1.6.7 Quality Control

In addition to the quality control measures addressed in Section 3.8.1.6, refer to Chapter 17 for a description of the quality assurance program for the U.S. EPR (GDC 1).

3.8.1.6.8 Special Construction Techniques

Special techniques are not used for construction of the RCB. Modular construction methods are used to the extent practical for prefabricating portions of the containment liner, equipment hatch, airlocks, penetrations, reinforcing steel, tendon conduits, and concrete formwork. Such methods have been used extensively in the construction industry. Rigging is pre-engineered for heavy lifts of modular sections. Permanent and temporary stiffeners are used on liner plate sections to satisfy code requirements for structural integrity of the modular sections during rigging operations.

3.8.1.7 Testing and Inservice Inspection Requirements

3.8.1.7.1 Structural Integrity Test

Following construction, the RCB is proof-tested at 115 percent of the design pressure. During this test, deflection measurements and concrete crack inspections are made to confirm that the actual structural response is within the limits predicted by the design analyses (GDC 1).

*[The SIT procedure complies with the requirements for prototype containments of Article CC-6000 of the ASME Code, Section III, Division 2,]** (Reference 1) and with Subsections IWL and IWE of Section XI of the ASME Code.

3.8.1.7.2 Long-Term Surveillance

The RCB is monitored periodically throughout its service life in accordance with 10 CFR 50.55a and 10 CFR 50, Appendix J, to evaluate the integrity of containment over time (GDC 1 and GDC 16). As part of this monitoring program, containment deformations and exterior surface conditions are determined while the building is pressurized. Initial measurements and in-service inspection meet the requirements of the following:

- ASME Code, Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components, Subsections IWE and IWL.
- Supplemental Inspection Requirements of 10 CFR 50.55a.
- ASME Code, Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components, Subsection IWL, does not contain specifications for inservice inspection of grouted tendons. For inservice inspection of grouted tendons, the guidelines of RG 1.90, Revision 1 are followed, with no exceptions.

The U.S. EPR containment differs in some aspects from the "reference containment" as defined in RG 1.90, Revision 1. The U.S. EPR containment ISI program will be developed using the concepts presented in RG 1.90, Revision 1. In accordance with RG 1.90, Revision 1, the tendons for the U.S. EPR will be included in an ISI program. The program will consist of:

- Force monitoring of ungrouted test tendons (supplemented by RG 1.35.1).
- Monitoring of deformations under pressure at prescribed locations (Alternative B of RG 1.90, Revision 1).

- Visual inspection of exposed structurally critical areas of the containment and containment prestressing system.

- A sample of sheathing filler grease from each of the ungrouted test tendons will be taken and analyzed according to the test methods and acceptance criteria of ASME Code Table IWL-2525-1.

  In addition, the amount of sheathing filler grease removed and replaced will be compared to assess grease leakage within the structure.

  A reportable condition exists if: (1) the acceptance limits in Table IWL-2525-1 are not met; (2) the amount of grease replaced exceeds five percent of the net duct volume injected at original installation pressure; (3) grease leakage is detected during general visual examination of the containment exterior surface; or (4) there is a presence of free water. Reportable conditions are reported to the NRC in accordance with Technical Specification 5.6.6, Pre-Stressed Concrete Containment Tendon Surveillance Report.

Three greased tendons of each type (vertical, gamma and horizontal hoop) will be provided for force monitoring. The test tendons are included in the number of tendons required by design and will be subjected to force measurement by lift-off or load cells to assess the effects of concrete shrinkage and creep and relaxation of the tendon steel. The nine greased tendons form the sample size for load cell or lift-off testing,

In accordance with the Alternative B of RG 1.90, Revision 1, the points to be instrumented for measurement of radial displacements under pressure will be located in six horizontal planes in the cylindrical portion of the shell with a minimum of four points in each plane.

The points to be instrumented for measurement of vertical (or radial) displacements under pressure will be located at the top of the cylinder relative to the base, at a minimum of four approximately equally spaced azimuths. Locations will also be selected at the apex of the dome and one intermediate point between the apex and the springline on at least three equally spaced azimuths.

The visual inspections will be performed of representative areas at structural discontinuities, areas around large penetrations or a cluster of small penetrations, and other areas where heavy loads are transferred to the containment structure. Visual inspection of these selected areas will be completed during the pressure tests while the containment is at maximum test pressure. Also included will be samples of the exposed portions of the tendon anchorage assembly hardware. The tendon anchorage assemblies utilized for the greased tendons will be representative of the grouted tendons except that provisions will be provided to allow force measurement by lift-off or load cells. The sample size of tendon anchorage assemblies will comply with the requirements of RG 1.90, Revision 1.

The pressure test schedule is part of the ISI program and is provided in Table 3.8-7—ISI Schedule for the U.S. EPR.

Section 6.2.6 contains a description of the associated leak-rate test procedure, Containment Integrated Leakage Rate Test (CILRT). Containment pressure testing will occur in conjunction with the CILRT.

Sufficient physical access is provided in the annulus between the RCB and the RSB to perform inservice inspections on the outside of the containment. There is approximately 18 inches clearance between the upper containment ring beam and the RSB. Space is available inside of the RCB to perform inservice inspections of the liner plate. Gaps are provided between the liner and RB internal structures concrete structural elements, which provide space necessary to inspect the liner at wall and floor locations inside containment. Inservice inspection of the embedded portion of the containment liner and the surface of the concrete containment structure covered by the liner are exempted in accordance with Section III of the ASME Code for Class CC components.

3.8.2 Steel Containment

The steel containment section describes major RCB penetrations and portions of penetrations not backed by structural concrete that are intended to resist pressure. Section 3.8.1 describes the concrete RCB.

3.8.2.1 Description of the Containment

Steel items that are part of the RCB pressure boundary and are not backed by concrete include the equipment hatch, airlocks, construction opening, piping penetration sleeves, electrical penetration sleeves, and fuel transfer tube penetration sleeve. Section 3.8.1.1 describes RCB steel items that are backed by concrete, such as the liner plate.

3.8.2.1.1 Equipment Hatch, Dedicated Spare Penetration, Airlocks, and Construction Opening

The equipment hatch, illustrated in Figure 3.8-25 is a welded steel assembly with a double-sealed, flanged, and bolted cover. The cover for the equipment hatch attaches to the hatch sleeve from inside of the RCB. The cover seats against the sealing surface of the penetration sleeve mating flange when subjected to internal pressure inside the RCB. The RCB penetration sleeve and the RSB penetration sleeve are connected by an expansion joint to allow for differential movement between the two walls, as shown in Figure 3.8-25. The equipment hatch opens into the Seismic Category I FB, which provides protection of the hatch from external environmental hazards (e.g., high wind, tornado and hurricane winds and missiles, and other site proximity hazards, including aircraft hazards and blasts). The equipment hatch sleeve has an inside diameter of approximately 27 feet, 3 inches.

The containment penetrations also include a 36-inch diameter spare containment penetration as shown in Figure 3.8-119. This penetration is dedicated for post-accident conditions as described in Section 19.2.3.3.8.

One personnel airlock and one emergency airlock are provided for personnel to access the RCB. Figure 3.8-26—Personnel Airlock, Emergency Airlock General Overview illustrates a typical arrangement for the airlocks. Each airlock is a welded steel assembly that has two doors, each with double seals. The airlocks open into containment so that internal pressure inside the RCB seats the doors against their sealing surfaces. The personnel airlock and emergency airlock are connected to the RSB wall by expansion joints to allow for differential movement.

The doors mechanically interlock so that one door can not be opened unless the second door is sealed during plant operation. Provisions are made for deliberately overriding the interlocks by the use of special tools and procedures for ease of access during plant maintenance. Each door is equipped with valves for equalizing the pressure across the doors. The doors are not operable unless the pressure is equalized. Pressure equalization is possible from the locations at which the associated door can be operated. The valves for the two doors interlock so that only one valve can open at a time and only when the opposite door is closed and sealed. Each door is designed to withstand and seal against design and testing pressures of the containment vessel when the other door is open. A visual indication outside each door shows whether the opposite door is open or closed. In the event that one door is accidentally left open, provisions outside each door allow remote closing and latching of the opposite door.

The personnel airlock at [ ] opens into a [ ] which is a Seismic Category I structure. The emergency airlock opens into the [ ], which is a Seismic Category I structure. Therefore, both airlocks are protected from external environmental hazards (e.g., high wind, tornado and hurricane winds and missiles, and other site proximity hazards, including aircraft hazards and blasts). The personnel airlock and the emergency airlock have inside diameters of approximately 10 feet, 2 inches.

The construction opening is located at [ ] and opens to the heavy load operating floor level from [ ] This passage serves as personnel and material access into the RB during construction. The construction opening has an outside diameter of approximately 9 feet, 6 inches. Upon completion of construction work, the cavity in the RCB is permanently sealed with a metal closure cap welded to an embedded sleeve. The construction opening is shown in Figure 3.8-123.

*[The equipment hatch, dedicated spare penetration, two airlocks, and construction opening closure cap and sleeve are designated as Class MC components in compliance with Article NE-3000 of the ASME Code, Section III, Division I, and are stamped pressure vessels designed and tested in accordance with this code]** (GDC 1 and GDC 16).

3.8.2.1.2 Piping Penetration Sleeves

Piping penetrations through the RCB pressure boundary are divided into the following three general groups:

- High-energy penetrations:

  This type of penetration is used for high-energy piping. Examples of high-energy penetrations are those provided for the safety injection or chemical and volume control lines. High-energy piping penetrations consist of the following major steel items:

  - *[Process pipe – Process pipes are welded or seamless and are made of carbon or stainless steel. The pipes are welded to a connecting part centrally located in the annulus between the inner containment wall and the outer shield wall. The connecting part is welded to an embedded sleeve in the inner containment wall. This acts as an anchor for the penetration. The guard pipe is also connected to the connecting part. The process pipes conform to the requirements of ASME Code Section III, Subsection NC and meet the requirements of the piping system they serve as described in Section 3.6.*

  - *Connecting part – Connecting parts are made from forged carbon or stainless steel and conform to ASME Code Section III, Division 1, Subsection NC. The connecting process pipes and connecting part are each designed and analyzed to be capable of carrying loads in the event of failure of the process pipes as described in Sections 3.6 and 3.9.*

  - *Pipe sleeve – Pipe sleeves are made from carbon or stainless steel and consist of the portion of the penetration that projects into the RCB and supports the connecting part. Pipe sleeves conform to ASME Code Section III, Division 1, Subsection NE (GDC 1).]**

- Main steam and feedwater penetrations:

  These penetrations are a special adaptation of the high-energy penetrations. The design is the same as the high-energy penetration except it has a guard pipe that fits tightly over the process pipe in the inner containment sleeve that is designed to dissipate heat and prevent the concrete from overheating. The protection pipes are connected to the RSB penetration sleeve by expansion bellows, as shown in Figures 3.8-120 and 3.8-27. The bellows allow differential movement and minimizes load transfer between the RCB and RSB.

- Standard piping penetration:

This penetration type is used for moderate or low energy piping lines. The basic configuration consists of an inline flued head component attached to the inner containment embedded pipe sleeve. There is no guard pipe, but an expansion joint attached to the pipe and sleeve allows differential movement and minimizes load transfer between the RCB and RSB. These penetrations consist of:

– Process pipe and flued head – Process pipes are welded or seamless and are made of carbon or stainless steel. The pipes are welded to the flued head. Flued heads are made from forged carbon or stainless steel. *[Process pipes and flued heads conform to Subsection NC of the ASME Code, Section III, Division 1, and]** meet the requirements of the piping system they serve as described in Section 3.6.

– Pipe Sleeve – Pipe sleeves are made from carbon or stainless steel and consist of the portion of the penetration that projects into the RCB and supports the flued head. *[Pipe sleeves conform to ASME Code Section III, Division 1, Subsection NE]** (GDC 16).

- Spare penetrations:

  Spare penetrations are reserved for future use. Spare penetrations consist of the following major items:

– Solid closure plate or pipe cap – *[Closure plates and pipe caps are made from carbon or stainless-steel and conform to the requirements of Subsection NC of the ASME Code, Section III, Division 1, Subsection NC.]**

– Pipe sleeve – *[Pipe sleeves are made from carbon or stainless-steel and consist of the portion of the penetration that projects into the RCB. Pipe sleeves conform to ASME Code Section III, Division 1, Subsection NE]** (GDC 16).

Typical details of piping penetrations are illustrated in Figure 3.8-27—Containment Penetration for Feedwater Pipe, Figure 3.8-28—Containment Penetrations for High Energy Pipes, Figure 3.8-29—Containment Standard Piping Penetrations – Single Pipe, Figure 3.8-30—Containment Standard Piping Penetrations – Multiple Pipes, and Figure 3.8-120—Containment Penetration for Main Steam Pipe.

3.8.2.1.3 Electrical Penetration Sleeves

Sleeves for electrical penetrations consist of the portion of penetrations that projects into the RCB and supports the electrical assembly. *[Sleeves conform to ASME Code Section III, Division 1, Subsection NE]** (GDC 16).

Typical details of electric penetrations are illustrated in Figure 3.8-121—Low Voltage Electrical Penetration Sleeve and in Figure 3.8-122—Medium Voltage Electrical Penetration Sleeve.

3.8.2.1.4 Fuel Transfer Tube Penetration Sleeve

The fuel transfer tube penetration is provided to transfer fuel between the refueling canal and the spent fuel pool during the refueling operations of the reactor. The penetration consists of an approximately 20 inch diameter stainless steel pipe installed inside a larger 36 inch diameter penetration sleeve that is anchored to the concrete RCB. *[The penetration sleeve conforms to Subsection NE of the ASME Code, Section III, Division 1]** (GDC 16). The inner pipe acts as the transfer tube. Expansion joints are provided around the fuel transfer tube where it passes through the RB internal structures refueling canal concrete and the RSB and FB concrete to allow for differential movement between the structures and to maintain leak-tight boundaries for the refueling pools and the annulus ventilation system. Figure 3.8-31—Fuel Transfer Tube Penetration (Conceptual View) illustrates the fuel transfer tube penetration.

3.8.2.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used in the design, fabrication, construction, testing, and inservice inspection of steel portions of the RCB that are intended to resist pressure, but are not backed by structural concrete (GDC 1, GDC 2, GDC 4, GDC 16 and GDC 50).

The boundaries between the RCB and the steel pressure boundary component consist of those defined in ASME Code, Section III, Division I, Paragraph NE-1132. Section 3.8.1.2 describes codes, standards, and specifications applicable to the containment steel liner.

3.8.2.2.1 Codes and Standards

- ANSI/AISC N690-1994 (R2004), Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2.
- ANSI/AWS D1.1-2000, Structural Welding Code – Steel.
- ANSI/AWS D1.6-1999, Structural Welding Code – Stainless Steel.
- ASME Code:
  - Section II – Material Specifications.
  - Section III, Division 1 – Nuclear Power Plant Components (Reference 73).
  - Section V – Nondestructive Examination.
  - Section VIII – Pressure Vessels.

- Section IX – Welding and Brazing Qualifications.

- Acceptable ASME Code cases per RG 1.84.

3.8.2.2.2 Specifications

Industry standards such as those published by ASTM are used to define material properties, testing procedures, fabrication, and construction methods. *[The applicable ASTM standard specifications for materials are those permitted by Article NE-2000 of Section III, Division 1 of the ASME Code. Applicable ASTM standard specifications for nondestructive methods of examination are those referenced in Appendix X and Article X-3000 of Section III, Division 1 of the ASME Code.]**

Structural specifications cover the design of steel portions of the containment pressure boundary. These specifications cover the following areas:

- Equipment hatch, airlocks, and construction opening closure cap and sleeve.
- Piping penetration sleeves.
- Electrical penetration sleeves.
- Fuel transfer tube penetration sleeve.

3.8.2.2.3 Design Criteria

The design of steel pressure retaining components of the RCB that are not backed by concrete complies with the following:

- Article NE-3000 of the ASME Code, Section III, Division 1 (Reference 73) (GDC 1 and GDC 16).

3.8.2.2.4 Regulations

- 10 CFR 50, Licensing of Production and Utilization Facilities.
- 10 CFR 50, Appendix A – General Design Criteria for Nuclear Power Plants GDC 1, 2, 4, 16, and 50.
- 10 CFR 50, Appendix J – Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors.

3.8.2.2.5 NRC Regulatory Guides

RGs applicable to the design and construction of steel portions of the RCB that resist pressure, but are not backed by structural concrete:

- RG 1.7, Revision 3.

- RG 1.57, Revision 1.
- RG 1.84, Revision 33.
- RG 1.136, Revision 3 (exception described in 3.8.1.3).
- RG 1.193, Revision 1.
- RG 1.216, August 2010.

3.8.2.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the expected loads over a broad range of site conditions. Design loads and loading combinations for steel portions of the RCB that are not backed by concrete are described in the following sections (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). Section 3.8.1.3 addresses loads and loading combinations for design of the steel liner plate.

3.8.2.3.1 Design Loads

Steel portions of the RCB that are not backed by concrete are designed for the following loads:

The effects of missiles and external events such as hurricanes, tornados, aircraft hazards, and explosion pressure waves are not considered because the containment is protected from these effects by the RSB. RCB and RSB penetrations are protected by other Seismic Category I structures (i.e., Safeguards or FB).

Service Loads

- Normal loads – Normal loads are those loads encountered during normal plant operation and shutdown (GDC 4). This load category includes:
  - Dead loads (D) – Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.
  - Live loads (L) – Live loads include any normal loads that vary with intensity or point of application, including moveable equipment. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, varied from zero to full value, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied as appropriate for the loading condition.
  - Thermal loads (T_o) – Thermal loads consist of thermally induced forces and moments resulting from normal plant operation and environmental conditions. These are described in Section 3.8.1.3.1.

- Pipe reactions (R_o) – Pipe reactions are those loads applied by piping system supports during normal operating or shutdown conditions based on the critical transient or steady state conditions. The dead weight of the piping and its contents are included. Appropriate dynamic load factors are used when applying transient loads, such as water hammers.

- Pressure variant loads (P_v) – Pressure variant loads are those external pressure loads resulting from pressure variation either from inside or outside of containment.

- Construction loads – Construction loads are those loads to which the structure may be subjected during construction of the plant. Construction loads will be applied to evaluate partially completed structures, temporary structures, and their respective individual members. Design load requirements during construction for buildings and other structures will be developed in accordance with Standard SEI/ASCE 37. The magnitude and location of construction loads will be applied to generate the maximum load effects of dead, live, construction, environmental, and lateral earth pressure loads. Consideration will be given to the loads and load effects of construction methods, equipment operation, and sequence of construction.

● Test loads – Test loads are those loads that are applied during structural integrity testing or leak-rate testing. This load category includes:

- Test pressure loads (P_t) – Test pressure loads are those loads resulting from the pressure exerted on the RCB during the SIT at 1.15 times the design pressure and during the leak-rate test at 1.0 times the DBA pressure.

- Test thermal loads (T_t) – Test thermal loads include thermal effects and loads experienced by the RCB during the structural integrity and leak-rate tests.

Factored Loads

● Severe environmental loads – Severe environmental loads are those loads that could be encountered infrequently during the life of the plant (GDC 2). This load category includes:

- Wind loads (W) – Wind loads are not applicable on steel portions of the RCB because it is surrounded by other Seismic Category I structures that subsequently provide a shield.

- OBE loads (E) – OBE loads are not applicable to the global design of the U.S. EPR because an OBE level of one-third the SSE has been selected (See Section 3.7.1 for a description of the OBE). However, penetrations will be evaluated for fatigue resulting from OBE-induced stress cycles as described in Section 3.7.3. For these evaluations, the OBE loads will be equal to one-third the SSE load. If a component screens out of an analysis for cyclic operation, Level B service limit load combinations may be eliminated.

- Extreme environmental loads – Extreme environmental loads are those loads that are credible but are highly improbable (GDC 2) This load category includes:

  – SSE loads (E') –SSE loads are those loads generated by an earthquake with a peak horizontal ground acceleration of 0.30 g. Seismic loads in the vertical direction and two orthogonal horizontal directions are considered to act simultaneous. Section 3.7 provides a description of how SSE loads are determined and combined. SSE loads are considered due to applied inertia loads. This includes dead loads, live loads, and hydrostatic loads (i.e., water in storage pools and tanks).

- Abnormal loads – Abnormal loads are those loads generated by a postulated high-energy pipe break accident (GDC 4 and GDC 50). This event is classified as a DBA.

  These loadings include an appropriate dynamic load factor to account for the dynamic nature of the load, unless a time-history analysis is performed to justify otherwise.

  Abnormal loads include the following:

  – Internal flooding loads (F_a) – Loads resulting from the internal flooding of containment during or following a postulated DBA.

  – Pressure load (P_a) – Pressure equivalent static load within or across a compartment or building, generated by a postulated pipe break.

  – Thermal loads (T_a) – Thermal loads generated by the postulated pipe break (including thermal load T_o).

  – Accident pipe reaction loads (R_a) – Pipe reactions generated by the postulated pipe break (including pipe reaction load R_o).

  – Pipe break loads (R_r) – Local loads following a postulated pipe break. Unless a time-history analysis is performed to justify otherwise, these loadings include an appropriate dynamic load factor to account for the dynamic nature of the load. This load category includes:

    - Pipe break reaction loads (R_{rr}) – R_{rr} is defined as the equivalent static load on the structure generated by the reaction of the high energy pipe during the postulated break.

    - Pipe break jet impingement loads (R_{rj}) – R_{rj} is defined as the jet impingement equivalent static load on the structure generated by the postulated break.

    - Pipe break missile impact loads (R_{rm}) – R_{rm} is defined as the missile impact equivalent static load on the structure generated by or during the postulated break, such as pipe whipping.

Other Loads

Other loads refer to postulated events or conditions that are not included in design basis (GDC 4). The loading conditions and effects are evaluated without regard to the bounding conditions under which SSC are required to perform design basis functions. This load category includes:

- Aircraft hazard (A) – Aircraft hazard refers to the loads on a structure resulting from the impact of an aircraft. The evaluation of this loading condition is considered as part of the plant safeguards and security measures. Aircraft hazard loads are not applicable to steel portions of the RCB because it is surrounded by other Seismic Category I structures that provide a shield.
- Explosion pressure wave (B) – Explosion pressure wave refers to the loads on a structure resulting from an explosion in the vicinity of the structure. The evaluation of this loading condition is considered as part of the plant safeguards and security measures. Explosion pressure wave loads are not applicable to steel portions of the RCB because it is surrounded by other Seismic Category I structures that provide a shield.
- Combustible gas loads (P_g1 and P_g2) – Combustible gas loads are pressure loads that result from a fuel-clad metal-water reaction followed by an uncontrolled hydrogen burn during a post-accident condition in a reactor containment (refer to Section 6.2.5).
- Missile loads other than hurricane- or tornado-generated missiles – Missile loads are not applicable to steel portions of the RCB resulting from activities of nearby military installations, turbine failures, or other causes. RCB and RSB penetrations are protected by other Seismic Category I structures (i.e., Safeguards or FBs).

3.8.2.3.2 Design Load Combinations

Loading combinations for steel items of the RCB that are not backed by concrete and are in accordance with Subsection NE of the ASME Code, Section III, Division 1, as augmented by the applicable provisions of RG 1.57 (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50).

The effects of missiles and external events such as hurricanes, tornados, aircraft hazards, and explosion pressure waves are not considered because the containment is protected from these effects by the RSB. RCB and RSB penetrations are protected by other Seismic Category I structures (i.e., Safeguards or FBs).

Loads and loading combinations encompass the soil cases described in Section 3.7.1, using the design criteria described in Section 3.7.1 and Section 3.7.2.

The following loading combinations are considered for ASME Code Class MC RCB components that are completely enclosed within Seismic Category I structures. Stress intensities will be computed in accordance with Article NE-3215 of the ASME Code, Section III, Division 1.

Testing Load Combination

Where:

P* = the set of calculated stress components associated with the analysis results for each load combination

$P^* = D + L + P_t + T_{t.}$

Construction Load Combinations

Temporary construction loads and the effects of environmental loads during construction of ASME Code Class MC components will be considered. SEI/ASCE 37-02 will be used to provide minimum design load requirements for these components during construction.

Design Load Combinations

These include design load combinations for which steel portions of the RCB ASME Code Class MC components will be designed, including LOCA conditions for which containment function is required (GDC 4 and GDC 50).

$P^* = D + L + P_a + T_a + R_{a.}$

Service Load Combinations

The service load combinations correspond to, and include Levels A, B, C, and D service limits as defined in Article NE-3113 of the ASME Code, Section III, Division 1. The post-LOCA flooding condition is also included. Loads are combined according to their sequence of occurrence with consideration of their dynamic effect on containment.

Level A Service Limits

These service limit load combinations are applicable to service loads to which the containment is subjected. This includes LOCA conditions, for which the containment function is required (GDC 4 and GDC 50).

$P^* = D + L + T_o + R_o + P_v$

$P^* = D + L + T_a + R_a + P_{a.}$

RG 1.57 contains load combinations which include P_{g3}, defined as pressure resulting from postaccident inerting. The U.S. EPR does not utilize a postaccident inerting hydrogen control system. Therefore, load combinations containing P_{g3} are not applicable.

Level B Service Limits

These service limit load combinations include the loads subject to Level A service limits, plus the additional loads resulting from natural phenomena during which the plant must remain operational (GDC 2, GDC 4, and GDC 50). For the load effects of the OBE, only the contribution to cyclic loading needs to be considered because the OBE is defined as one-third of the SSE. If a component screens out of an analysis for cyclic operation, based on ASME Section III, Division I, Subsection NE, Subparagraph NE-3221.5, Level B service limit load combinations may be eliminated.

$$P^* = D + L + T_o + R_o + P_v + E$$

$$P^* = D + L + T_a + R_a + P_a + E$$

Level C Service Limits

These service limit load combinations include the loads subject to Level A service limits, plus the additional loads resulting from natural phenomena for which safe shutdown of the plant is required (GDC 2, GDC 4, GDC 50).

$$P^* = D + L + T_o + R_o + P_v + E'$$

$$P^* = D + L + T_a + R_a + P_a + E'$$

$$P^* = D + P_{g1} + P_{g2.}$$

In the last load combination, $P_{g1} + P_{g2}$ should not be less than 45 psig and evaluation of instability is not required as specified by the code.

Level D Service Limits

These service limit load combinations include other applicable service limits and dynamic loads for which containment function is required (GDC 2, GDC 4, and GDC 50).

$$P^* = D + L + T_a + R_a + P_a + R_{rr} + R_{rj} + R_{rm} + E'$$

$$P^* = D + L + F_a + E.$$

3.8.2.4 Design and Analysis Procedures

The steel items described in Section 3.8.2.1 are designed and analyzed in accordance with Article NE-3000 of Subsection NE of the ASME Code, Section III, Division 1, and as augmented by the applicable provisions of RG 1.57 (GDC 1 and GDC 16).

Containment penetrations, or portions thereof, within the jurisdictional boundaries defined by ASME Code, Section III, Division 1, Subsection NE do not exceed the stress intensity limits defined by Articles NE-3221.1, NE-3221.2, NE-3221.3, and NE- 3221.4 of the ASME BPV Code. The stresses induced by the concrete displacements on ASME Subsection NE, Class MC components are displacement limited and hence secondary in nature. Therefore the qualification of the design for primary stress criteria does not consider the effects of concrete displacement. The concrete displacements are non-cyclical. Therefore, ratcheting and fatigue failure of the penetrations due to concrete displacements are not evaluated. The concrete displacements are considered for the qualification of the ASME Subsection CC sleeve components.

Buckling analyses are performed for the equipment hatch, airlocks, construction opening, and high energy piping penetrations (main steam and feedwater). The equipment hatch was qualified in accordance with NE-3222 and Code Case N-284-1. The airlocks were qualified in accordance with NE-3222. The construction opening is qualified in accordance with NE-3133. For high energy piping penetrations (main steam and feedwater); it was determined that buckling is not a failure mechanism for these penetrations.

Equipment Hatch

A rigorous buckling analysis was performed in accordance with NE-3222.1(a)(1). Three-dimensional (3D) finite element submodels of each appurtenance were prepared in ANSYS, Version 11.0. Material nonlinearities and large deformations were considered in accordance with NE-3222.1(a)(1). Material nonlinearity was simulated using the Bilinear Kinematic Hardening material model (BKIN) in ANSYS. Large deflection command NLGEOM in ANSYS was enabled to account for geometric nonlinearity.

In the analysis, the steel liner and ring plate were fixed while constant increments of pressure are applied on the external surface. Other loads, such as seismic and dead weight, do not have any significant effect on buckling of the equipment hatch and have not been applied. The applied pressure was increased until the solution began to diverge. At this point, the analysis was stopped and the critical buckling stress was reached.

*[The maximum allowable buckling stress for Design and Levels A & B service limits was determined by NE-3222.1(a) to be one-third the value of the critical buckling stress. In accordance with NE-3222.2, the allowable limits for Level C and D service limits are 120 percent and 150 percent of the value given in NE-3222.1, respectively.]** The applied pressure in each load condition was compared to the allowable limit to verify that the criterion is met.

*[Consideration of geometric imperfections in the equipment hatch is in accordance with RG 1.193 and ASME Code Case N-284-1.]**

Airlocks

A linear eigen value buckling analysis of the airlock cylinder and torispherical head and penetration sleeve assembly was also performed.

A rigorous buckling analysis was performed in accordance with NE-3222.1(a)(1). 3D finite element submodels of the airlocks were prepared in ANSYS, Version 11.0. Material nonlinearities and large deformations were considered in accordance with NE-3222.1(a)(1). Material nonlinearity was simulated using the elastic-perfectly plastic model for the airlock and the BKIN in ANSYS for the airlock hatch. Large deflection command NLGEOM in ANSYS was enabled to account for geometric nonlinearity. The initial geometric imperfections were conservatively simulated in the model based on the tolerances of the assembly.

In the analysis, the steel liner and ring plate were fixed while constant increments of pressure were applied on the external surface. The full magnitude of the other loads, such as seismic, dead weight, and live, was applied at the first load step. This is conservative since the application of a full magnitude of non-symmetric loads results in a lower critical buckling stress value. The applied pressure was increased until the solution began to diverge. At this point, the analysis was stopped and the critical buckling stress was considered to have been reached.

The maximum allowable buckling stress for design and Levels A and B service limits was determined as one-third the value of the critical buckling stress per NE-3222.1(a). In accordance with NE-3222.2, the allowable limits for Level C and D service limits are 120 percent and 150 percent of the value given in NE-3222.1, respectively. The applied pressure in each load condition was compared to the allowable limit to verify that the criterion was met.

Construction Opening

Due to the simple geometry, the buckling analysis for the construction opening was performed in accordance with NE-3133, and an evaluation according to code Case N-284-1 is not required.

Main Steam Line and Feedwater Line Penetration

Buckling is not a failure mechanism for the main steam and feedwater line penetrations. These penetrations were analyzed using classical analysis where the slenderness ratio (KL/r) must be large enough for buckling to occur. For short columns (or piers) with a KL/r less than 89 (structural steel), the columns will reach yield before buckling occurs. The main steam line (MSL) and feedwater line penetrations are defined as short columns (or piers). The calculated slenderness ratio is below 89, and therefore buckling will not occur.

Combustible gas loads are pressure loads that result from a fuel-clad metal-water reaction followed by an uncontrolled hydrogen burn during a post-accident condition in a reactor containment (refer to Section 6.2.5). Combustible gas loads are evaluated according to the requirements of RG 1.216 and RG 1.136. RG 1.136, Regulatory Position C.5 provides acceptable loads and load combinations for use in reactor containment analysis and design of containments exposed to combustible gas loading conditions. U.S. EPR design does not include an inerting gas system. Hydrogen is the principal combustible gas considered in U.S. EPR design. The maximum containment pressure calculated for the U.S. EPR RCB is 75 psig. This pressure is taken from pressure load time histories of calculations that assume 100 percent fuel clad-coolant reaction followed by burning the hydrogen released by this reaction. RG 1.136, Regulatory Position C.5 and RG 1.7 specify a pressure of 45 psig combined with dead load (D) as a minimum design condition.

Evaluation of the containment penetrations use 3D finite element modeling techniques (ANSYS) using loads and load combinations discussed in Sections 3.8.2.3.1 and 3.8.2.3.2, respectively.

Code class MC components are screened for cyclic service analysis according to the criteria in Article NE-3221.5 of the ASME Code.

Refer to Section 3.5.3 for a description of requirements for missile barrier design and ductility requirements applicable to the design of steel portions of the RCB.

The following sections provide individual descriptions of the design and analysis procedures performed to verify the structural integrity of the steel items. Section 3.8.1 addresses the design and analysis procedures used to qualify the RCB concrete structure for openings provided through the containment pressure boundary for these items. Containment ultimate capacity analysis results are described in Section 3.8.1.4.11, which includes evaluation of major containment steel penetrations.

3.8.2.4.1 Equipment Hatch, Dedicated Spare Penetration, Airlocks, and Construction Opening

The equipment hatch described in Section 3.8.2.1.1 is supported entirely by the concrete shell of the RCB. The sleeve of the equipment hatch is embedded in the concrete containment shell and welded at the periphery to the liner plate. Expansion joints, located in the annulus, allow for differential movement and minimize load transfer between the RCB and RSB walls. The expansion joints maintain the pressure boundary for the annulus ventilation system. The liner plate is thickened in the vicinity of the equipment hatch penetration. The equipment hatch cover is dished and stiffened by a reinforcing ring where it interfaces with the sleeve of the equipment hatch.

The 36-inch diameter dedicated containment spare penetration for containment filtered pressure release is shown in Figure 3.8-119. The portions of the penetration that fall under the jurisdiction of ASME BPV, Subsection NE are bounded by the construction opening closure. The spare penetration has the same cap thickness, but smaller opening size compared to the construction opening closure. Therefore, the stresses in the NE portions of the spare penetration are bounded by the stresses calculated for the construction closure opening. The portions of the penetration that fall under the jurisdiction of ASME BPV, Subsection CC are bounded by the main steam line penetration. The spare penetration is located at a similar elevation, has the same thickness, but smaller opening size compared to the main steam line penetration. Therefore, the strains induced in the CC portions of the spare penetration are bounded by the strains calculated for the main steam line.

The two airlocks described in Section 3.8.2.1.1 are supported by attachment to penetration sleeves embedded in the concrete shell of the RCB. Expansion joints provide for differential movements and minimize load transfer between containment and shield walls. The doors for both ends of the airlocks are flat, and the bulkhead ends of the components are dished.

The construction opening closure cap described in Section 3.8.2.1.1 is attached to and supported by a sleeve embedded in the concrete shell of the RCB. The closure cap is a dish shaped metal structure welded to the embedded sleeve flange.

The equipment hatch, airlocks, and construction opening closure cap and sleeve are evaluated for the combinations of loads described in Section 3.8.2.3.2. Analyses and limits for the resulting stress intensities in the equipment hatch, airlocks, and the construction opening closure cap and sleeve are designed in accordance with Articles NE-3130, NE-3200, NE-3325, and NE-3326 of Section III, Division 1 of the ASME Code.

3.8.2.4.2 Piping, Electrical, and Fuel Transfer Tube Penetration Sleeves

The penetration sleeves are welded to the containment liner plate and are anchored to the RCB concrete shell. Penetration sleeves are subjected to various combinations of mechanical, thermal, and seismic loadings and will be evaluated for the combination of loads described in Section 3.8.2.3.2.

If the penetration sleeves are subjected to cyclic service, the associated peak stress intensities will be evaluated. The required analysis and associated stress intensity limits will be in accordance with Articles NE-3130 and NE-3200 of Section III, Division 1 of the ASME Code.

3.8.2.4.3 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.4, 2.5, 3.3, 3.5, 3.7, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections. A cross-reference between U.S. EPR FSAR sections and information required by SRP Section 3.8.4, Appendix 3C is provided in Table 3.8-17.

3.8.2.5 Structural Acceptance Criteria

Structural acceptance criteria for steel containment items described in Section 3.8.2.1 are in accordance with Subsection NC and NE of the ASME Code, Section III, Division 1, including allowable stress limits, strain limits, deformation limits, and factors of safety. These are augmented by the requirements of RG 1.57 and RG 1.216 (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). Containment steel items not backed by concrete that are intended to resist pressure will be designed to meet the acceptance criteria for the load combinations listed in Section 3.8.2.3.2.

Steel items that are an integral part of the RCB pressure boundary will be designed to meet minimum leakage rate requirements. The leakage rate must not exceed the acceptable value indicated in the applicable technical specification.

The design and analysis methods, as well as the type of construction materials, are chosen to allow assessment of the capability of steel items to function properly throughout the plant life.

A SIT is performed as described in Section 3.8.2.7. Surveillance testing provides assurance of the continuing ability of each item to meet its design functions. Surveillance requirements are addressed in Section 3.8.2.7.

Items that form part of the containment pressure boundary are stamped in accordance with the applicable section of the ASME Code used for their design or fabrication.

3.8.2.6 Materials, Quality Control, and Special Construction Techniques

*[Steel items that are not backed by concrete that are part of the containment pressure boundary are fabricated from materials that meet the requirements specified in Article NE-2000 of Section III, Division 1 of the ASME Code, except as modified by applicable and acceptable ASME Code cases]** (GDC 1). SA-516 Grade 70 material is used for major steel components of the penetration assemblies. The materials are defined in Table 6.1-1.

Quality control for containment steel items conforms to Articles NE-2000, NE-4000, and NE-5000 of Section III, Division 1 of the ASME Code (GDC 1).

Section 3.8.1.6 provides a description of welding requirements for steel items for the RCB, quality control for steel items for the RCB, and materials used for penetration sleeves, steel embedments, and corrosion retarding compounds.

Use of neoprene-based seals are kept to a minimum because of the presence of fluoride or chloride ions and the increased potential for stress corrosion cracking.

The seals for the airlocks and the equipment hatch make use of elastomer seal material (Dupont Viton®, or equal) which is compressed by the action of the mechanical closure devices associated with each of the components. This material is recessed into two concentric grooves (double seals) around the perimeter of the airlock doors and around the equipment hatch flange penetration mating flange. This material is selected based on its ability to maintain elasticity at elevated temperatures for extended durations and to be in compliance with the materials tested for severe accident conditions as specified in NUREG/CR-5096 (Reference 64).

Steel items such as the equipment hatch, airlocks, fuel transfer tube, and penetrations are prefabricated and installed as subassemblies during construction. No special techniques are used for construction of containment steel items not backed by concrete. Section 3.8.1.6 provides additional information of modular construction techniques used for the RCB.

3.8.2.7 Testing and Inservice Inspection Requirements

*[A SIT is performed for steel containment components not backed by concrete in accordance with Article NE-6000 of Subsection NE of the ASME Code, Section III, Division 1]** (GDC 1).

Inservice inspections for the steel pressure retaining subassemblies follow the requirements of the ASME Code, Section XI, Subsection IWE with the additional requirements of 10 CFR 50.55a (GDC 1 and GDC 16). Section 6.2.6 describes the leakage tests and associated acceptance criteria.

Vendor testing and in-situ testing of the seals is conducted to provide assurance of the seal performance for normal operating conditions and for temperature and pressure conditions associated with a loss of coolant accident. Once this equipment is installed in containment, the air space between the two seals will be continuously maintained under a negative pressure by connection to the Leak-Off system. This system is also used to pressurize the air space between the seals for in-situ testing operations.

3.8.3 Concrete and Steel Internal Structures of Concrete Containment

3.8.3.1 Description of the Internal Structures

RB internal structures consist of reinforced concrete walls and floors, steel framing members, and other concrete and steel structural elements that are located inside of the RCB. The RB internal structures provide support for components and radiation shielding for the RCS and refueling operations. The foundation basemat inside of the RCB supports the RB internal structures at the bottom interface. To prevent an interaction between the structures for design basis loading conditions, clearance is maintained between the containment wall and internal structures. RB internal structures important to safety are not shared with another unit (GDC 5).

The RB internal structures are Seismic Category I, except for miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items. These miscellaneous structures are designed as Seismic Category II to prevent impairment of the design basis safety function of the Seismic Category I safety-related SSC in the event of a SSE. Seismic classification of structures, systems and components (SSC) is addressed in Section 3.2.

The following figures show the main levels of the RB internal structures and sectional views of the building:

- Figure 3.8-2—Reactor Building Plan at Elevation -20 Feet (top of the foundation basemat inside containment).
- Figure 3.8-3—Reactor Building Plan at Elevation -8 Feet (top of concrete at start of containment wall).
- Figure 3.8-4—Reactor Building Plan at Elevation +5 Feet (top of heavy floor for nuclear steam supply system (NSSS) component support).
- Figure 3.8-5—Reactor Building Plan at Elevation +17 feet (plan at centerline of reactor vessel piping nozzles).
- Figure 3.8-6—Reactor Building Plan at Elevation +29 feet (top of grating floor for component access).
- Figure 3.8-7—Reactor Building Plan at Elevation +45 feet (top of grating floor for component access).
- Figure 3.8-8—Reactor Building Plan at Elevation +64 feet (top of concrete operating floor).
- Figure 3.8-9—Reactor Building Plan at Elevation +79 feet (top of partial concrete floor).

- Figure 3.8-10—Reactor Building Plan at Elevation +94 feet (top of pressurizer cubicle).
- Figure 3.8-11—Reactor Building Section A-A.
- Figure 3.8-12—Reactor Building Section B-B.
- Figure 3.8-13—Reactor Building Section C-C.

The RB internal structures consist of the following major structures that support nuclear steam supply system (NSSS) components, provide access for plant operation and maintenance, and support safety-related functions of the plant:

- Reactor vessel (RV) support structure and reactor cavity.
- Steam generator (SG) support structures.
- Reactor coolant pump (RCP) support structures.
- Pressurizer (PZR) support structure.
- Operating floor and intermediate floors.
- Secondary shield walls.
- Refueling canal walls.
- Polar crane support structure.
- The RB internal structures basemat.
- IRWST.
- Core melt retention area.
- Convection and rupture foils.
- Reactor containment building doors.

These major RB internal structures are further described in Section 5.4, which contains descriptions of steel supports for the RV, four SGs, four RCPs, and the PZR.

Supports are also provided for distribution systems as part of the RB internal structures, which include pipe supports; equipment supports; cable tray and conduit supports; and heating, ventilation and air conditioning (HVAC) duct supports. Platforms, ladders, stairs, guard rails, and other miscellaneous structures are provided for equipment access and maintenance.

3.8.3.1.1 Reactor Vessel Support Structure and Reactor Cavity

The RV support structure is comprised of a reinforced circular concrete wall that extends from the top of the RB internal structures basemat at elevation -20 feet, 2 inches to the steel supports for the RV piping at approximately elevation +20 feet. This circular wall also serves as the interior wall for the IRWST, and provides radiation shielding for the RV and RCP. A narrow chamber extends through the circular wall just above the internal structures basemat to provide an outlet from the bottom of the RV cavity to the core melt retention area. The top, inside edge of the circular concrete wall supports eight steel RV support assemblies that are located under the RCP nozzles. Section 5.4.14 describes the design of the RV steel supports. The circular concrete wall also functions as the primary radiation shield wall around the RV. The wall is approximately 8 feet, 11 inches thick. The reactor refueling cavity begins at the top of the circular wall at elevation +24 feet, 5 inches.

Large penetrations in the circular RV support concrete wall are provided for the primary loop piping and the cavity ventilation system. A permanently installed cavity ring (Refer to Figure 9.1.4-12—Permanent RPV Refueling Cavity Ring - General Configuration) and neutron shield assembly rests on an embedded ring at the top of the wall. This cavity ring and shield assembly is fabricated of stainless steel and radiation shielding material that bridges the annular gap between the RV and vessel cavity concrete wall. This ring seals the lower RV cavity to prevent water leakage from the refueling canal located above.

The cavity ring is designed to accommodate the expansion and contraction of the RPV during heatup and cooldown. The cavity ring is designed to Seismic Category I requirements and to meet the stress limits of ASME BPVC, Section III, Subsection ND. Base metal and weld materials are consistent with specifications in ASME Code Section II. Welding procedures and welders will be qualified in accordance with ASME Code Section IX. Welds will be examined in accordance with ASME Code Section V. The cavity ring does not rely on inflated seals, gaskets, o-rings, or other active components. To withstand the impact of one fuel assembly, the portion of the cavity ring that functions as an expansion joint has a protective cover plate (Refer to Figure 9.1.4-12).

The RV supports and cavity concrete wall resists normal operating loads, seismic loads, and loads induced by postulated pipe rupture events, including a LOCA (GDC 4 and GDC 5). The supports limit the movement of the RV within allowable limits under the applicable combinations of loadings, and minimize resistance to thermal movements during plant operations.

Refer to Figure 3.8-2, Figure 3.8-3, Figure 3.8-4, Figure 3.8-11, Figure 3.8-12, and Figure 3.8-13 for general arrangement layouts of the RV support structure.

3.8.3.1.2 Steam Generator Support Structures

The SGs are supported and restrained to resist normal operating loads, seismic loads, and loads induced by pipe rupture. The supports prevent the rupture of the primary reactor coolant pipes due to a postulated rupture in the main steam (MS) or feedwater lines. The supports minimize resistance to thermal movements during operation.

The 6 feet, 7 inches thick heavily reinforced concrete floor at elevation +4 feet, 11 inches supports the four SGs. Four steel columns with pinned joints are provided under each SG to support the vessels vertically from the concrete floor. Keyed joints at the top of the steel support columns interface with lower lateral steel supports that connect to steel embedments in the concrete cubicle walls for each SG. Section 5.4.14 describes the design of these steel component supports.

The RB internal structure concrete walls form individual cubicles for each of the SGs. These walls isolate the SGs to protect other plant SSC in the event of a pipe rupture in one of the piping reactor coolant loops (RCL). The SG cubicle outer walls also serve as secondary shield walls for protection against radiation from the reactor piping and coolant, as described in Section 3.8.3.1.6.

Steel supports within each of the cubicles, which are mounted to the concrete slab at approximately elevation +64 feet, provide upper lateral support for the SGs. Connection of the upper lateral supports to the concrete includes steel subassemblies that are embedded in the concrete slab and cubicle walls. Section 5.4.14 provides a description of the design of the SG upper lateral supports.

Steel framing and grating platforms provided throughout each SG cubicle provide maintenance access to the SGs. Pipe supports and restraints are mounted to the SG cubicle walls to support piping during normal and abnormal plant conditions, and to protect other plant equipment from the effects of a broken pipe in the event of a postulated accident.

Removable steel grating is provided over the top of each SG cubicle at elevation +113 feet. This allows for potential replacement of the SGs over the life of the plant. Removable panels in the interior wall of each SG cubicle above approximately elevation +70 feet enable future component replacement or major maintenance activities. These reinforced concrete wall panels are keyed into the side walls of the SG cubicles and to the slab at the bottom of the panels to prevent dislodgment during a seismic event.

See Figures 3.8-4, 3.8-5, 3.8-6, 3.8-7, 3.8-8, 3.8-9, 3.8-10, 3.8-11, 3.8-12, and 3.8-13 for the general arrangement layout of the SG support structures.

3.8.3.1.3 Reactor Coolant Pump Support Structures

The RCPs are supported to prevent excessive deflection during normal operating, seismic, and pipe rupture conditions. Under LOCA and other loading conditions, the pumps are prevented from generating missiles that might damage other safety-related components. The RCP supports minimize resistance to thermal movements during operation.

The 6 feet, 7 inches thick concrete floor supports the four RCPs at elevation +4 feet, 11 inches adjacent to the SGs. Three steel columns with pinned joints are provided under each RCP for support. Steel subassemblies are embedded in the concrete floor to connect the RCP support columns to the structure. Section 5.4.14 describes the design of these steel component supports.

The RB internal structure walls form cubicles for each of the RCPs that are adjacent to but separate from each of the SG cubicles. These walls isolate each of the RCPs to protect plant SSC in the event of a pipe rupture in one of the RCLs. As with the SG cubicles, the RCP cubicle outer walls also serve as secondary radiation shield walls. Steel supports mounted to the walls of the RCP cubicles provide lateral support for the top of the RCPs. There are two lateral supports for each RCP, which are located at approximately elevation +28 feet, with portions of the supports embedded into the concrete walls. Section 5.4.14 provides a description of the design of the RCP lateral supports.

Steel framing and grating platforms provide maintenance access to the pumps throughout each RCP cubicle. Pipe supports and restraints are mounted to the cubicle walls to support piping during normal and abnormal plant conditions and to protect other plant equipment from the effects of a broken pipe in the event of a postulated accident. Removable steel grating or concrete hatches over the top of each RCP cubicle allow for pump maintenance and replacement.

See Figures 3.8-4, 3.8-5, 3.8-6, 3.8-7, and 3.8-11 for the general arrangement layout of the RCP support structures.

3.8.3.1.4 Pressurizer Support Structure

The PZR is supported and restrained to resist normal operating loads, seismic loads, and loads induced by postulated pipe rupture. Upper lateral supports minimize resistance to thermal movements during operation.

The PZR is located in a compartment formed by the secondary shield walls, SG cubicle walls, reactor cavity, and other intermediate walls. The PZR is supported vertically by three steel assemblies located at an intermediate concrete floor at approximately elevation +49 feet. Steel subassemblies are embedded in the concrete floor to connect the PZR support assemblies to the structure. Eight lugs connected to the embedments in an intermediate concrete floor at approximately elevation +68 feet support the top of the PZR laterally. Section 5.4.14 describes the design of the steel supports.

Several intermediate concrete and steel grating floors are provided inside the PZR cubicle for maintenance access. A removable concrete hatch is provided in the top of the cubicle at elevation +93 feet, 6 inches to allow the replacement of the PZR.

See Figures 3.8-6, 3.8-7, 3.8-8, 3.8-9, 3.8-10, 3.8-12, and 3.8-13 for the general arrangement layout of the PZR support structure.

3.8.3.1.5 Operating Floor and Intermediate Floors

The operating floor of the RB internal structures is located at approximately elevation +64 feet. Refueling operations take place at the operating floor, which provides support for the refueling machine and other equipment. The equipment hatch provides access at the operating floor level for bringing large components and equipment into the RCB. The emergency airlock provides access to the operating floor at [ ]

Intermediate floors are provided throughout the RB internal structures. These floors support and provide access to equipment and components throughout the building. The floor at elevation +4 feet, 11 inches supports the SGs and RCPs and also serves as the roof for the IRWST.

As described in Section 3.8.1.1, separation is provided between RB internal structures floor slabs and walls and the RCB to allow for differential movement. Internal floors are constructed of reinforced concrete or steel grating supported by structural steel framing. The RB internal structures floors are shown in Figures 3.8-2, 3.8-3, 3.8-4, 3.8-5, 3.8-6, 3.8-7, 3.8-8, 3.8-9, 3.8-10, 3.8-11, 3.8-12 and 3.8-13.

3.8.3.1.6 Secondary Shield Walls

The secondary shield wall provides radiation shielding and support for components. This shield wall is a circular reinforced concrete wall with an external radius of approximately 63 feet. The reinforced concrete secondary shield wall is 3 feet, 11 inches thick where it shields the RCS pipes and SG tube bundles. The secondary shield wall is anchored to the RB internal structures basemat at approximately elevation -7 feet, 6 inches. The secondary shield wall extends from the RB internal structures basemat at approximately elevation -7 feet, 6 inches up to the operating floor at approximately elevation +64 feet. This wall continues up to approximately elevation +113 feet around the SG cubicles.

Figures 3.8-2, 3.8-3, 3.8-4, 3.8-5, 3.8-6, 3.8-7, 3.8-8, 3.8-9, 3.8-10, 3.8-11, 3.8-12 and 3.8-13 show the layout of the secondary shield walls.

3.8.3.1.7 Refueling Canal Walls

The refueling canal is located above and to the south of the reactor cavity on the FB side of the RB internal structures. The refueling canal is filled with borated water during reactor refueling operations for radiation shielding of the reactor fuel. The refueling canal enables the transfer of the reactor fuel to and from the FB through the fuel transfer tube, as described in Section 3.8.2, and provides storage areas for reactor components and lifting equipment.

The refueling canal is constructed of approximately 4 feet thick reinforced concrete walls. The interior surface of the refueling canal concrete walls and floor are lined with 0.25 inch thick stainless steel liner plates. The liner plates are not load bearing structural members. Leak detection channels are provided at joints in the refueling canal liner plates to contain and monitor potential leakage of water when the canal is flooded during reactor refueling operations. The refueling canal walls in conjunction with the secondary shield walls form the SG and RCP cubicles. The refueling canal walls connect the cubicles and help provide lateral support for the RB internal structures.

A concrete missile shield is provided on top of the refueling canal walls to absorb the impact of a control rod ejection due to the postulated failure of a control rod drive mechanism (CRDM) housing. See Section 3.5.1.2 for a description of the CRDM missile shield.

See Figures 3.8-5, 3.8-6, 3.8-7, 3.8-8, 3.8-11, 3.8-12 and 3.8-13 for the general arrangement layout of the refueling canal walls.

3.8.3.1.8 Polar Crane Support Structure

Structural steel built-up crane girders mounted on crane brackets evenly spaced around the inside face of the RCB wall support the polar crane. The crane brackets are welded from steel plates and embedded in the RCB wall concrete. Section 9.1.5 provides the details of the overhead heavy handling system, which includes the polar crane.

3.8.3.1.9 Reactor Building Internal Structures Basemat, In-Containment Refueling Water Storage Tank, and Core Melt Retention Area

The RB internal structures basemat is comprised of the reinforced concrete in the lower part of containment above the liner plate. The top of the RB internal structures basemat is nominally at elevation -20 feet, 2 inches, with the top elevation of the basemat lowered in the core melt retention area. The bottom of the RB internal structures basemat is at elevation -25 feet, 7 inches.

The internal structures basemat concrete fills the lower area of the containment liner and supports the RB internal structures. Anchors are not provided on the inside surface of the liner or on the underside of the flat portion of the liner. Anchors are provided around the outer surface of the embedded sides of the liner. Overturning moments and sliding forces of the RB internal structures relative to the liner plate are resisted by structural dead weight and lateral bearing. Section 3.8.1.4.10 describes the interface of the RB internal structures with the RCB liner plate.

The IRWST is located above the internal structures basemat. The IRWST provides storage of refueling water and serves as the source of water for the safety injection and containment spray systems. The concrete floor at top elevation +4 feet, 11 inches forms the roof of the IRWST. The reactor cavity circular concrete wall and the concrete side walls of the core melt retention area form the inside walls of the IRWST. The concrete around the embedded lower portion of the containment liner forms the outside walls of the IRWST. The entire surface of the IRWST is lined with stainless steel. The liner plates are not load bearing structural members.

The core melt retention area is located above the internal structures basemat between azimuths 129° and 231°. The core melt retention system provides for mitigation of potential severe accident (SA) scenarios. This area provides a volume to capture molten core debris up to and including the total inventory of the reactor core, RV internals, and lower RV head. The core melt retention area spreads the molten core debris over a large area and stabilizes the core debris with water. Spreading increases the surface-to-volume ratio of the melt to promote fast cooling and to limit the release of radionuclides into the containment atmosphere. These features provide a passive transformation of the molten core into a cooled, solid configuration without operator action. The core melt retention features are described in Section 19.2.

See Figures 3.8-2, 3.8-11, 3.8-12, and 3.8-13 for the general arrangement layout of the RB internal structures basemat, IRWST, and core melt retention area.

3.8.3.1.10 Distribution System Supports

Structural steel supports are provided for distribution systems as part of the RB internal structures. These include pipe supports, equipment supports, cable tray and conduit supports, HVAC duct supports, and other component supports. Distribution system supports are primarily constructed of steel shapes and tubing, which are anchored to the concrete RB internal structures using embedded steel plates, cast-in-place anchor bolts, and drilled-in concrete anchors.

3.8.3.1.11 Platforms and Miscellaneous Structures

Platforms and miscellaneous structures (e.g., ladders, guard rails, stairs) are provided for access and maintenance to plant equipment and components. These items are primarily constructed of steel beams, angles, channels, tubing, and grating.

3.8.3.1.12 Reactor Containment Building Rupture and Convection Foils

The rupture and convection foils are installed at the steam generator pressure equalization ceiling and are part of the CONVECT system. The design and performance of the CONVECT system is described in Section 6.2.5.

3.8.3.1.13 Reactor Containment Building Doors

The RCB design allows personnel access to designated areas during plant normal operating conditions for planned maintenance and inspection activities. To make the RCB accessible during these normal plant operating conditions, the RCB is separated into two regions. This two-region concept is referred to as the “two-room” or “two zone containment.” The two regions or rooms in the RCB are the non-accessible and the accessible area. The containment dome, the annular rooms, and the operation area are part of the accessible or service areas. The equipment spaces inside the secondary shield wall and the primary or bioshield wall are part of the non-accessible area, which is surrounded by walls, closed radiation doors (radiation shielding for personnel and equipment), and the CONVECT system discussed in Section 3.8.3.1.12.

The two-room RCB design, versus one room, maintains a permanent containment environmental envelope that allows access and entry to the accessible service spaces at defined periods during power operation. Separation is provided by concrete walls and physical barriers, or “doors,” that can be closed to separate adjoining rooms or compartments. Doors, used as barriers as part of the two-room containment, are designed to open or provide an opening to relieve pressure in the case of a high energy line break (HELB), making them a “pressure relief device” for their respective compartments.

The RCB has several different doors designs. Table 3.8-18 lists the RCB doors, and Figure 3.8-137 through Figure 3.8-144 show their location by elevation. Different door types provide barriers for access control, water tight sealing, pressure relief, and radiation shielding. Some doors provide multiple functions. The radiation doors provide shielding protection for equipment and personnel. The shielding provided by the doors is equivalent to the concrete wall in which the door is installed.

RCB doors fall into four general types of design:

- Radiation protection doors, with a pressure relief function.
- Radiation protection doors, without a pressure relief function.
- Interior building room doors, with a pressure relief function.
- Interior building room doors, without a pressure relief function.

RCB radiation protection doors are large and are typically integrated in the shield walls surrounding the equipment spaces or inaccessible areas of the RCB during normal operation of the plant. The RCB has 47 radiation protection doors. Forty-five of the radiation protection doors also provide a pressure relief function. These 45 radiation doors are designed so that the whole door "swings" open during a pressure differential related accident to meet its pressure relief function. This swinging or bursting open response to the differential pressure generated across the doors during a HELB accident verifies that an acceptable differential pressure across structural walls of adjacent rooms or compartments is maintained. These 45 radiation protection doors have a swing/burst pressure of 2.9 psid and only open in one direction. The doors open into the room where the hinges are mounted. In the case of the radiation protection doors that have an active and an inactive leaf, both the active and the inactive leaf are assumed to swing open. These doors require special attention because their momentum and significant weight in a "burst" opening can impact civil structures. Door stops and other features are used to limit adverse impact, while still maintaining the opening efficiency of the doors. A portion of these doors are credited in the analyses to prevent compartment overpressurization during a HELB and are classified as safety-related and Seismic Category I as shown in Table 3.2.2-1 and Table 3.8-18. Refer to Section 6.2.1.2 for the dynamic effects of postulated HELB in individual compartments and allowable venting capability to prevent differential pressures from reaching the structural limits of the compartment walls. The only doors credited to open are safety-related doors identified in Table 6.2.1-13 and Table 3.8-18. The remaining radiation doors are classified as non-safety augmented quality (NS-AQ) and Seismic Category II as shown in Table 3.2.2-1 and Table 3.8-18.

The radiation protection doors in the Reactor Building that include a pressure relief function are designed so that the whole door "swings" open during a pressure differential related accident (High Energy Line Break) to meet its pressure relief function. The locking mechanisms of those radiation protection doors incorporate a "shear" pin which shears under the designated burst pressure in Table 3.8-18 to provide for the opening of these doors in the event of a High Energy Line Break. This burst pressure value for the radiation swing doors specified in Table 3.8-18, when applied across the surface area of the door, results in a significantly large force required to break or shear the pin such that a person pulling or pushing on these doors with a reasonable pulling or pushing force (without using extraordinary force, tools, equipment, etc.) could not inadvertently open this door when the pin is engaged as part of the locking mechanism.

During normal operation, the shear pin also functions to engage as part of the locking mechanism on the radiation swing door to maintain access control and prevent unauthorized entry, and disengages to allow for emergency egress. The design of the locking mechanism for the radiation protection swing doors is such that the shear pin will engage after door closure and disengage prior to opening to preclude damage to the shear pin while performing the entry and egress functions. For emergency egress, a "panic" lock system is employed which allows for manual opening of the swing doors from the inside to provide an exit out of the room during an emergency even if the door is locked from the outside.

Thus, the radiation protection doors dual function design allows it to meet 10 CFR 20.1601 and 10 CFR 20.1602 requirements for access control to high radiation areas and very high radiation areas, respectively, while also meeting its pressure relief function for accident mitigation.

Two radiation protection doors exist at Elevation +5 feet that do not provide a pressure relief function. These doors are motor-operated, rolling doors without hinges.

There is no shear pin incorporated into the locking mechanism design of these doors. The locking mechanism for the motor-operated rolling doors is designed so that these doors cannot be inadvertently opened during operation while also allowing for emergency egress. To address inadvertent operation/opening of the door, the locking mechanism is designed such that locking the door will de-energize the motor such that the door cannot be opened. For emergency egress, a "panic" lock system is employed which allows for manual opening of the motor-operated rolling doors from the inside using a handle which decouples the electric drive to allow door opening even with the motor de-energized. This provides an exit out of the room during an emergency even if the door is locked from the outside.

Periodic door testing at an ongoing torque of 100 ft. lbs. for emergency egress will be performed during plant operation, using the applicable prerequisites, test methods, data required, and acceptance criteria of Section 14.2.12.5.2, Reactor Containment Building Doors, test #044, to confirm that the radiation doors are capable of meeting the applicable requirements of Technical Specification 5.7.1, Technical Specification 5.7.2, 10 CFR 20.1601(d) and 10 CFR 20.1602. Periodic testing provides reasonable assurance that the doors will maintain their ability to adequately allow emergency egress. Doors that are not capable of meeting design requirements per this test will be administratively controlled until repairs have been completed. A combined license (COL) applicant that references the U.S. EPR design certification will include in its normal radiation protection program administrative controls to ensure the requirements of 10 CFR 20.1601(d) and 10 CFR 20.1602 are met through periodic testing of reactor containment building doors (i.e., every 24 months).

Other interior building room doors are primarily used to divide hallways and rooms. In addition to providing personnel access, these interior door types may also provide functions such as leak tightness. The RCB has 45 interior building doors. Forty-three of these doors provide a pressure relief function. These doors are designed with an integral pressure relief aperture or "blowout panel." The pressure relief aperture is designed to open or burst in only one direction to meet its pressure relief function during a pressure differential accident. Generally, these 43 interior building room doors have a burst pressure of 1.45 psid. A portion of these doors are credited in the analyses to prevent compartment over pressurization during a HELB and are classified as safety-related and Seismic Category I as shown in Table 3.2.2-1 and Table 3.8-18. Refer to Section 6.2.1.2 for the dynamic effects of postulated HELB in individual compartments and allowable venting capability to prevent differential pressures from reaching the structural limits of the compartment walls. The only doors with burst panels credited to open are safety-related doors identified in Table 6.2.1-13 and Table 3.8-18. The remaining interior building doors are classified as NS-AQ and Seismic Category II as shown in Table 3.2.2-1 and Table 3.8-18.

The pressure relief door -8 feet, door 8 is the venting area room door for the spreading compartment and has a higher burst pressure up to 2.9 psid.

The doors with blowout panels are provided with panel or missile restraints to prevent their momentum from adversely impacting civil, mechanical, electrical, or I&C components in the immediate area.

There are two interior building room doors at Elevation +17 feet that do not provide a pressure relief function. These two doors provide access and entrance to the RV cavity pool area and the transfer canal pool area, are part of the pool liner, and are required to be water tight, with no pressure relief function or bust pressure capability required in support of a HELB.

3.8.3.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used in the design, fabrication, construction, testing, and inservice inspection of concrete and steel RB internal structures (GDC 1, GDC 2, GDC 4 and GDC 5). Section 5.4.14 describes the applicable codes, standards, and specifications for the design of NSSS component supports.

3.8.3.2.1 Codes and Standards

- ACI 301-05, Specifications for Structural Concrete for Buildings.
- ACI 304R-00, Guide for Measuring, Mixing, Transporting, and Placing Concrete.
- ACI 305.1-06, Specification for Hot-Weather Concreting.
- ACI 306R-88 (Re-approved 2002), Cold-Weather Concreting (Reference 49).
- ACI 306.1-90 (Re-approved 2002), Standard Specification for Cold Weather Concreting.
- ACI 308R-01, Guide to Curing Concrete (Reference 50).

- ACI 308.1-98, Standard Specification for Curing Concrete (Reference 39).
- ACI 311.4R-05, Guide for Concrete Inspection (Reference 40).
- ACI 347-04, Guide to Formwork for Concrete.
- ACI 349-01/349-R01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (exception described in 3.8.4.4 and 3.8.4.5) (GDC 1).
- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of Condition A strength reduction factors even when supplemental reinforcement is provided (Reference 63).
- ACI 349.1R-07, Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures (Reference 41).
- AISC 303-00, Code of Standard Practice for Steel Buildings and Bridges (Reference 42).
- ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2, 2004 (GDC 1).
- AISC 348-00/2000 RCSC, Specification for Structural Joints Using ASTM A325 and A490 Bolts (Reference 44).
- ANSI/AWS D1.1-2000, Structural Welding Code - Steel.
- ANSI/AWS D1.4-2005, Structural Welding Code - Reinforcing Steel.
- ANSI/AWS D1.6-1999, including January 6, 2005 update, Structural Welding Code – Stainless Steel.
- ANSI/AWS D1.8-2005, Structural Welding Code – Seismic Supplement (Reference 45).
- ASME Boiler and Pressure Vessel Code, Section III, Division 2 - Code for Concrete Reactor Vessels and Containments (Reference 1) (GDC 1).
- ASME Boiler and Pressure Vessel Code, Section III, Division 1 –Nuclear Power Plant Components (Reference 73) (GDC 1).
- ASME NOG-1-04, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder).

3.8.3.2.2 Specifications

Industry standards (e.g., those published by the ASTM) are used to specify material properties, testing procedures, fabrication methods, and construction methods.

Section 3.8.3.6 addresses the applicable standards used.

Structural specifications cover areas related to the design and construction of the RB internal structures. These specifications emphasize important points of the industry standards for these structures and reduce options that otherwise would be permitted by the industry standards. These specifications cover the following areas:

- Concrete material properties.
- Mixing, placing, and curing of concrete.
- Reinforcing steel and splices.
- Structural steel.
- Stainless steel liner plate and embedments.
- Miscellaneous and embedded steel.
- Anchor bolts.
- Expansion anchors.
- Polar crane.
- Miscellaneous cranes and hoists.

3.8.3.2.3 Design Criteria

- ACI 349-01/349-R01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures (GDC 1).
- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of Condition A strength reduction factors even when supplemental reinforcement is provided (Reference 63).
- ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).

3.8.3.2.4 Regulations

- 10 CFR 50, Appendix A, General Design Criteria for Nuclear Power Plants, GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50.
- 10 CFR 50, Appendix B, Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants.

- 10 CFR 50, Appendix S, Earthquake Engineering Criteria for Nuclear Power Plants.

3.8.3.2.5 NRC Regulatory Guides

RGs applicable to the design and construction of RB internal structures:

- RG 1.61, Revision 1, March 2007 (exception described in 3.7.1).
- RG 1.69, December 1973.
- RG 1.136, Revision 3, March 2007 (exception described in 3.8.1.3).
- RG 1.142, Revision 2, November 2001 (exception described in 3.8.3.3).
- RG 1.160, Revision 2, March 1997.
- RG 1.199, November 2003 (exception described in 3.8.1.4).

3.8.3.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the loads over a broad range of site conditions (GDC 1, GDC 2, GDC 4, GDC 5 and GDC 50). The loads on RB internal structures are separated into the following categories:

- Normal loads.
- Severe environmental loads.
- Extreme environmental loads.
- Abnormal loads.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for RB internal structures, or perform additional analyses to verify structural adequacy.

Section 5.4.14 addresses the loads and loading combinations and design stress limits for the RCS component and pipe supports.

3.8.3.3.1 Design Loads

*[Loads on RB internal structures are in accordance with ACI 349-2001 and the guidelines of RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures.]** RG 1.142 delineates the acceptability of ACI 349-1997 with exceptions. The U.S. EPR standard plant design is based on the 2001 edition of the code, with the exceptions noted above. Use of the 2001 edition of the code is acceptable as it incorporates needed updates to the 1997 version. This includes anchorage of wall

reinforcing without the use of confined cores in certain situations, and is in keeping with RG 1.199, which adopted the 2001 version Appendix B with exceptions in the area of load combinations. In addition, the guide has supplementary recommendations in the areas of materials, installation, and inservice inspection. The guidelines of RG 1.199 are followed with the exception described in Section 3.8.1.4.10. This exception allows the use of Appendix D to ACI 349-06 (with exception stated in Section 3.8.1.2.1) for concrete anchorage design. This exception is acceptable as it results in an equivalent or conservative anchorage design when compared to that of Appendix B to ACI 349-01.

Seismic Category I safety-related RB internal structures are designed for the following loads.

Normal Loads

Normal loads are those loads encountered during normal plant operation, startup, shutdown, and construction (GDC 4). This load category includes:

- Dead Loads (D)—Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.
- Live Loads (L)—Live loads include any normal loads that vary with intensity or point of application (or both), including moveable equipment. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, varied from zero to full value, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied according to the loading condition. In general, a live load of 500 pounds per square foot is applied to RB internal structures concrete floors and a load of 175 pounds per square foot is applied to steel grating floors and platforms. Live loads are applied to cranes and their supports for the lifting capacity and test load applied for the lifting device. Additional point loads are applied to concrete floors and concrete and steel floor beams in local design.
- Hydrostatic Loads (F)—Hydrostatic loads are due to fluids stored in pools and tanks in the RB internal structures (e.g., the IRWST and refueling canal). Pools and tanks may have either constant or fluctuating liquid levels. Hydrodynamic loads resulting from seismic excitation of fluids are included as a component of the safe shutdown earthquake load.
- Thermal Loads (T_o)—Thermal loads consist of thermally induced forces and moments resulting from normal plant operation and environmental conditions. Thermal loads and their effects are based on the critical transient or steady-state condition. Thermal expansion loads due to axial restraint, as well as loads resulting from thermal gradients, are considered. The following ambient air temperatures are for normal operation.

  RB internal ambient temperatures:

- During normal operation:
  - Equipment Area: 131°F (maximum), 59°F (minimum).
  - Service Area: 86°F (maximum), 59°F (minimum).
- During normal shutdown: 86°F (maximum), 59°F (minimum).

- Pipe Reactions (R_o)—Pipe reactions are those loads applied by piping system supports during normal operating or shutdown conditions based on the critical transient or steady-state conditions. The dead weight of the piping and its contents are included. Dynamic load factors are used when applying transient loads, such as water hammer.

- Construction Loads—Construction loads are those loads to which the structure may be subjected during construction of the plant. Construction loads will be applied to evaluate partially completed structures, temporary structures, and their respective individual members. Design load requirements during construction for buildings and other structures will be developed in accordance with Standard SEI/ASCE 37-02. The magnitude and location of construction loads will be applied to generate the maximum load effects of dead, live, construction, and environmental loads. Consideration will be given to the loads and load effects of construction methods, equipment operation, and sequence of construction.

Severe Environmental Loads

Severe environmental loads are those loads that could be encountered infrequently during the life of the plant (GDC 2). The RB internal structures are protected by the RSB and the RCB; therefore, wind, earth pressure, or external flood loads do not apply. There are no OBE loads applicable to the overall design of the RB internal structures because an OBE level of one-third the SSE has been selected. See Section 3.7.1 for a description of the OBE. Severe environmental loads are not applicable to the design of RB internal structures.

Extreme Environmental Loads

Extreme environmental loads are those loads that are credible but are highly improbable (GDC 2). The RB internal structures are protected by the RSB and the RCB; therefore, tornado, hurricane, and external missile loads do not apply. This load category includes:

- Safe Shutdown Earthquake (E')—SSE loads are those loads generated by an earthquake with a peak horizontal ground acceleration of 0.30g. Seismic loads in the vertical direction and two orthogonal horizontal directions are considered to act simultaneously. Section 3.7 provides a description of how SSE loads are determined and combined. SSE loads are considered due to applied inertia loads, including dead loads, live loads, and hydrodynamic loads (i.e., water in storage pools and tanks), including combination of these loads using the square root of the sum of the squares (SRSS) method.

Abnormal Loads

Abnormal loads are those loads generated by a postulated high-energy pipe break causing a LOCA within a building or compartment (GDC 4 and GDC 50). This event is classified as a DBA. Included in this category are: Internal flooding loads (F_a), Pressure loads (P_a), Thermal loads (T_a), Accident pipe reaction loads (R_a), and Pipe break loads (R_r).

The Pipe break load is subcategorized as Pipe break reaction loads (R_{rr}), Pipe break jet impingement loads (R_{rj}), and Pipe break missile impact loads (R_{rm}). These loadings include a dynamic load factor to account for the dynamic nature of the load, unless a time-history analysis is performed to justify otherwise.

Abnormal loads include the following loads:

- Internal flood loads (F_a)—Loads resulting from internal flooding of containment during or following a postulated DBA.
- Pressure load (P_a)—Pressure equivalent static load within or across a compartment generated by the postulated pipe break and including a dynamic load factor to account for the dynamic nature of the load.
- Thermal load (T_a)—Thermal loads generated by the postulated pipe break and including T_o.
- Accident pipe reactions (R_a)—Pipe reactions generated by the postulated pipe break and including R_o.
- Pipe break loads (R_r)—Local equipment and piping loads generated following a postulated pipe break. Unless a time-history analysis is performed to justify otherwise, these loadings include a dynamic load factor to account for the dynamic nature of the load. The pipe break load (R_r) is considered to act as three separate components (R_{rr}, R_{rj}, R_{rm}), which are defined in the following paragraphs. In determining an appropriate equivalent static load for R_{rr}, R_{rj}, and R_{rm}, elasto-plastic behavior may be assumed with appropriate ductility ratios, provided excessive deflections do not result in the loss of function of any safety-related SSC.
  - Pipe break reaction loads (R_{rr})—R_{rr} is defined as the equivalent static load on the structure generated by the reaction of the high-energy pipe during the postulated break.
  - Pipe break jet impingement loads (R_{rj})—R_{rj} is defined as the jet impingement equivalent static load on the structure generated by the postulated break.

– Pipe break missile impact loads (R_{rm})—R_{rm} is defined as the missile impact equivalent static load on the structure generated by or during the postulated break, such as pipe whipping.

Other Loads

Other loads refer to postulated events or conditions that are not included in the design basis (GDC 4). These loading conditions and effects are evaluated without regard to the bounding conditions under which SSC perform design basis functions. This load category includes:

- Aircraft hazard (A)—Aircraft hazard refers to loads on a structure resulting from the impact of an aircraft. The evaluation of this loading condition is considered as part of the plant safeguards and security measures. There are no aircraft hazard loads on the RB internal structures since they are surrounded by other Seismic Category I structures that shield them from these loads.
- Explosion pressure wave (B)—Explosion pressure wave refers to loads on a structure resulting from an explosion in the vicinity of the structure. The evaluation of this loading condition is considered as part of the plant safeguards and security measures. There are no explosion pressure wave loads on the RB internal structures because they are surrounded by other Seismic Category I structures that shield them from these loads.
- Missile loads other than hurricane- or tornado-generated missiles—The RSB and the RCB protect the RB internal structures from impact of externally generated missiles. The RB internal concrete and steel structures are designed for internally generated missile loads as described in Section 3.5.

3.8.3.3.2 Load Combinations

*[Load combinations for design of RB internal structures are in accordance with ACI 349-2001 and guidelines of RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures]** (GDC 1, GDC 2, GDC 4, GDC 5 and GDC 50).

The NI Common Basemat Structure is a monolithic concrete structure. However, various portions of the structure have different classifications (i.e., RCB, RB internal structures, and other Seismic Category I structures) and correspondingly different design requirements, as shown in Figure 3.8-118. In some instances, the load combinations identified in ACI 349-2001 do not include certain independent loadings which should be considered to account for potential structure-to-structure effects (i.e., the effect on one structure resulting from loadings applied to a separate, but monolithically connected, structure). To account for potential structure-to-structure effects, the loading combinations from ACI 349-2001 are adjusted by including the necessary additional independent loadings. For concrete structures, the independent loadings added to the load combinations include buoyant force (F_b) and post-tension load (J). For steel structures, the independent loadings added to the load combinations include hydrostatic load (F), buoyant force (F_b), post-tension load (J), and soil load/lateral earth pressure (H). In load combinations where abnormal loads are considered, internal flood load (F_a) is added for both steel and concrete structures. The load factors for hydrostatic load (F), buoyant force (F_b), and post-tension load (J) are matched to that of the dead load (D) for each loading combination, while the load factors for soil load/lateral earth pressure (H) and internal flood load (F_a) are matched to that of the live load (L). Section 3.8.3.3.1 provides details regarding the loads considered for the design of the RB internal structures, while Section 3.8.1.3.1 provides the description of the post-tension load (J) which is included to account for the global effect of post-tension loads (J) on the NI Common Basemat.

The following definitions apply to load combinations for concrete and steel RB internal structures:

- *[For concrete members, U is defined as the section strength required to resist design loads based on the strength design methods described in ACI 349.*
- *For steel members, S is defined as the required section strength based on the elastic design methods and the allowable stresses defined in Part Q1 of ANSI/AISC N690.*
- *For steel members, Y is defined as the section strength required to resist design loads based on plastic design methods described in Part Q2 of ANSI/AISC N690.]**

Loads and loading combinations encompass the soil cases described in Section 2.5, using the design criteria described in Section 3.7.1 and Section 3.7.2.

Concrete Reactor Containment Building Internal Structures

The following load combinations define the design limits for Seismic Category I concrete RB internal structures.

- Normal load combinations (for strength design method):

  $U = 1.4\,(D + F + F_b + J) + 1.7\,(L + H + R_o)$

  $U = 1.05(D + F + F_b + J) + 1.3(L + H + R_o) + 1.2T_o$

  $U = 1.4(D + F + F_b + J) + 1.7(L + H + R_o + W)$

  $U = 1.05(D + F + F_b + J) + 1.3(L + H + R_o + W) + 1.2T_o$

- Factored load combinations (for strength design method):

  $U = D + L + H + F + F_b + T_o + R_o + J + E'$

  $U = D + L + H + F + F_b + J + F_a + 1.4P_a + T_a + R_a$

$U = D + L + H + F + F_b + J + E' + F_a + P_a + T_a + R_a + R_r$

$U = D + L + H + F + F_b + T_o + R_o + J + W_t$

$U = D + L + H + F + F_b + J + E' + F_a + P_a + T_a + R_a$

Steel Reactor Containment Building Internal Structures

The following load combinations define the design limits for Seismic Category I steel RB internal structures. For normal service load conditions, either the elastic working stress design methods of Section Q1 or the plastic design methods of Section Q2 of ANSI/AISC N690, including Supplement 2, are used. For factored load conditions, the elastic working stress design method is used.

- Service load combinations for elastic working stress design method:

  $S = D + L + H + F + F_b + J$

  $S = D + L + H + F + F_b + J + W$

  If thermal stresses due to T_o and R_o are present, the following load combination is also considered:

  $1.5S = D + L + H + F + F_b + T_o + R_{o + J}$

  $1.5S = D + L + H + F + F_b + T_o + R_{o + J + W}$

- Service load combinations for plastic design method:

  $Y = 1.7(D + L + H + F + F_b + J + W)$

  $Y = 1.7(D + L + H + F + F_b + J)$

  $Y = 1.3(D + L + H + F + F_b + T_o + R_{o + J})$

  $Y = 1.3(D + L + H + F + F_b + T_o + R_{o + J + W})$

- Factored load combinations for elastic working stress design method:

  $1.6S = D + L + H + F + F_b + T_o + R_o + J + W_t$

  $1.6S = D + L + H + F + F_b + T_o + R_o + J + E'$

  $1.6S = D + L + H + F + F_b + J + F_a + T_a + R_a + P_a$

  $1.7S = D + L + H + F + F_b + J + F_a + T_a + R_a + P_a + R_r + E'$

  $1.6S = D + L + H + F + F_b + J + F_a + T_a + P_a$

- Factored load combinations for plastic design method:

  $0.9Y = D + L + H + F + F_b + T_o + R_o + J + E'$

  $0.9Y = D + L + H + F + F_b + J + F_a + T_a + 1.25P_a + R_a$

  $0.9Y = D + L + H + F + F_b + J + F_a + T_a + P_a + R_a + R_r + E'$

  $0.9Y = D + L + H + F + F_b + T_o + R_o + J + W_t$

3.8.3.4 Design and Analysis Procedures

[Seismic Category I concrete structural elements and members are designed in accordance with ACI 349-2001 and its appendices (GDC 1). Exceptions to the code found in RG 1.142 are incorporated into the design and are accommodated in the loading combinations described in Section 3.8.3.3.2 for concrete structures.

Seismic Category I steel members and assemblies are designed in accordance with the requirements of ANSI/AISC N690-1994 (R2004) (GDC 1).

*Design of concrete embedments and anchors conforms to ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1) and guidelines of RG 1.199 (with exception described in Section 3.8.1.4.10).]**

Section 5.4.14 describes the applicable design and analysis procedures used for the design of steel portions of the NSSS component supports which interface with the RB internal structures concrete and steel embedments.

Computer modeling and classical manual techniques are used to analyze the RB internal structures by applying loads and loading combinations as described in Section 3.8.3.3. An overall computer model of the NI Common Basemat Structure is used which includes the RB internal structures. Local analyses are then performed for specific structural walls, slabs, and members to account for local effects of specific equipment loads, localized pipe break loads, hydrostatic and hydrodynamic loads, and other conditions (e.g., openings and local changes in member cross-sections). The results from the local analyses are combined with overall analysis results to produce the final analysis for the design of Seismic Category I concrete and steel elements and members.

The following sections describe specific techniques and criteria used for analysis and design of the RB internal structures.

Appendix 3E provides a description of specific analysis and design procedures for RB internal structures critical sections.

3.8.3.4.1 Overall Analysis and Design Procedures

The RB internal structures are included in the ANSYS V10.0 SP1 finite element overall computer model of the NI Common Basemat Structure that is described in Section 3.8.1.4. Boundary conditions for the ANSYS computer model and methods used for application of axisymmetric and non-axisymmetric loads, transient and localized loads, and other parameters used in the model are described in Section 3.8.1.4. The RB internal structures are modeled in combination with the overall NI Common Basemat Structure and basemat using a mesh of ANSYS finite elements. The FEM for the RB internal structures consists of the primary load-carrying walls, floors, columns, beams, and NSSS concrete equipment supports. Gaps are maintained between the internal structures and the containment wall to allow for structural movements during seismic events, containment pressurization, and other loading conditions. The RB internal structures are supported by the internal structures basemat. Section 3.8.1.4 provides a description of how the internal structures basemat interfaces with the containment liner above the NI Common Basemat Structure foundation basemat in the computer model. Section 3.8.5 describes modeling of the NI Common Basemat Structure foundation basemat.

ANSYS SHELL43 elements are used to model walls and floors and other concrete elements in the RB internal structures. SHELL43 is a three-dimensional, four-node shell element that is suitable for moderately thick shell structures. It can also provide out-of-plane shear forces and has an elastic-plastic capability. The steel liner causes discontinuity between the NI Common Basemat Structure foundation basemat and the base of the RB interior structure. This transition is modeled using multi-point constraints to allow sub-modeling of the interior structure and interface, as needed. For static modeling considerations, the individual companion nodes are coupled together. The strength of the liner is not relied upon to carry structural loadings.

Overall analysis of the RB internal structures considers the loads and loading combinations defined in Section 3.8.3.3. The following localized abnormal loads are not included in the overall analysis:

- Subcompartment pressure loads (P_a).
- Pipe break thermal loads (T_a).
- Accident pipe reactions (R_a).
- Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}).
- Local flood loads (F_a).

Local analyses address these localized loads. The overall analysis includes reaction loads from the RCS components (i.e., RV, SGs, RCPs, and PZR) due to LOCA pipe breaks (GDC 4 and GDC 50).

Figure 3.8-32—Reactor Building Internal Structures ANSYS Model, Figure 3.8-33—Reactor Building Internal Structures ANSYS Model – Section through Center of Building Looking West, Figure 3.8-34—Reactor Building Internal Structures ANSYS Model – Section through Reactor Cavity and Refueling Canal, Figure 3.8-35—Reactor Building Internal Structures ANSYS Model – Section through Center of Building Looking North, Figure 3.8-36—Reactor Building Internal Structures ANSYS Model – Section through Center of Building Looking Northwest, Figure 3.8-37—Reactor Building Internal Structures ANSYS Model – View of IRWST and Internal Structures Basemat show the FEM used for analysis of the RB internal structures. Additional descriptions of the RB internal structures computer model are provided in Appendix 3E.

Loads and load combinations defined in Section 3.8.3.3 are used to determine the strength requirements of members and elements. The following criteria apply for load combinations for concrete and steel RB internal structures:

- Where any load reduces the effects of other loads, the corresponding coefficient for that load is 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads.
- Where the structural effects of differential settlement, creep, or shrinkage may be significant, they are included with the dead load (D) as applicable.
- For load combinations in which a reduction of the maximum design live load (L) has the potential to produce higher member loads or stresses, multiple cases are considered where the live load (L) is varied between its maximum design value and zero.
- For load combinations including the loads P_a, T_a, R_a, R_{rr}, R_{rj}, or R_{rm}, the maximum values of these loads, including an appropriate dynamic load factor, are used unless a time-history analysis is performed to justify otherwise.
- For load combinations including loads R_{rr}, R_{rj}, and R_{rm}, the load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of intended function of the structural member or a loss of function of any safety-related SSC.

Concrete and steel structural elements and members are designed for axial tension and compression forces, bending moments, torsion, and in-plane and out-of-plane shear forces for the controlling loading combinations that are determined from the ANSYS computer analysis and local analyses. Internal structures behave within the elastic range under design basis loads. However, the ability of the structures to perform beyond yield is considered for abnormal loads associated with a pipe break, which results in rupture reactions, jet impingement and pipe whip, and for missile impact loads.

[The strength-design methods described in ACI 349-2001 and its appendices, including the exceptions detailed in RG 1.142, are used for the design of concrete walls, floors and other structural elements for RB internal structures (GDC 1). The ductility requirements of this code are satisfied so that a steel reinforcing failure mode controls over concrete failure modes. The recommendations of Appendix C of ACI 349-2001 are met for impulsive and impactive loading conditions (e.g., loading combinations that include pipe break missile impact loads).

Steel member and assembly design utilizes the allowable stress design methods of ANSI/AISC N690-1994 (R2004), including Supplement 2 (GDC 1). Steel items are maintained elastic for normal and extreme loadings in their respective combinations. Local yielding is permitted for abnormal loadings (e.g., pipe break accident loadings).

*A local analysis and design of concrete members will be performed for impactive and impulsive loads according to ACI 349, with exceptions noted in RG 1.142. A local analysis and design of steel members will be performed for impactive and impulsive loads according to ANSI/AISC N690.]**

It is acceptable to assume non-linear (elasto-plastic) response of structural members for evaluation of the response of reinforced concrete and steel structures subject to impactive or impulsive loads. Deformation under impactive and impulsive loads is controlled by limiting the ductility ratio, μ_d, which is defined as the ratio of maximum acceptable displacement, χ_m (or maximum strain, ε_m), to the displacement at the effective yield point, χ_y (or yield strain, ε_y), of the structural member. In addition to the specified deformation limits, maximum deformation will not result in the loss of intended function of the structural member nor impair the design basis safety function of other systems and components.

Regarding structural capacity, a structural member will retain its ability to perform its design basis function when ductility limits for concrete and steel members presented in Table 3.5-3 are satisfied. As deformation limits of the member may be governed by attached structures, systems and components (SSC), the member will also satisfy deformation limits imposed by attached SSC to prevent loss of design basis function.

3.8.3.4.2 Local Analysis and Design

Local analyses are performed for concrete and steel structural elements and members by using sub-models expanded from the overall analysis model and by using manual techniques, in combination with overall model analysis results. Sub-models are performed by refining the element mesh in the overall ANSYS model. Local discontinuities (e.g., openings, thickened areas, local loads, and changes in member cross-section) are included in the sub-models.

Local analysis and design consider the same member and element forces and moments as described for overall design. In addition, local effects (e.g., punching shear and transfer of anchorage loads to the structure) are considered. Local analyses also are used for design of secondary structures (e.g., platforms, equipment supports, crane supports).

[The recommendations of ACI 349-2001 and its appendices, including the exceptions in RG 1.142, are followed for concrete element and member local design (GDC 1).

Design of concrete embedments and anchors conforms to ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1) and guidelines of RG 1.199 (with exception described in Section 3.8.1.4.10).

ANSI/AISC N690-1994 (R2004), including Supplement 2, are followed for local steel member design (GDC 1).

*The design of bolted connections is in accordance with ANSI/AISC N690, Section Q1.16]** and AISC 348-00/2000 RCSC. Bolted in connections are fully tensioned, regardless of design methodology, unless justified otherwise.

The design of welded connections is in accordance with ANSI/AWS D1.1-2000 and ANSI/AWS D1.6-99, including January 6, 2005 update.

*[The design of bolted connections in combination with welded connections is in accordance with Section Q.15.10 of ANSI/AISC N690.]**

Openings in walls and slabs of RB internal structures are shown on construction drawings. Openings in slabs are acceptable without analysis if they meet the criteria identified in ACI 349, Section 13.4.2. Round pipe sleeves are used in lieu of rectangular penetrations, where possible. Corners of rectangular openings in walls or slabs are provided with diagonal reinforcing to reduce cracking due to stress concentrations at these locations in accordance with ACI 349, Section 14.3.7.

Appendix 3E provides a description of analysis and design results for critical areas of the RB internal structures.

Section 5.4.14 describes the design of interfacing steel assemblies which support the NSSS components and attach to, or interact with, embedments in the concrete. *[Steel supports for the RCS components and piping, including the base plates at the face of concrete structures, are designed in accordance with ASME Section III Division 1, Subsection NF.]* [Embedded portions of RCS component and pipe supports, which are beyond the jurisdictional boundary of the ASME Code, are designed in accordance* *with ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1), RG 1.199 (with exception described in Section 3.8.1.4.10), and also in accordance with ANSI/ AISC N690-1994 (R2004), including Supplement 2.]**

3.8.3.4.3 Static Analysis and Design

Dead loads (D), live loads (L), hydrostatic loads (F), pipe reactions (R_o), and normal thermal loads (T_o) are considered in the analysis and design of RB internal structures for the static normal load concrete and service load steel loading combinations. Normal thermal loads are considered as self-relieving for the overall RB internal structures. Concrete and steel members are designed to accommodate these static loads within the elastic range of their section strength.

Static fluid pressure loads are considered for design of the walls and floors of the IRWST and refueling canal. Moving loads are considered for mobile plant equipment (e.g., the polar crane, refueling machine, and other cranes and hoists).

3.8.3.4.4 Seismic and Other Dynamic Analyses and Design

Seismic analyses and designs of the RB internal structures conform to the procedures described in Section 3.7.2. Seismic accelerations are determined from the dynamic FEM described in Section 3.7.2. These accelerations are applied to the static FEM model of the RB internal structures as static-equivalent loads at the elevations used in the dynamic FEM.

Seismic SSE (E') loads are obtained by multiplying the dead load and 25 percent of the design live load by the structural acceleration obtained from the seismic analysis of the structure. Seismic loads are also considered due to the mass of fluids in tanks and canals as described herein (Section 3.8.3.4.4). Consideration is given to the amplification of these accelerations due to local flexibility of structural elements and members. Construction loads are not included when determining seismic loads. Other temporary loads are evaluated for contributing to the seismic loads on a case-by-case basis.

Seismic loads from the three components of the earthquake are combined using the SRSS method, where resultants are obtained using the following formulas:

$$P_R = +- \text{sqrt}(P_x^2 + P_y^2 + P_z^2)$$

$$M_R = +- \text{sqrt}(M_x^2 + M_y^2 + M_z^2)$$

The number of permutations for design are $2^n = 2^2$ = (++, --, +-, -+).

The effects of local flexibilities in floor slabs and wall panels are considered to determine if additional seismic accelerations should be applied to their design beyond those determined from the seismic stick model. Local flexibility evaluations are performed by determining the natural frequency of the floor or wall panel and comparing this to the frequency of the zero period acceleration on the applicable response spectra. Additional acceleration is applied when the natural frequency of the panel results in higher accelerations than the zero period acceleration. In cases where local flexibilities are determined to be a factor, additional out-of-plane accelerations are applied to the inertia loads on these panels for determining out-of-plane bending and shear loads.

Additional seismic loads due to accidental torsion are considered as described in Section 3.7.2. This is to account for variations in material densities, member sizes, architectural variations, equipment loads, and other variations from the values used in the analysis and design of the RB internal structures. Due to these potential variations, an additional eccentricity of the mass is included at the floor elevations and is equivalent to five percent of the maximum building dimension.

*[Seismic Category I concrete structural elements and their connections are detailed for ductility in accordance with ACI 349-2001, Chapter 21.]**

Structural Stiffness Considerations

Conservative values of concrete creep and shrinkage based on past experience are used in the design of the RB internal structures. Moments, forces, and shears are obtained on the basis of uncracked section properties in the analysis. However, in sizing the reinforcing steel, the concrete is not relied upon for resisting tension. Thermal moments are modified by cracked-section analysis using analytical techniques, when appropriate.

The effect of local wall and floor slab flexibility is included where necessary. The concrete section properties used in calculating the amplified seismic forces include an appropriate level cracking for the particular element under consideration. The amplified forces are also used in the design of the structural members that support the flexible element.

Section 3.8.3.6 describes methods used to confirm that concrete properties satisfy design requirements.

Seismic Structural Damping

Seismic analysis of RB internal structures uses the following SSE structural damping values recommended by RG 1.61.

Structure Type	Percent of Critical Damping
● Welded Steel	4

• Bolted Steel, Slip-Critical Connections	4
• Bolted Steel, Bearing Connections	7
• Reinforced Concrete	7

Hydrodynamic Load Analyses

Hydrodynamic loads are applied to the IRWST and refueling canal walls and floors to account for the impulsive and convective effects of water moving and sloshing in the tank as a result of seismic excitation. These loads are considered as part of the seismic SSE loads, and components of these loads in the three orthogonal directions are combined in the same manner as other seismic loads. Methodology consistent with USAEC TID-7024 is used to determine hydrodynamic loadings. The methodology and hydrodynamic loads are described in Section 3.8.4.4.1.

Polar Crane Seismic Analyses

Design of the RCB for seismic loads from the polar crane is performed with the crane in positions that result in maximum stresses on the supporting containment wall. See Section 3.8.1 for additional information on the design of the RCB.

For seismic load combinations, the polar crane design is based on the trolley being located in different positions along the bridge girders. Seismic evaluations are performed with and without the critical load raised to different positions for the trolley locations to determine which hook position produces the primary response of the crane. For analysis purposes, the critical load is defined as that of the reactor head. The design of the crane includes seismic restraints (up-kick lugs), which prevent the bridge and trolley from dislodging from their respective rails.

Refer to Section 9.1.5 for additional information on the polar crane.

Pipe Rupture Loads

Local analyses of the RB internal structures consider the following abnormal loads:

- Sub-compartment pressure loads (P_a).
- Pipe break thermal loads (T_a).
- Accident pipe reactions (R_a).
- Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}).
- Local flood loads (F_a).

These loads are applied to concrete and steel structures that enclose and support the RV, SGs, RCPs, PZR, RCS piping, MS and feedwater system piping, and other areas subject to abnormal loads.

Subcompartment pressure loads (P_a) are not applied to the overall ANSYS computer model because they do not result in global loadings on the RB internal structures. Subcompartment pressure loads are evaluated in local design of the concrete walls and floors for the applicable compartments. Subcompartment pressure loads resulting from a LOCA event are evaluated as time-dependent loads across concrete walls and floors that enclose the SGs, RCPs, PZR, and the RCS piping. Pipe breaks are not postulated in the reactor cavity. Concrete and steel members are designed to accommodate subcompartment pressure loads within the elastic range of the section strength.

Pipe break thermal loads (T_a) are considered in local analyses of concrete walls and floors. Accident thermal loads are evaluated as time-dependent loads across concrete walls and floors that enclose the SGs, RCPs, and the PZR. Concrete temperature is limited to 150°F for normal loading conditions. For short-term and accident thermal conditions, the concrete temperature is allowed to increase to 350°F for interior surfaces. Localized areas are allowed to reach 650°F from fluid jets in the event of a pipe failure. *[ACI 349-01/349-R01, Appendix A and ACI 349.1R-07 is the basis used for thermal design of concrete.]**

Accident pipe reaction loads (R_a) are considered on the NSSS equipment and piping supports, including supports for the RV, SGs, RCPs, PZR, and RCS piping. These loads are applied to the overall ANSYS computer model by applying worst-case LOCA loads to these component supports in separate load cases to determine overall effects on the RB internal structures (GDC 4 and GDC 50). Worst-case accident pipe reaction loads are further evaluated in local designs of the component supports in the critical sections described in Appendix 3E. Concrete and steel members are designed to accommodate accident pipe reaction loads within the elastic range of their section strength.

Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}) are not applied to the overall ANSYS computer model because they do not result in global loadings on the RB internal structures. These loads are considered in local design of concrete walls and floors and steel members. As defined in Section 3.8.3.3.1 under the definitions of abnormal loads, dynamic load factors are applied when analyzing structures for the static equivalent of these loads. Elasto-plastic behavior may be assumed with appropriate ductility ratios, provided that excessive deflections do not result in the loss of function of any safety-related SSC. *[Appendix C of ACI 349-2001 is used to determine pipe break reactions, jet impingement, and missile impact impulsive and impactive loads.]** The design of the RB internal structures for these loads conforms to the procedures described in Section 3.5 for internally generated missiles. Section 3.5 also describes ductility limits that are met for impactive and impulsive loadings.

Local flood loads (F_a) are applied to walls and floors of the RB internal structures in the overall ANSYS computer model. Concrete and steel members are designed to accommodate these flood loads within the elastic range of their section strength.

3.8.3.4.5 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.4, 2.5, 3.3, 3.5, 3.7, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections. A cross-reference between U.S. EPR FSAR sections and information required by SRP Section 3.8.4, Appendix C is provided in Table 3.8-17.

3.8.3.5 Structural Acceptance Criteria

[Limits for allowable stresses, strains, deformations, and other design criteria for reinforced concrete RB internal structures are in accordance with ACI 349-2001, and its appendices, including the exceptions specified in RG 1.142, The exceptions specified in RG 1.142 (GDC 1, GDC 2, GDC 4 and GDC 50) are considered.

Limits for allowable loads on concrete embedments and anchors are in accordance with ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1) and guidance given in RG 1.199 (with exception described in Section 3.8.1.4.10).

*Limits for the allowable stresses, strains, deformations and other design criteria for structural steel RB internal structures are in accordance with ANSI/AISC N690-1994, including Supplement 2 (GDC 1, GDC 2, GDC 4 and GDC 50).]**

Limits for allowable stresses, strains, and deformations on steel RCS component and pipe supports, including the base plates for these supports at the face of concrete structures, are in accordance with ASME Code, Section III, Division 1, Subsection NF.

The design of RB internal structures is generally controlled by load combinations containing SSE seismic loads. *[Stresses and strains are within the ACI 349-2001 and ANSI/AISC N690-1994 limits.]**

Appendix 3E provides design results for critical areas of the RB internal structures.

An as-built report is prepared to summarize deviations from the approved design and confirm that the as-built RB internal structures are capable of withstanding the design basis loads described in Section 3.8.3.3 without loss of structural integrity or safety-related functions.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control programs, and special construction techniques used in the fabrication and construction of concrete and steel internal structures of the RB internal structures (GDC 1).

3.8.3.6.1 Concrete Materials

*[Concrete materials for the RB internal structures conform to ACI 349-2001, Chapter 5, as supplemented by RG 1.142, and ACI 301-05 (GDC 1).]** Where required for radiation shielding, concrete conforms to RG 1.69.

Concrete Mix Design

Structural concrete used in the construction of the RB internal structures has a minimum compressive strength (i.e., f'_c) of 6000 psi at 90 days. The concrete density is between 140 pounds per cubic foot and 160 pounds per cubic foot. Poisson's ratio for the concrete is 0.17, unless otherwise justified.

Concrete mix design is determined based on field testing of trial mixtures with actual materials used.

Testing:

- Ultimate concrete strength, as well as early strength in support of an aggressive construction schedule.
- Concrete workability and consistency.
- Concrete admixtures.
- Heat of hydration and temperature control for large or thick concrete pours.
- Special exposure requirements when identified on design drawings.

Cement:

- Cement used for the concrete RB internal structures conforms to ASTM C150, ASTM C595 (excluding Types S and SA), or ASTM C845-04.
- Low-alkali cement, as defined in ASTM C150, is used in concrete with aggregates that are potentially reactive per ASTM C33.

Aggregates:

- *[Aggregates used for the RB internal structures conform to ACI 349-2001, Section 3.3.]**
- Aggregates conform to ASTM C33.

- ASTM Standards C1260 and C1293 (References 71 and 72) shall be used in testing aggregates for potential alkali-silica reactivity (ASR).

Admixtures:

- Air-entraining admixtures conform to ASTM C260.
- Chemical admixtures conform to ASTM C494 or ASTM C1017.
- Fly ash and other pozzolanic admixtures conform to ASTM C618.
- Grout fluidizers conform to ASTM C937.
- Ground-granulated blast furnace slag used as an admixture conform to ASTM C989.
- Silica fume used as an admixture conforms to ASTM C1240.
- Admixtures used in concrete mixtures containing ASTM C845 expansive cement are compatible with the cement and produce no deleterious effects.

Mix Water:

- *[Mix water used for the RB internal structures conforms to ACI 349-2001, Section 3.4.]**

Concrete Placement

Site-specific construction specifications address requirements and procedures for concrete placement. Construction specifications address the following:

- Desired volume of concrete pours and rate of deposition.
- Special forming requirements.
- Maximum height of pours.
- Temperature limitations; weather conditions and concrete mix, including methods for temperature control.
- Curing requirements and procedures.

Placement of concrete is performed with consideration given to the following codes:

- ACI 304R-00, Guide for Measuring, Mixing, Transporting, and Placing Concrete.
- ACI 305.1-06, Specification for Hot-Weather Concreting (Reference 9).
- ACI 306R-88 (Re-approved 2002), Cold-Weather Concreting.

- ACI 306.1-90 (Re-approved 2002), Standard Specification for Cold Weather Concreting.
- ACI 308R-01, Guide to Curing Concrete (Reference 52).
- ACI 308.1-98, Standard Specification for Curing Concrete.
- ACI 311.4R-05, Guide for Concrete Inspection.
- ACI 347-04, Guide to Formwork for Concrete.

3.8.3.6.2 Reinforcing Steel and Splice Materials

*[Reinforcing steel materials for the RB internal structures conform to ACI 349-2001 (GDC 1).]**

Materials

- *[Reinforcing steel used in the concrete RB internal structures conforms to ASTM A615 or ASTM A706 and the additional items specified in ACI 349-2001, Sections 3.5.1 through 3.5.4.]**
- Smooth wire for spiral reinforcement conforms to ASTM A82 (Reference 51).
- Welded plain wire fabric reinforcement conforms to ASTM A185 (Reference 52).
- Welded deformed wire fabric reinforcement conforms to ASTM A497 (Reference 53).
- Welded splices and mechanical splices of reinforcing bars are used.
- Materials used for bar-to-bar sleeves for mechanical cadweld-type rebar splices in the RB internal structures conform to ASTM A513, ASTM A519, or ASTM A576.
- *[Material for threaded and swaged reinforcement splices are determined by the manufacturer and are qualified in accordance with provisions of ACI 349-01, Section 12.14.3.]** These devices meet the provisions of Subarticle CC-4333 of the ASME Code, Section III, Division 2.

Fabrication and Placement

*[Fabrication and placement of reinforcing bars for RB internal structures is in accordance with ACI 349-2001, Chapter 7.]**

Welding conforms to the ASME Code, Section III, Division 2, Subsection CC, as supplemented by RG 1.136 and AWS D1.4-2005 (GDC 1).

*[Mechanical splices are subject to the testing and acceptance criteria of ACI 349-2001, Section 12.14.3.]**

3.8.3.6.3 Structural Steel

*[Structural steel materials for the RB internal structures conform to ANSI/AISC N690-1994 including Supplement 2 (2004)]** and AISC 303-00 (GDC 1).

Materials

*[Seismic Category I structural steel conforms to ASTM material specifications identified in ANSI/AISC N690, Section Q1.4.1.]** Materials for structural steel members include those listed in Table 3.8-8.

High strength bolting materials conform to ASTM A325 (Reference 54), or ASTM A490 (Reference 55). Other bolting materials conform to ASTM A307 (Reference 56).

*[Structural bolts conform to the ASTM material specifications identified in ANSI/AISC N690, Section Q1.4.3, or other materials identified in the AISC/RCSC.]** Bolting materials for structural steel include those listed in Table . Anchor rods conform to the material specifications in ASTM F1554 (Reference 46).

Structural bolts utilize nuts and washers as recommended by ASTM for the particular bolting material and as identified in AISC/RCSC. Structural bolting nut and washer materials for structural steel include those listed in Table 3.8-10—Structural Bolting Nut and Washer Materials.

*[Structural steel, steel pipe, or tubing used in composite compression members in Seismic Category I concrete structures conforms to the specifications in Section 3.5.6 of ACI 349-2001.]**

Welding materials conform to ANSI/AWS D1.1-2000, or ANSI/AWS D1.6-99, including the January 6, 2005 update, except as modified by ANSI/AISC N690, Sections Q1.17.1 and Q1.17.2.1. The compatibility of filler metal with base metal is specified in Table 3.1 of AWS D1.1.

Fabrication and Erection

Fabrication and erection of structural steel, welding, and bolting conforms to the following codes:

- *[ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004).]**
- AISC 348-00/2000 RCSC, Specification for Structural Joints Using ASTM A325 and A490 Bolts.
- ANSI/AWS D1.1-2000, Structural Welding Code – Steel.

- ANSI/AWS D1.6-1999, including January 6, 2005 update, Structural Welding Code – Stainless Steel.
- ANSI/AWS D1.8 2005, Structural Welding Code – Seismic Supplement.

3.8.3.6.4 Quality Control

In addition to the quality control procedures addressed in Section 3.8.3.6.1, Section 3.8.3.6.2, and Section 3.8.3.6.3, refer to Chapter 17 for a description of the quality assurance program for the U.S. EPR (GDC 1).

3.8.3.6.5 Special Construction Techniques

The RB internal structures are constructed using proven methods common to heavy industrial construction. Special, new, or unique construction techniques are not used.

Modular construction methods are used to the extent practical for prefabricating portions of the IRWST liner, refueling canal liner, reinforcing, concrete formwork, and other portions of the RB internal structures. Such methods have been used extensively in the construction industry. Rigging is pre-engineered for heavy lifts of modular sections. Permanent and temporary stiffeners are used on liner plate sections and other modularized items to satisfy code requirements for structural integrity of the modular sections during rigging operations.

Steel decking and plates and supporting steel beams may be used to form concrete floors. In these instances, the decking thickness is in addition to the floor thickness shown on the dimensional arrangement drawings, provided in Appendix 3B. The decking, plates, and beams may be left in place, in which case they are designed for applicable seismic loads and other loading conditions. Other types of formwork that may also be used is left in place and become a permanent part of the structure. Such items conform to code requirements and are designed to prevent their failure from affecting Seismic Category I SSC.

3.8.3.7 Testing and Inservice Inspection Requirements

Section 5.4.14 describes the tests and inspections for the RCS component supports.

Monitoring and maintenance of RB internal structures is performed in accordance with 10 CFR 50.65 and supplemented with the guidance in RG 1.160 (GDC 1).

Section 9.1.5 describes the tests and inspections for the polar crane. Physical access is provided to perform inservice inspections of the RB internal structures. Gaps are provided between the containment liner and concrete RB internal structures, which provide space necessary to inspect the liner at wall and floor locations inside containment

3.8.4 Other Seismic Category I Structures

3.8.4.1 Description of the Structures

Other Seismic Category I structures in the U.S. EPR include the following buildings and structures:

- Reactor Shield Building (RSB) and annulus – located on the Nuclear Island (NI) Common Basemat Structure foundation basemat.
- Fuel Building (FB) – located on the NI Common Basemat Structure foundation basemat.
- Safeguard Buildings (SB) 1, 2, 3, and 4 – located on the NI Common Basemat Structure foundation basemat.
- Vent Stack – supported on the roof slab of the Fuel Building.
- Emergency Power Generating Buildings (EPGB) 1 and 2, and 3 and 4 – two separate buildings.
- Essential Service Water Buildings (ESWB) 1, 2, 3, and 4 - the ESWB houses the Essential Service Water Cooling Towers (ESWCT) and the Essential Service Water Pump Buildings (ESWPB).
- Distribution system supports – included in the above structures.
- Platforms and miscellaneous structures – included in the above structures.

Figure 3B-1 provides a site plan of the U.S. EPR showing the location of these Seismic Category I buildings and structures.

Structures described within this section are not shared with any other power plant units (GDC 5).

Section 3.7.2 addresses design requirements for non-safety-related structures to preclude adverse interaction effects on Seismic Category I structures.

A combined license (COL) applicant that references the U.S. EPR design certification will describe any differences between the standard plant layout and design of Seismic Category I structures required for site-specific conditions.

A COL applicant that references the U.S. EPR design certification will address site-specific Seismic Category I structures that are not described in this section.

3.8.4.1.1 Reactor Shield Building and Annulus

The RSB is a heavily reinforced concrete structure comprised of a cylindrical wall and dome roof. The RSB is approximately 186 feet in diameter by 230 feet high, which completely encloses the Reactor Containment Building (RCB). The RSB protects the RCB from missiles and loadings resulting from external events (e.g., hurricanes, tornados, aircraft hazards, and explosion pressure waves). The RSB serves as an additional preventative barrier to the release of radiation or contamination in the event of accident conditions. The NI Common Basemat Structure foundation basemat supports the RSB.

The RSB is surrounded by SBs 1, 2, 3, 4, and by the FB, which are Seismic Category I safety-related structures. The walls and slabs of SBs 1 and 4 frame into the RSB cylindrical wall for support. The roofs and external walls of SBs 2 and 3 and the FB frame into the RSB wall for support; however, the interior walls and floors of these buildings are separated from the RSB to isolate the interior portions of the structures in the event of an aircraft hazard or blast loading event. Where they are enclosed within the SBs and the FB, the lower portions of the RSB cylindrical wall are approximately 4 feet, 3 inches thick. The RSB cylindrical wall and dome that are exposed to the environment above the roofs of the adjacent SBs and FBs are approximately 5 feet, 11 inches thick.

The Reactor Building (RB) annulus is the space between the RSB and the RCB. The annular space is approximately 5 feet, 11 inches wide between the faces of the concrete walls of the two buildings. The RB annulus is an area that provides access for personnel to inspect the outside of the RCB, and to route piping, ventilation ducts, electrical cables, and other items. A slight negative pressure is maintained in the annulus to facilitate the secondary function of the RSB as a barrier to the release of contamination.

Figure 3.8-3—Reactor Building Plan at Elevation -8 Feet (top of concrete at start of containment wall), Figure 3.8-4—Reactor Building Plan at Elevation +5 Feet (top of heavy floor for nuclear steam supply system (NSSS) component support), Figure 3.8-5—Reactor Building Plan at Elevation +17 feet (plan at centerline of reactor vessel piping nozzles), Figure 3.8-6—Reactor Building Plan at Elevation +29 feet (top of grating floor for component access), Figure 3.8-7—Reactor Building Plan at Elevation +45 feet (top of grating floor for component access), Figure 3.8-8—Reactor Building Plan at Elevation +64 feet (top of concrete operating floor), Figure 3.8-9—Reactor Building Plan at Elevation +79 feet (top of partial concrete floor), Figure 3.8-10—Reactor Building Plan at Elevation +94 feet (top of pressurizer cubicle), Figure 3.8-11—Reactor Building Section A-A, Figure 3.8-12—Reactor Building Section B-B, and Figure 3.8-13—Reactor Building Section C-C show the arrangements of the RSB and annulus.

3.8.4.1.2 Fuel Building

The FB is a reinforced concrete structure that extends approximately 58 feet out from the RSB wall by 160 feet long by 140 feet high. The FB is located on the side of the

RSB that is opposite of SBs 2 and 3. Hardening of the exterior walls and roof of the FB protects it against external events (e.g., hurricane and tornado missiles, aircraft hazard and blast loadings). Dual exterior walls are provided from the foundation up to the ceiling, except on either end of the FB where the stair towers are located, to isolate interior structures from the exterior walls in order to mitigate the effects of external events.

The FB houses various structures, systems, and components (SSC) related to fuel storage and handling operations, including storage areas for new and spent fuel, the fuel pool cooling system, and handling systems that allow the placement of spent fuel into casks for storage. The operating floor level of the FB provides access to the RCB equipment hatch. Stair towers on either end of the FB connect the FB to SBs 1 and 4 and provide protection to the lower portion of the RSB wall. The NI Common Basemat Structure foundation basemat supports the FB.

A large pool in the FB stores and handles new and spent reactor fuel. The fuel transfer tube through the RCB shell wall connects the spent fuel pool to the RB internal structures refueling cavity. The spent fuel pool is lined with stainless steel plate. Section 9.1.2 addresses fuel storage racks and leak chase collection and monitoring. Section 9.1.5 addresses cranes in the FB.

The vent stack is a steel structure approximately 12 feet, 6 inches in diameter by 100 feet high located on top of the Fuel Building roof as shown on Figure 3B-1. The vent stack is classified as Seismic Category I and serves as the exhaust for the Nuclear Auxiliary Building (NAB). The effects of loadings from the vent stack are considered in the design of the FB.

Figure 3.8-38—Fuel Building Plan Elevation -31 Feet, Figure 3.8-39—Fuel Building Plan Elevation -20 Feet, Figure 3.8-40—Fuel Building Plan Elevation -11 Feet, Figure 3.8-41—Fuel Building Plan Elevation 0 Feet, Figure 3.8-42—Fuel Building Plan Elevation +12 Feet, Figure 3.8-43—Fuel Building Plan Elevation +24 Feet, Figure 3.8-44—Fuel Building Plan Elevation +36 Feet, Figure 3.8-45—Fuel Building Plan Elevation +49 Feet, Figure 3.8-46—Fuel Building Plan Elevation +64 Feet, Figure 3.8-47—Fuel Building Plan Elevation +79 Feet, Figure 3.8-48—Fuel Building Plan Elevation +90 Feet, Figure 3.8-49—Fuel Building Plan Elevation +112 Feet, Figure 3.8-50—Fuel Building Plan Section A-A, Figure 3.8-51—Fuel Building Plan Section B-B, and Figure 3.8-52—Fuel Building Plan Section C-C show the arrangements of the FB.

3.8.4.1.3 Safeguard Buildings

The SBs 1, 2, 3, and 4 are reinforced concrete structures located around approximately three-fourths of the periphery of the RSB. Stair towers provide personnel access among the various elevations of the NI Common Basemat Structure. The RCB airlocks are accessible from the SBs.

The SBs house four divisions (Divisions 1 through 4) that are contained within three separate structures. Divisions 1 and 4 are housed in SBs 1 and 4 located on opposite sides of the RSB. A common wall in a third structure separates SBs 2 and 3 Divisions 2 and 3. Each division of the SBs contains a redundant safety system train. The lower levels of the SBs house mechanical systems, while the upper levels contain electrical, instrumentation, controls, and heating, ventilation, and air conditioning (HVAC) systems. Emergency feedwater storage tanks are provided in the SBs, which are lined with stainless steel to prevent leakage. Cable, pipe, and duct shafts are located within the SBs for routing distribution systems between the various elevations of the buildings. These shafts are constructed of reinforced concrete and steel. The main control room (MCR) is located [

]

Physical separation of the three SB structures and additional hardening of the buildings 2 and 3 structure protects against damage to multiple divisions in the case of external events (e.g., hurricane and tornado missiles, aircraft hazard or blast loadings). SBs 2 and 3 are hardened by providing a dual roof and exterior walls, thickening the roof slab, and decoupling interior walls and slabs from the exterior walls and roof. The combined structure for buildings 2 and 3 extends approximately 92 feet out from the RSB wall by 180 feet long by 140 feet high. SB 1 has overall dimensions of approximately 87 feet out from the RSB wall by 100 feet long by 115 feet high. SB 4 has dimensions of approximately 87 feet out from the RSB wall by 100 feet long by 150 feet high. The NI Common Basemat Structure foundation basemat supports the SBs.

The main steam (MS) and feedwater valve rooms are comprised of reinforced concrete compartments located within the SBs. Divisions 1 and 2 of the valve rooms are located in SB 1, while Divisions 3 and 4 of the valve rooms are located in SB 4. This arrangement results in a two-by-two redundancy. The physical separation of the valve rooms results in at least two valve rooms remaining unaffected in the case of external events (e.g., aircraft hazard). The reinforced concrete walls protect the individual valve rooms against internal hazards.

Figure 3.8-53—Safeguard Building 1 Plan Elevation -31 Feet, Figure 3.8-54—Safeguard Building 1 Plan Elevation -16 Feet, Figure 3.8-55—Safeguard Building 1 Plan Elevation 0 Feet, Figure 3.8-56—Safeguard Building 1 Plan Elevation +15 Feet, Figure 3.8-57—Safeguard Building 1 Plan Elevation +27 Feet, Figure 3.8-58—Safeguard Building 1 Plan Elevation +39 Feet, Figure 3.8-59—Safeguard Building 1 Plan Elevation +55 Feet, Figure 3.8-60—Safeguard Building 1 Plan Elevation +69 Feet, Figure 3.8-61—Safeguard Building 1 Plan Elevation +81 Feet, Figure 3.8-62—Safeguard Building 1 Plan Elevation +96 Feet, and Figure 3.8-63—Safeguard Building 1 Section A-A show the arrangements of SB 1.

Figure 3.8-64—Safeguard Buildings 2 and 3 Plan Elevation -31 Feet, Figure 3.8-65—Safeguard Buildings 2 and 3 Plan Elevation -16 Feet, Figure 3.8-66—Safeguard Buildings 2 and 3 Plan Elevation 0 Feet, Figure 3.8-67—Safeguard Buildings 2 and 3 Plan Elevation +15 Feet, Figure 3.8-68—Safeguard Buildings 2 and 3 Plan Elevation +27 Feet, Figure 3.8-69—Safeguard Buildings 2 and 3 Plan Elevation +39 Feet, Figure 3.8-70—Safeguard Buildings 2 and 3 Plan Elevation +53 Feet, Figure 3.8-71—Safeguard Buildings 2 and 3 Plan Elevation +69 Feet, Figure 3.8-72—Safeguard Buildings 2 and 3 Plan Elevation +79 Feet, Figure 3.8-73—Safeguard Buildings 2 and 3 Plan Elevation +94 Feet, Figure 3.8-74—Safeguard Buildings 2 and 3 Section B-B show the arrangements of SBs 2 and 3.

Figure 3.8-75—Safeguard Building 4 Plan Elevation -31 Feet, Figure 3.8-76—Safeguard Building 4 Plan Elevation -16 Feet, Figure 3.8-77—Safeguard Building 4 Plan Elevation 0 Feet, Figure 3.8-78—Safeguard Building 4 Plan Elevation +15 Feet, Figure 3.8-79—Safeguard Building 4 Plan Elevation +26 Feet, Figure 3.8-80—Safeguard Building 4 Plan Elevation +39 Feet, Figure 3.8-81—Safeguard Building 4 Plan Elevation +55 Feet, Figure 3.8-82—Safeguard Building 4 Plan Elevation +69 Feet, Figure 3.8-83—Safeguard Building 4 Plan Elevation +81 Feet, Figure 3.8-84—Safeguard Building 4 Plan Elevation +96 Feet, and Figure 3.8-85—Safeguard Building 4 Section A-A show the arrangements of SB 4.

3.8.4.1.4 Emergency Power Generating Buildings

The EPGB 1 and 2 are housed in one building while EPGB 3 and 4 are housed in a separate building. These two buildings are identified hereafter as EPGBs.

The two EPGBs are located adjacent to the NI Common Basemat Structure and in the general vicinity of the ESWBs. As depicted in Figure 3B-1, each building is physically separated from the NI Common Basemat Structure and is located on the opposite sides to provide sufficient separation to protect against common external events (e.g., aircraft hazard).

The EPGBs are essentially identical but are mirror images of one another. Each EPGB is approximately 178 feet long by 95 feet wide. The height of the EPGBs varies from approximately 51 feet, 6 inches above the top of the basemat foundation in the areas of the diesel fuel storage tanks, to 68 feet for the remainder of the structure.

Each EPGB is primarily constructed of reinforced concrete and supported by its own independent reinforced concrete basemat foundation. Structural steel framing is limited to steel platforms and composite beams.

Each EPGB contains two main diesel generators, the supporting equipment and also contains two fuel storage tanks, HVAC equipment, electrical equipment, and batteries. Within each structure, reinforced concrete walls separate the two main diesels and the diesel fuel storage tanks to protect against internal hazards. External walls and slabs are sized to protect against external hazards (e.g., wind, missile and explosion pressure wave).

Figure 3.8-89—Emergency Power Generating Buildings Plan Elevation 0'-0", Figure 3.8-90—Emergency Power Generating Buildings Plan Elevation 33'-4", Figure 3.8-91—Emergency Power Generating Buildings Plan Elevation 51'-6", Figure 3.8-92—Emergency Power Generating Buildings Plan Elevation 68'-0", Figure 3.8-93—Emergency Power Generating Buildings Section A-A, and Figure 3.8-94—Emergency Power Generating Buildings Section B-B provide the elevation and section views of the EPGBs.

3.8.4.1.5 Essential Service Water Buildings

The ESWBs house the ESWCTs and the ESWPBs. The function of the ESWBs is to house equipment and cooling water associated with the essential service water system (ESWS). This system provides a source of cooling water to the component cooling water system (CCWS) heat exchangers, the Emergency Power Generator heat exchangers, and Essential Service Water HVAC system to support the safe operation and orderly shutdown of the plant, during normal operation or under accident conditions. As depicted in Figure 3B-1 each of the four structures is located in the vicinity of the NI Common Basemat Structure, but ESWBs 1 and 2 are physically separated from ESWBs 3 and 4 by the NI Common Basemat Structure to provide sufficient protection against external events (e.g., aircraft hazard).

Each ESWB is a reinforced concrete, shear wall structure approximately 164 feet by 108 feet wide by 118 feet high (i.e., from the bottom of the basemat to elevation 96 feet). Each structure is embedded 21 feet below grade. The primary portion of the structure is approximately 128 feet long by 108 feet wide, and houses two cooling towers, each with a water storage basin. On the side of the cooling towers facing the containment building, a structurally integrated pump house structure is located, enclosing primarily pumps and electrical equipment. The ESWPB is approximately 35 feet by 64 feet, with a roof at elevation 63 feet.

Exterior walls and slabs are sized for protection against external hazards, including tornado and hurricane-generated missiles and postulated blast loads. Two compartments are provided for air draft between elevation 14 feet and 43 feet, 6 inches.

Figure 3.8-95—Essential Service Water Building Plan Elevation 0'-0", Figure 3.8-96—Essential Service Water Building Plan Elevation 14'-0", Figure 3.8-97—Essential Service Water Building Plan Elevation 47'-0", Figure 3.8-98—Essential Service Water Building Plan Elevation 63'-0", Figure 3.8-99—Essential Service Water Building Plan Elevation 80'-0", Figure 3.8-100—Essential Service Water Building Roof Plan Elevation 96'-0", Figure 3.8-101—Essential Service Water Building Section A-A, and

Figure 3.8-102—Essential Service Water Building Section B-B provide the elevation and section views of the ESWBs.

3.8.4.1.6 Distribution System Supports

Structural steel supports are provided for Seismic Category I distribution systems as part of other Seismic Category I structures. These include pipe supports, equipment supports, cable tray and conduit supports, HVAC duct supports, and other component supports. Distribution system supports are primarily constructed of steel shapes and tubing, which are anchored to other Seismic Category I concrete structures using embedded steel plates, cast-in-place anchor bolts, or drilled-in concrete anchors.

3.8.4.1.7 Platforms and Miscellaneous Structures

Platforms and miscellaneous structures (e.g., ladders, guard rails, stairs) are provided for access and maintenance to plant equipment and components housed in other Seismic Category I structures. These items are primarily constructed of steel beams, angles, channels, tubing, and grating. Platforms and miscellaneous structures are Seismic Category I, Seismic Category II, or Conventional Seismic depending on their safety function and potential interaction of the items with Seismic Category I SSC.

3.8.4.1.8 Buried Conduit and Duct Banks

The design of buried conduit and duct banks is site-specific. The design criteria for safety-related buried conduit and duct banks are provided below and in Section 3.8.4.4:

Safety-related conduit located outside of the building envelope is installed Seismic Category I and buried individually, as multiple conduits or in assemblies known as duct banks. [[Buried conduits are steel while conduits in encased duct banks may be poly-vinyl-chloride (PVC) or steel. Duct banks may be directly buried in the soil; encased in lean concrete, concrete, or reinforced concrete. Concrete or reinforced concrete encased duct banks will be used in heavy haul zones, under roadway crossings, or where seismic effects dictate the requirement. Encasement in lean concrete may be used in areas not subject to trenching or passage of heavy haul equipment, or where seismic effects on the conduit are not significant.]] Duct bank depth and encasement methods will also consider effects from external hazards (e.g. hurricane and tornado missiles).

The analysis of duct banks considers the type of loading imposed on the duct bank (seismic wave passage load, static surcharge, buoyancy, settlement, hurricane and tornado missiles), soil properties, the geometry of the duct bank (curved versus straight), and boundary conditions imposed on the ends of the duct bank. Reinforced concrete encasement for duct banks used in heavy haul routes or road crossings are evaluated for postulated loadings and provisions defined in Section 3.8.4.4.

A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried conduit and duct banks.

3.8.4.1.9 Buried Pipe and Pipe Ducts

The design of buried pipe and pipe ducts is site-specific. Buried pipe and pipe ducts are either directly buried in the soil or placed in reinforced concrete structures, or other materials, that are constructed below grade for routing Seismic Category I pipes between structures. Section 3.8.4.4 provides the design criteria for Buried Pipes and Pipe Ducts.

A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried pipe and pipe ducts.

3.8.4.1.10 Masonry Walls

No masonry walls are used in Seismic Category I structures in the U.S. EPR.

3.8.4.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used for the design, fabrication, construction, testing, and inservice inspection of Seismic Category I structures other than the RCB and RB internal structures (GDC 1, GDC 2, GDC 4, and GDC 5).

3.8.4.2.1 Codes and Standards

- ACI 301-05 - Specifications for Structural Concrete for Buildings.
- ACI 304R-00 - Guide for Measuring, Mixing, Transporting, and Placing Concrete.
- ACI 305.1-06 - Hot-Weather Concreting.
- ACI 306R-88 (Re-approved 2002) - Cold-Weather Concreting.
- ACI 306.1-90 (Re-approved 2002) - Standard Specification for Cold Weather Concreting.
- ACI 308R-01 - Guide to Curing Concrete.
- ACI 308.1-98 - Standard Specification for Curing Concrete.
- ACI 311.4R-05 - Guide for Concrete Inspection (Reference 40).
- ACI 347-04 - Guide to Formwork for Concrete.
- ACI 349-01/349-R01 - Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (exception described in 3.8.4.4 and 3.8.4.5) (GDC 1).

- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of Condition A strength reduction factors even when supplemental reinforcement is provided (Reference 63).
- ACI 349.1R-07 - Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures.
- ACI 350-06 - Code Requirements for Environmental Engineering Concrete Structure (Reference 58).
- ACI 350.3-06 - Seismic Design of Liquid-Containing Concrete Structures (Reference 59).
- AISC 303-00 - Code of Standard Practice for Steel Buildings and Bridges.
- ANSI/AISC N690-1994 - Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).
- ANSI/ANS-6.4-2006 - Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants (Reference 4).
- AISC 348-00/2000 RCSC - Specification for Structural Joints Using ASTM A325 and A490 Bolts.
- ANSI/AWS D1.1-2000 - Structural Welding Code – Steel.
- ANSI/AWS D1.4-2005 - Structural Welding Code - Reinforcing Steel.
- ANSI/AWS D1.6-99, including January 6, 2005 update - Structural Welding Code – Stainless Steel.
- ANSI/AWS D1.8 2005 - Structural Welding Code – Seismic Supplement.
- ASME Code, Section III, Division 2 – Code for Concrete Reactor Vessels and Containments (Reference 1).
- ASME NOG-1-2004 - Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girders).
- ASME B31.3 - 1996 - Process Piping, American Society of Mechanical Engineers (Reference 60).
- ASME B31.4 - 1992 - Liquid Transportation System for Hydrocarbon, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols (Reference 61).
- ASME B31.8 - 1995 - Gas Transportation and Distribution Piping Systems.

3.8.4.2.2 Specifications

Industry standards (e.g., those published by the ASTM) are used to specify material properties, testing procedures, fabrication methods, and construction methods.

Structural specifications cover areas related to the design and construction of other Seismic Category I structures. These specifications emphasize important points of the industry standards for these structures and reduce options that would otherwise be permitted by the industry standards. These specifications cover the following areas:

- Concrete material properties.
- Mixing, placing, and curing of concrete.
- Reinforcing steel and splices.
- Structural steel.
- Steel liner plate and embedments.
- Miscellaneous and embedded steel.
- Anchor bolts.
- Expansion anchors.
- Cranes and hoists.

3.8.4.2.3 Design Criteria

- ACI 349-01/349-R01 - Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (GDC 1).
- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of Condition A strength reduction factors even when supplemental reinforcement is provided (Reference 63).
- ANSI/AISC N690-1994 - Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).

3.8.4.2.4 Regulations

- 10 CFR 50, Appendix A - General Design Criteria for Nuclear Power Plants, GDC 1, GDC 2, GDC 4, and GDC 5.
- 10 CFR 50, Appendix B - Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants."

- 10 CFR 50, Appendix S - Earthquake Engineering Criteria for Nuclear Power Plants.

3.8.4.2.5 NRC Regulatory Guides

Regulatory Guides applicable to the design and construction of other Seismic Category I structures:

- RG 1.61, Revision 1, March 2007 (exception described in 3.7.1).
- RG 1.69, December 1973.
- RG 1.115, Revision 1, July 1977.
- RG 1.142, Revision 2, November 2001 (exception described in 3.8.3.3).
- RG 1.160, Revision 2, March 1997.
- RG 1.199, November 2003 (exception described in Section 3.8.1.4).

3.8.4.3 Loads and Load Combinations

The U.S. EPR design loads envelope includes the loads over a broad range of site conditions. The loads on other Seismic Category I structures are separated into the following categories:

- Normal loads.
- Severe environmental loads.
- Extreme environmental loads.
- Abnormal loads.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for other Seismic Category I structures, or perform additional analyses to verify structural adequacy.

3.8.4.3.1 Design Loads

*[Loads on other Seismic Category I structures are in accordance with ACI 349-2001 and RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures]**(GDC 1, GDC 2, GDC 4, and GDC 5).

Other Seismic Category I structures are designed for the following loads, as described in Section 3.8.4.4:

Normal Loads

Normal loads are those loads encountered during normal plant operation, startup, shutdown, and construction (GDC 4). This load category includes:

- Dead loads (D)—Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.

  For buried items, the dead load includes the weight of the soil overburden. The soil overburden load includes the weight of the overlying soil prism.

- Live loads (L)—Live loads include any normal loads that vary with intensity and point of application, including moveable equipment and precipitation loads. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, varied from zero to full value, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied for the loading condition.

  In general, a live load of 500 pounds per square foot is applied to FB concrete floors and a load of 175 pounds per square foot is applied to FB and SB steel grating floors and platforms. A live load of 300 pounds per square foot is applied to SB concrete floors. Finally, a live load of 100 pounds per square foot is applied to concrete floors, steel grating floors, and platforms in other Seismic Category I structures. Floor live loads may vary according to the function of individual floors. Truck loads, fuel cask shipment loads, and loads due to replacement of RCS components are considered as live loads in the loading and material handling bays of the FB. Live loads are applied to cranes and their supports for the lifting capacity and test loads applied for lifting devices. Additional point loads are applied to concrete floors and to concrete and steel beams in local design.

  The design live load for rainfall is based on a rate of 19.4 inches per hour, as described in Section 2.4.

  The design live load due to rain, snow, and ice is based on a ground load of 143 pounds per square foot, which corresponds to a roof load of 100 pounds per square foot, as described in Section 2.3. This value is postulated as a meteorological site parameter for the extreme winter precipitation load and includes the weight of the normal winter precipitation event and the weight of the extreme winter precipitation event. Roof snow and ice loads are determined using Chapter 7 of ASCE/SEI 7-05, "Minimum Design Loads for Buildings and Other Structures." The following factors are used to compute snow loads on the roofs of Seismic Category I structures:

  - C_e, exposure factor = 1.0.

  - C_t, thermal factor = 1.0.

  - I, importance factor = 1.00 for Seismic Category I structures.

For buried items, the live load includes the effects of surface traffic such as truck loads, rail loads, construction equipment, and construction or maintenance activities.

- Hydrostatic loads (F)—Hydrostatic loads are due to fluids stored in pools and tanks in other Seismic Category I structures (e.g., the spent fuel pool and emergency feedwater storage tanks). Pools and tanks may have either constant or fluctuating liquid levels. Hydrodynamic loads resulting from seismic excitation of fluids are included as a component of the safe shutdown earthquake (SSE) load.

- Thermal loads (T_o)—Thermal loads consist of thermally induced forces and moments resulting from normal plant operation and environmental conditions. Thermal loads and their effects are based on the critical transient or steady-state condition. Thermal expansion loads due to axial restraint, as well as loads resulting from thermal gradients, are considered.

  The external ambient air temperatures for other Seismic Category I structures are as follows:

  – Maximum outside air temperature: 115°F.

  – Minimum outside air temperature: -40°F.

  – Ground temperature: 50°F.

  Internal ambient temperatures for other Seismic Category I structures, except the FB, are as follows:

  – During normal operation: 100°F (maximum).

  – During normal operation: 50°F (minimum).

  FB internal ambient temperatures are as follows:

  – During normal operation: 110°F (maximum).

  – During normal operation: 50°F (minimum).

  – During abnormal conditions: 200°F (maximum).

  Spent fuel pool fluid temperatures are as follows:

  – During normal operation: 150°F (maximum).

  – During normal operation: 70°F (minimum).

  – During abnormal conditions: 180°F (maximum).

- Pipe reactions (R_o)—Pipe reactions are those loads applied by piping system supports during normal operating or shutdown conditions based on the critical transient or steady state conditions. The dead weight of the piping and its contents are included. Dynamic load factors are used when applying transient loads, such as water hammers.

- Soil loads and lateral earth pressure (H)—Soil loads and lateral earth pressure are loads that result from soil bearing pressures applied to buried exterior walls and structures up to the finished grade elevation of the surrounding soil. Refer to Section 2.5.4.2 for the soil parameters used to determine soil loads and lateral earth pressure. Normal soil loads consider saturated soil up to a groundwater elevation of -3.3 feet relative to the site finished grade.

- Construction loads—Construction loads are those loads to which the structure may be subjected during construction of the plant. Construction loads will be applied to evaluate partially-completed structures, temporary structures, and their respective individual members. Design load requirements during construction for buildings and other structures will be developed in accordance with Standard SEI/ASCE 37-02, "Design Loads on Structures During Construction." The magnitude and location of construction loads will be applied to generate the maximum load effects of dead, live, construction, environmental, and lateral earth pressure loads. Consideration will be given to the loads and load effects of construction methods, equipment operation, and sequence of construction.

Severe Environmental Loads

Severe environmental loads are those loads that could be encountered infrequently during the plant life (GDC 2). This load category includes:

- Wind loads (W)—Wind loads are those loads resulting from wind pressure acting on external surfaces of structures due to normal design wind speeds. See Section 3.3.1 for wind parameters and methods used to determine wind loads. Wind loads in this category do not include hurricane and tornado wind forces.

- Operating basis earthquake (OBE)—There are no OBE loads applicable to the design of other Seismic Category I structures, since an OBE level of one-third the SSE has been selected. See Section 3.7 for a description of the OBE.

Extreme Environmental Loads

Extreme environmental loads are those loads that are credible but are highly improbable (GDC 2). This load category includes:

- Safe shutdown earthquake (E')—SSE loads are those loads generated by an earthquake with a peak horizontal ground acceleration of 0.30 g. Seismic loads in the vertical direction and two orthogonal horizontal directions are considered to act simultaneously. Section 3.7 provides a description of how SSE loads are determined and combined. SSE loads are considered due to applied inertia loads, including dead loads, live loads, hydrodynamic loads (i.e., water in storage pools and tanks), and soil loads, including combination of these loads using the square root of the sum of the squares (SRSS) method.

The SSE component of soil loads is determined using densities for saturated soil to account for the weight of the soil plus the weight of either normal or flood water levels. This includes using load cases for normal groundwater level at 3.3 feet below plant grade, and for flood water level at 1.0 foot below plant grade. Earthquake-induced soil pressures are developed in accordance with Section 3.5.3 of ASCE 4-98.

- Tornado and Hurricane loads (W_t)—Tornado and Hurricane loads are those loads on external surfaces of structures resulting from a design basis tornado and hurricane. See Section 3.3.2 for tornado and hurricane design parameters and methods used to determine tornado and hurricane loads. See Section 3.5 for design methods and parameters used to determine hurricane- or tornado-generated missile loads. Tornado or hurricane loads include:
  - Tornado or hurricane wind pressure (W_w).
  - Tornado differential pressure (W_p) = 0 for hurricane.
  - Hurricane- or Tornado-generated missiles (W_m).
- External flood loads—External flood loads are included with soil loads and lateral earth pressure loads (H) and with SSE loads (E') as previously described by considering saturated soil conditions.

Abnormal Loads

Abnormal loads are those loads generated by a postulated high-energy pipe break accident (i.e., loss of coolant accident (LOCA)) within a building or compartment (GDC 4). This event is classified as a design basis accident. Included in this category are: Internal Flooding loads (F_a), Pressure loads (P_a), Thermal loads (T_a), Accident pipe reaction loads (R_a), and Pipe break loads (R_r). The Pipe break load is subcategorized as: Pipe break reaction loads (R_{rr}), Pipe break jet impingement loads (R_{rj}), and Pipe Break Missile Impact loads (R_{rm}). These loadings include a dynamic load factor to account for the dynamic nature of the load, unless a time-history analysis is performed to justify otherwise. Abnormal loads include the following loads:

- Internal flood loads (F_a)—Loads resulting from internal flooding of other Seismic Category I structures during or following a postulated pipe system failure that presents the risk of common mode failures of safety-related equipment (e.g., failures of cooling water systems in SBs 1 through 4). Hydrostatic loads from the maximum possible water level are applied to affected walls, slabs, and the basemat foundation.
- Pressure load (P_a)—Pressure equivalent static load within or across a compartment or building generated by the postulated break and including a dynamic load factor to account for the dynamic nature of the load.

- Thermal load (T_a)—Thermal loads generated by the postulated break and including T_o.

- Accident pipe reactions (R_a)—Pipe reactions generated by the postulated break and including R_o.

- Pipe break loads (R_r)—Local equipment and piping loads generated following a postulated pipe break. Unless a time-history analysis is performed to justify otherwise, these loadings include a dynamic load factor to account for the dynamic nature of the load. The pipe break load (R_r) is considered to act as three separate components (R_{rr}, R_{rj}, R_{rm}), which are defined below. In determining the equivalent static load for R_{rr}, R_{rj}, and R_{rm}, elasto-plastic behavior may be assumed with ductility ratios, provided excessive deflections do not result in loss of function of any safety-related SSC.

  – Pipe break reaction loads (R_{rr})—R_{rr} is defined as the equivalent static load on the structure generated by the reaction of the high-energy pipe during the postulated break.

  – Pipe break jet impingement loads (R_{rj})—R_{rj} is defined as the jet impingement equivalent static load on the structure generated by the postulated break.

  – Pipe break missile impact loads (R_{rm})—R_{rm} is defined as the missile impact equivalent static load on the structure generated by or during the postulated break, such as pipe whipping.

Other Loads

Other loads refer to postulated events or conditions that are not included in the design basis (GDC 4). These loading conditions and effects are evaluated without regard to the bounding conditions under which SSC are required to perform design basis functions. This load category includes:

- Aircraft hazard (A)—Aircraft hazard refers to loads on a structure resulting from the impact of an aircraft. The evaluation of this loading condition is considered as part of the plant safeguards and security measures.

- Explosion pressure wave (B)—Explosion pressure wave refers to loads on a structure resulting from an explosion in the vicinity of the structure. The evaluation of this loading condition is considered as part of the plant safeguards and security measures.

- Missile loads other than hurricane- or tornado-generated missiles—The hurricane- and tornado-generated missile spectra presented in Table 3.5-1 is considered to bound other external missile loads for the U.S. EPR other Seismic Category I structures. Turbine missiles and conformance to RG 1.115 are addressed in Section 3.5. As described in Section 3.5.1.3, the impact of turbine missiles on other Seismic Category I structures is not considered safety significant based on the redundancy and the low probability of a turbine missile being generated. Other Seismic Category I concrete and steel structures are designed for internally generated missile loads as described in Section 3.5.

3.8.4.3.2 Loading Combinations

*[Load combinations for design of other Seismic Category I structures are in accordance with ACI 349-2001 and RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures]** (GDC 1, GDC 2, GDC 4, and GDC 5).

The NI Common Basemat Structure is a monolithic concrete structure. However, various portions of the structure have different classifications (i.e., RCB, RB internal structures, and other Seismic Category I structures) and correspondingly different design requirements, as shown in Figure 3.8-118. In some instances, the load combinations identified in ACI 349-2001 do not include certain independent loadings which should be considered to account for potential structure-to-structure effects (i.e., the effect on one structure resulting from loadings applied to a separate, but monolithically connected, structure). To account for potential structure-to-structure effects, the loading combinations from ACI 349-2001 are adjusted by including the necessary additional independent loadings. For concrete structures, the independent loadings added to the load combinations include buoyant force (F_b) and post-tension load (J). For steel structures, the independent loadings added to the load combinations include hydrostatic load (F), buoyant force (F_b), post-tension load (J), and soil load/ lateral earth pressure (H). In load combinations where abnormal loads are considered, internal flood load (F_a) is added for both steel and concrete structures. The load factors for hydrostatic load (F), buoyant force (F_b), and post-tension load (J) are matched to that of the dead load (D) for each loading combination, while the load factors for soil load/lateral earth pressure (H) and internal flood load (F_a) are matched to that of the live load (L). Section 3.8.4.3.1 provides details regarding the loads considered for the design of other Seismic Category I structures, while Section 3.8.1.3.1 provides the description of the post-tension load (J) which is included to account for the global effect of post-tension loads (J) on the NI Common Basemat.

The following criteria apply for load combinations for concrete and steel Seismic Category I structures other than the RCB and RB internal structures:

- *[For concrete members, U is defined as the section strength required to resist design loads based on the strength design methods described in ACI 349.*
- *For steel members, S is defined as the required section strength based on the elastic design methods and the allowable stresses defined in Part Q1 of ANSI/AISC N690.*
- *For steel members, Y is defined as the section strength required to resist design loads based on plastic design methods described in Part Q2 of ANSI/AISC N690.]**

Loads and loading combinations encompass the soil cases described in Section 3.7.1, using the design criteria described in Section 3.7.1 and Section 3.7.2.

Other Seismic Category I Structures – Concrete

The following load combinations define the design limits for concrete Seismic Category I structures, other than the RCB and RB internal structures:

- Service load combinations for the strength design method.

  $U = 1.4(D + F + F_b + J) + 1.7(L + H + R_o)$

  $U = 1.4(D + F + F_b + J) + 1.7(L + H + W + R_o)$

  $U = 1.05(D + F + F_b + J) + 1.3(L + H + R_o) + 1.2T_o$

  $U = 1.05(D + F + F_b + J) + 1.3(L + H + R_{o + W}) + 1.2T_o$

- Factored load combinations for the strength design method.

  $U = D + L + H + F + F_b + T_o + R_o + J + E'$

  $U = D + L + H + F + F_b + T_o + R_o + J + W_t$

  $U = D + L + H + F + F_b + J + T_a + R_a + F_a + 1.4P_a$

  $U = D + L + H + F + F_b + J + T_a + R_a + F_a + P_a + E'$

  $U = D + L + H + F + F_b + J + T_a + R_a + F_a + P_a + R_r + E'$

Other Seismic Category I Structures - Steel

The following load combinations define the design limits for steel Seismic Category I structures, other than the RCB and RB internal structures. For normal service load conditions, either the elastic working stress design methods of Section Q1 or the plastic design methods of Section Q2 of ANSI/AISC N690, including Supplement 2, are used.

- Service load combinations for the elastic working stress design method.

  $S = D + L + H + F + F_b + J$

  $S = D + L + H + F + F_b + J + W$

  $1.5S = D + L + H + F + F_b + T_o + R_o + J$

  $1.5S = D + L + H + F + F_b + T_o + R_o + J + W$

- Service load combinations for the plastic design method.

$Y = 1.7(D + L + H + F + F_b + J)$

$Y = 1.7(D + L + H + F + F_b + J + W)$

$Y = 1.3(D + L + H + F + F_b + T_o + R_o + J)$

$Y = 1.3(D + L + H + F + F_b + T_o + R_o + J + W)$

- Factored load combinations for the elastic working stress design method.

$1.6S = D + L + H + F + F_b + T_o + R_o + J + E'$

$1.6S = D + L + H + F + F_b + T_o + R_o + J + W_t$

$1.6S = D + L + H + F + F_b + J + T_a + R_a + F_a + P_a$

$1.6S = D + L + H + F + F_b + J + T_a + F_a + P_a$

(This load combination is used when the global non-transient sustained effects of T_a are considered).

$1.7S = D + L + H + F + F_b + J + T_a + R_a + F_a + P_a + R_r + E'$

- Factored load combinations for the plastic design method.

$0.9Y = (D + L + H + F + F_b + T_o + R_o + J + E')$

$0.9Y = (D + L + H + F + F_b + T_o + R_o + J + W_t)$

$0.9Y = (D + L + H + F + F_b + J + T_a + R_a + F_a + 1.25P_a)$

$0.9Y = (D + L + H + F + F_b + J + T_a + R_a + F_a + P_a + R_r + E')$.

3.8.4.4 Design and Analysis Procedures

Analysis and design procedures are similar for the various concrete and steel other Seismic Category I structures but vary somewhat from structure to structure. The general analysis and design procedures applicable to other Seismic Category I structures are explained below. The procedures specific to the following other Seismic Category I structures are also described.

- The RSB and annulus, FB, and SBs.
- The EPGBs.
- The ESWBs.
- Buried conduit and duct banks, and buried pipe and pipe ducts.

Design and analysis procedures described in the following sections also apply to the design of supports for Seismic Category I distribution systems (i.e., pipe supports, equipment supports, cable tray supports, conduit supports, HVAC duct supports, and other component supports) and to Seismic Category I platforms and miscellaneous steel structures located within other Seismic Category I buildings and structures.

3.8.4.4.1 General Procedures Applicable to Other Seismic Category I Structures

*[Other Seismic Category I concrete structural elements and members are designed in accordance with the requirements of ACI 349-2001 and its appendices]** (GDC 1). Exceptions to code requirements specified in RG 1.142 are incorporated into the design and are accommodated in the loading combinations described in Section 3.8.4.3.2 for concrete structures.

[The design of concrete walls, floors, and other structural elements for other Seismic Category I structures is performed using the strength-design methods described in ACI 349-2001. The ductility requirements of ACI 349-2001 are satisfied to provide a steel reinforcing failure mode and prevent concrete failure for design basis loadings.

The design of anchors and embedments conforms to the requirements of ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1)] and RG 1.199 (with exception described in Section 3.8.1.4.10). [The requirements of Appendix C of ACI 349-2001 are followed for impulsive and impactive loading conditions (e.g., loading combinations that include pipe break missile impact loads, hurricane- or tornado-generated missile impact loads).*

Other Seismic Category I steel members and assemblies are designed in accordance with ANSI/AISC N690-1994 (R2004, including Supplement 2) (GDC 1). Steel member design uses the allowable stress design methods of ANSI/AISC N690.

*The design of bolted connections is in accordance with ANSI/AISC N690, Section Q1.16 and AISC 348-00/2000 RCSC, "Specification for Structural Joints Using ASTM A325 and A490 Bolts."]** Bolted connections are designed to be fully tensioned (e.g., slip critical) unless justified otherwise.

The design of welded connections is in accordance with AWS D1.1 or AWS D1.6.

*[The design of bolted connections in combination with welded connections is in accordance with Section Q.15.10 of ANSI/AISC N690.]**

Loads and load combinations defined in Section 3.8.4.3 are used to determine strength requirements of members and elements of other Seismic Category I structures. Abnormal pipe break accident loads only apply to limited areas of structures located on the NI Common Basemat Structure. The following criteria apply for load combinations for concrete and steel other Seismic Category I structures:

- Where any load reduces the effects of other loads, the corresponding coefficient for that load is 0.9 if it can be demonstrated that the load occurs simultaneously with other loads.

- Where the structural effects of differential settlement, creep, or shrinkage may be significant, they are included with the dead load (D) as applicable.

- For load combinations in which a reduction of the maximum design live load (L) has the potential to produce higher member loads and stresses, multiple cases are considered where the live load (L) is varied between its maximum design value and zero.

- Roofs with a slope of less than 0.25 inches per foot are analyzed for adequate stiffness to preclude progressive deflection as water ponding is created from the snow load or from rainfall on the surface. The analysis considers the potential blockage of the primary drainage system of the area that is subject to ponding loads. The analysis uses the larger of the snowmelt depth or rain load.

- For load combinations including the loads P_a, T_a, R_a, R_{rr}, R_{rj}, or R_{rm}, the maximum values of these loads, including a dynamic load factor, are used unless a time-history analysis is performed to justify otherwise.

- For load combinations including loads R_{rr}, R_{rj}, R_{rm}, or W_m, these load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of intended function of the structural member or a loss of function of any safety-related SSC.

- Tornado and hurricane loads are applied to roofs and exterior walls of other Seismic Category I structures. If tornado and hurricane pressure boundaries are not established at the exterior walls, interior walls are designed as tornado and hurricane pressure boundaries.

- For load combinations that include a tornado and hurricane load (W_t), the tornado and hurricane load parameter combinations described in Section 3.3 are used.

Concrete and steel structural elements and members are designed for axial tension and compression forces, bending moments, torsion, and in-plane and out-of-plane shear forces for the controlling loading combinations that are determined from analysis. Concrete and steel members and elements remain elastic for loadings other than impact. Local yielding is permitted for localized areas subjected to hurricane- and tornado-generated missile loads, pipe break accident loadings, and beyond design basis loadings. The structural integrity of members and elements is maintained for the loading combinations described in Section 3.8.4.3.

[A local analysis and design of concrete members will be performed for impactive and impulsive loads according to ACI 349, with exceptions noted in RG 1.142. A local

*analysis and design of steel members will be performed for impactive and impulsive loads according to ANSI/AISC N690.]**

It is acceptable to assume non-linear (elasto-plastic) response of structural members for evaluation of the response of reinforced concrete and steel structures subject to impactive or impulsive loads. Deformation under impactive and impulsive loads is controlled by limiting the ductility ratio, μ_d, which is defined as the ratio of maximum acceptable displacement, χ_m (or maximum strain, ε_m), to the displacement at the effective yield point, χ_y (or yield strain, ε_y), of the structural member. In addition to the specified deformation limits, maximum deformation will not result in the loss of intended function of the structural member nor impair the design basis safety function of other systems and components.

Regarding structural capacity, a structural member will retain its ability to perform its design basis function when ductility limits for concrete and steel members presented in Table 3.5-3 are satisfied. As deformation limits of the member may be governed by attached structures, systems and components (SSC), the member will also satisfy deformation limits imposed by attached SSC to prevent loss of design basis function.

Analysis and design of other Seismic Category I structures are performed using a combination of computer models and local analyses. Computer models are used to perform overall analysis of major structures. The loads and loading combinations described in Section 3.8.4.3 are applied to the overall computer model to design for global effects of the loadings. Local analyses and designs are performed using refined computer submodels and manual calculations. Local analyses and designs are used to account for local discontinuities (e.g., openings, thickened areas, local loads, punching shear checks, and changes in member cross-section). Local analyses are also used to determine designs for items such as component supports, embedments, anchors, platforms, and other miscellaneous structural items. Techniques used for major structures are described in Sections 3.8.4.4.2 through 3.8.4.4.5.

Refer to Section 9.1.5 for design requirements applicable to cranes located in other Seismic Category I structures.

Openings in walls and slabs of other Seismic Category I structures are shown in construction drawings. Openings are acceptable without analysis if they meet the criteria identified in ACI 349, Section 13.4.2. Round pipe sleeves are used in lieu of rectangular penetrations where possible. Corners of rectangular openings in walls and slabs are provided with diagonal reinforcing to reduce cracking due to stress concentration at these locations in accordance with ACI 349, Section 14.3.7.

Appendix 3E describes analysis and design results for critical sections of other Seismic Category I structures.

Section 3.7.2 addresses design procedures applicable to non-safety-related structures to preclude adverse interaction effects on Seismic Category I structures.

Static Analysis and Design

Dead loads (D), live loads (L), hydrostatic loads (F), soil loads and lateral earth pressure loads (H), wind loads (W), pipe reactions (R_o), and normal thermal loads (T_o) are considered in the analysis and design of other Seismic Category I structures for the static normal load concrete and service load steel loading combinations. Concrete and steel members are designed to accommodate these static loads within the elastic range of their section strength. For concrete structures, uncracked section properties are used to proportion loadings to members. However ultimate strength design is used to reinforce concrete elements and members subjected to the normal factored loading combinations defined in Section 3.8.4.3.2.

Static fluid pressure loads are considered for design of the walls and floors of tanks and storage pools. Moving loads are considered for mobile plant equipment (e.g., cranes, hoists, truck bays in buildings, maintenance aisles).

Seismic and Other Dynamic Analyses and Design

Seismic analyses and designs of other Seismic Category I structures conform to the procedures described in Section 3.7.2. Seismic accelerations are determined from dynamic FEM as described in Section 3.7.2. These accelerations are applied to the static FEM of other Seismic Category I structures as static-equivalent loads at the elevations used in the dynamic FEM.

Seismic SSE (E') loads are obtained by multiplying the dead load and 25 percent of the design live load by the structural accelerations obtained from the seismic analyses of each structure. A minimum of 75 percent of the roof snow load is included in the structural mass for seismic analysis of Seismic Category I structures. Seismic loads are also considered due to the mass of fluids in tanks and canals as described below for hydrodynamic loads. Consideration is given to the amplification of seismic accelerations obtained from the dynamic FEM, due to local flexibility of structural elements and members. Construction loads are not included when determining seismic loads. Other temporary loads are evaluated for contributing to the seismic loads on a case-by-case basis.

Seismic loads from the three components of the earthquake are combined using the SRSS method, where resultants are obtained using the following formulas:

$$P_R = \pm\ \mathrm{sqrt}(P_x^2+P_y^2+P_z^2)$$

$$M_R = \pm\ \mathrm{sqrt}(M_x^2+M_y^2+M_z^2)$$

The number of permutations for design are $2^n = 2^2$ = (++, --, +-, -+).

The effects of local flexibilities in floor slabs and wall panels are considered to determine if additional seismic accelerations should be applied to their design beyond those determined from the seismic stick model. Local flexibility evaluations are performed by determining the natural frequency of the floor or wall panel and comparing this to the frequency of the zero period acceleration on the applicable response spectra. Additional acceleration is applied when the natural frequency of the panel results in higher accelerations than the zero period acceleration. In cases where local flexibilities are determined to be a factor, additional out-of-plane accelerations are applied to the inertia loads on these panels for determining out-of-plane bending and shear loads.

Additional seismic loads due to accidental torsion are considered as described in Section 3.7.2. This is to account for variations in material densities, member sizes, architectural variations, equipment loads, and other variations from the values used in the analysis and design of other Seismic Category I structures. Due to these potential variations, an additional eccentricity of the mass is included at the floor elevations that are equivalent to 5 percent of the maximum building dimension.

*[Seismic Category I concrete structural elements and their connections are detailed for ductility in accordance with ACI 349-2001, Chapter 21.]**

Structural Stiffness Considerations

Conservative values of concrete creep and shrinkage are used in the design of other Seismic Category I structures. Moments, forces, and shears are obtained on the basis of uncracked section properties in the analysis. However, in sizing the reinforcing steel required, the concrete is not relied upon for resisting tension. Thermal moments are modified by cracked-section analysis using analytical techniques, when the state of loading indicates the development of cracks.

The effect of local wall and floor slab flexibility is included where the analysis indicates the existence of this condition. The concrete section properties used in calculating the amplified seismic forces include an appropriate level cracking for the particular element under consideration. The amplified forces are also used in the design of the structural members that support the flexible element.

Section 3.8.4.6 describes methods used to confirm that concrete properties satisfy design requirements.

Seismic Structural Damping

Seismic analysis of other Seismic Category I structures uses the following SSE structural damping values as recommended by RG 1.61.

Structure Type	Percent of Critical Damping
• Welded Steel	4
• Bolted Steel, Slip Critical Connections	4
• Bolted Steel, Bearing Connections	7
• Reinforced Concrete	7

Hydrodynamic Loads

Hydrodynamic loads are applied to the walls and floors of the spent fuel pool and liquid storage tanks in the SBs and in the ESWBs to account for the impulsive and convective effects of the water moving and sloshing in the tanks as a result of seismic excitation. These loads are considered as part of the seismic SSE loads, and components of these loads in the three orthogonal directions are combined in the same manner as other seismic loads. The requirements of ASCE Manual No. 58, USAEC TID-7024, and other proven methods are used to determine hydrodynamic loadings. For the impulsive mode of the NI Pools, the enveloped Zero Period Acceleration (ZPA) of the tank is used to determine hydrodynamic pressure. The tank structural flexibility is included in values obtained from the In-Structure Response Spectra (ISRS). For the convective mode of the NI Pools and the ESWB Pools, the natural frequency of sloshing water is determined and the corresponding acceleration is based on 0.5 percent damping.

In the static FEM, hydrodynamic loads are applied statically in each of six directions (i.e., east, west, north, south, up, and down). The hydrodynamic loads due to a horizontal earthquake are a combination of impulsive and convective forces simultaneously acting on the pool walls and slabs, accounting for the rotational effects of water motion. The hydrodynamic pressure on the slab and walls due to a vertical earthquake are calculated using the fluid mass density, multiplied by the vertical spectral acceleration of each pool slab location as a function of water depth below the surface.

For the spent fuel pool, the combined rack and hydrodynamic loads including rack sliding/impact loads are applied separately in the static FEM. The combined loads need to be higher than the whole pool seismic analysis results, considering the rack, fluid, and pool dynamic interaction. The impact, friction, and/or hydrodynamic peak instantaneous loads due to rack rocking/sliding will be considered in a local design for punching shear and bending checks.

Design for hydrodynamic loads is within the elastic range of concrete and steel members and elements.

Thermal Analysis and Design

Normal thermal loads (T_o) are considered in the analysis and design of other Seismic Category I structures. Abnormal pipe break accident thermal loads (T_a) are considered to have no effect on the overall structure of other Seismic Category I structures and are only considered in local analyses.

For concrete structures, the requirements of ACI 349, Appendix A, ACI 349.1R, or thermal analysis computer programs or similar procedures are used to evaluate thermally induced forces and moments. When considering the combined effects of thermal stress and stress due to other loads, the analysis satisfies the requirements of Appendix A of ACI 349.

Pipe Rupture Loads

Other Seismic Category I structures will be evaluated for pipe rupture loads. Local analyses of other Seismic Category I structures consider the following abnormal loads for areas that house high-energy piping systems:

- Subcompartment pressure loads (P_a).
- Pipe break thermal loads (T_a).
- Accident pipe reactions (R_a).
- Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}).
- Local flood loads (F_a).

Subcompartment pressure loads (P_a) resulting from a LOCA event are evaluated as time-dependent loads across concrete walls and floors that enclose high-energy piping systems. Concrete and steel members are designed to accommodate subcompartment pressure loads within the elastic range of the section strength.

Pipe break thermal loads (T_a) are considered in local analyses of concrete walls and floors. Accident thermal loads are evaluated as time-dependent loads across concrete walls and floors that enclose high-energy piping systems subject to LOCA events. *[The thermal design of concrete is in accordance with ACI 349-01/349-R01, Appendix A and ACI 349.1R-07.]**

Accident pipe reaction loads (R_a) are considered on piping supports, including supports for the MS and feed water piping. Concrete and steel members are designed to accommodate accident pipe reaction loads within the elastic range of their section strength.

Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}) are considered in local design of concrete walls and floors and steel members. Dynamic load factors are applied when analyzing structures for the static equivalent of these loads. Elasto-plastic behavior may be assumed with ductility ratios, provided that excessive deflections do not result in the loss of function of any safety-related SSC. *[Pipe break reactions, jet impingement, and missile impact impulsive and impactive loads are in accordance with Appendix C of ACI 349-2001.]** The design of other Seismic Category I structures for these loads conforms to the procedures described in Section 3.5 for internally generated missiles. Section 3.5 also describes ductility limits that are followed for impactive and impulsive loadings.

Flood loads (F_a) are applied to walls and floors in the local design of other Seismic Category I structures. Concrete and steel members are designed to accommodate these flood loads within the elastic range of their section strength.

Missile Impact Design

The design of Seismic Category I structures for internally generated and externally generated missiles conforms to the procedures described in Section 3.5.

*[Concrete missile barriers subject to missile impact loads are designed in accordance with Appendix C of ACI 349.]** Steel missile barriers subject to missile impact loads are designed in accordance with the requirements of ASCE No. 58. Missile protection barriers that use composite sections will be evaluated for local damage using the residual velocity of the missile perforating the first element as the striking velocity of the missile for the next element in the section.

Seismic Category I structures, shields, and barriers designed to withstand the effects of missile impacts are evaluated for local damage in the impacted area, including an estimation of the depth of penetration and, in the case of concrete barriers, the potential for generation of secondary missiles by spalling or scabbing. Global and regional effects of missile impact are also evaluated for concrete and steel missile barriers.

Dynamic load factors are applied when analyzing structures for the static equivalent of missile impact loads. Elasto-plastic behavior may be assumed with ductility ratios, provided excessive deflections do not result in loss of function of any safety-related SSC.

Structures that are not classified as Seismic Category I structures are not relied upon to shield Seismic Category I structures from the effects of missile impact.

Flood Design

In addition to designing for the external flood loads described in Section 3.8.4.3.1, Seismic Category I structures are protected against external flooding by the following methods:

- Exterior wall penetrations below plant flood level are sealed to prevent flood waters from entering Seismic Category I buildings.
- Finished yard grade around Seismic Category I structures is sloped to direct flood water and runoff away from the structures.
- Finished floor elevations are at one foot above plant finished grade where openings are provided for personnel and maintenance access.
- Water stops are provided in below grade exterior construction joints.
- Floor drainage is provided for building interior floors to collect water that could potentially enter the buildings.

See Section 3.4 for additional information on flood protection.

3.8.4.4.2 Reactor Shield Building and Annulus, Fuel Building, and Safeguard Buildings – NI Common Basemat Structure

Loads from the loading combinations described in Section 3.8.4.3 are applied to the NI Common Basemat Structure, which includes the RSB, the FB, and SBs. Vertical loads transfer to the NI Common Basemat Structure foundation basemat through concrete exterior walls, concrete interior walls, and concrete and steel columns. Lateral loads transfer to the NI Common Basemat Structure foundation basemat by diaphragm action of the concrete roof slabs and intermediate concrete floor slabs, which transfer loads to the interior and exterior concrete shear walls. Lateral loads transfer to the soil subgrade by friction and passive earth pressure.

The reinforced concrete roof slabs and intermediate floor slabs are analyzed and designed as two-way slabs. Reinforced concrete walls are analyzed and designed as shear walls and compression members, which are also subjected to out-of-plane bending moments, torsion, and out-of-plane shear. Analysis and design of the NI Common Basemat Structure foundation basemat is addressed in Section 3.8.5.

Lateral pressure due to seismic loads for the below grade NI perimeter walls are obtained from an FEM SSI analysis.

The NI Common Basemat Structure is included in the ANSYS V10.0 SP1 finite element overall computer model of the NI Common Basemat Structure that is described in Section 3.8.1.4.1. The NI Common Basemat Structure model includes the RSB, FB, and SBs as well as the RCB, RB internal structures, and the NI Common

Basemat Structure foundation basemat that are described in other sections. Boundary conditions for the ANSYS computer model and methods used for application of axisymmetric and non-axisymmetric loads, transient and localized loads, and other parameters used in the model are described in Section 3.8.1.4.

The NI Common Basemat Structure is modeled using a mesh of ANSYS finite elements representing primary load-carrying walls, floors, columns, and beams. Gaps are maintained between structures adjacent to Seismic Category I structures to allow for structural movements during seismic events, containment pressurization, missile strikes, aircraft impact, explosions, and other loading conditions. Exterior walls and roofs of the hardened SBs 2 and 3, RSB, and the FB are modeled to be independent of the internal structures, because there is no physical connection of internal walls and slabs in these structures with the outside walls and roof.

ANSYS SHELL43 solid shell elements are used to model walls and floors and other concrete elements in the NI Common Basemat Structure. SHELL43 is a three-dimensional, four-node shell element that is suitable for moderately thick shell structures. SHELL43 can also provide out-of-plane shear forces and has an elastic-plastic capability. BEAM44 members are used to model beams and columns. The ANSYS finite element computer program is used to analyze the NI Common Basemat Structure for the loads and load combinations described in Section 3.8.4.3.

The FEM used for the analysis of the NI Common Basemat Structure is shown in Figure 3.8-86—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Outside View, Figure 3.8-87—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Section Through Fuel Building and Safeguard Building 2/3 Island, and Figure 3.8-88—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Section Through Safeguard Buildings 4 and 1.

Local analyses are used to analyze other Seismic Category I structures for locally applied loadings that have no significant effect on the overall behavior of the structures. Local analyses are performed for the pipe rupture loads described in Section 3.8.4.4.1, for the missile impact loads also described in Section 3.8.4.4.1, and for the spent fuel pool, as well as for other loadings and local structural areas. Spent fuel pool local analysis is performed using LS-DYNA, Version 971 software. The whole pool FEM contains the pool concrete walls, fuel racks, and water.

Section 9.1.2 addresses fuel storage racks.

Subsystem supports (i.e., pipe supports, HVAC duct supports, electrical conduit supports, cable tray supports) are analyzed and designed using local analyses. Analysis and design of subsystem supports are performed in accordance with the same criteria and codes specified for design of other Seismic Category I structures. See Section 3.7.3

for additional descriptions of methods used for seismic analyses of distribution systems.

3.8.4.4.3 Emergency Power Generating Buildings

The EPGBs are reinforced concrete shear wall structures. Vertical loads transfer to the reinforced concrete foundation basemat through the reinforced concrete walls. Lateral loads transfer to the foundation basemat by diaphragm action of the reinforced concrete roof slabs to the reinforced concrete walls. Lateral loads from the foundation basemat are transferred to the supporting soil through bearing, friction, and passive earth pressure.

The reinforced concrete walls are designed as shear walls which are subjected to compression loads, in-plane and out-of-plane bending moments, and in-plane and out-of-plane shear. The floor slab at elevation 51 feet, 6 inches consists of a composite slab with composite structural steel beams. The roof slab at elevation +68 feet, zero inches is primarily designed as a one-way slab due to the relative aspect ratio between the lines of support.

The EPGBs are analyzed and designed using a 3D FEM representing the structure. The FEM is generated using the GT STRUDL computer code to accomplish the following:

- Provide an accurate representation of the structure for translation to a soil structure interaction (SSI) model for seismic analysis (See Section 3.7.2 for information on the extrapolation of the GT STRUDL FEM for the seismic analysis).
- Conduct a static analysis of the EPGBs using equivalent static seismic loads; and other applicable design loads.
- Provide input for the design of reinforced concrete structural elements.

The FEM of the EPGBs consists of SBHQ6 and SBHT6 elements representing the load carrying reinforced concrete walls and slabs, as these element types are suitable for capturing both the in-plane and out-of-plane effects from the corresponding applied loads.

The EPGB is a surface-founded structure, with compression only spring boundary conditions utilized to represent the soil. Soil bearing pressures are determined from the SSI analysis.

For uniformity of site characteristics, the required bearing demand will be the same as for the NI.

The equivalent SSI model includes modifications to the stiffness of the various composite beams at elevation 51 feet, 6 inches, as well as modifications to account for cracking. The stiffness of these composite beams is included in the model to capture out-of-plane response. Stiffness of the composite beams is not required in the static analysis model as only in-plane stresses in the concrete slab are determined.

For the composite beams and floor slab at elevation 51 feet, 6 inches, the corresponding floor accelerations from the SASSI analysis output are applied to tributary floor areas and walls to obtain the seismic loads associated with the out-of-plane loads. Dead load, live load, equipment loads, and piping loads are combined with the seismic loads. The composite beams are analyzed outside of the FEM. *[Structural design of the composite beams is in accordance with the provisions of ANSI/AISC N690-1994 (R2004).]**

The in-plane and out-of-plane results from the GT STRUDL equivalent static analysis are extracted and used to design reinforced concrete shear walls and slabs according to provisions of ACI 349-01. The evaluation of walls and slabs for external hazards (e.g., hurricane- or tornado generated missiles and blast loads) is also performed by local wall and slab analyses. Structural element reinforcement is designed to provide sufficient ductility.

Additional information on the seismic analysis approach for the EPGBs is contained in Section 3.7.2.

For the design of the EPGBs, some details for the composite beams and slabs at elevation 51 feet, 6 inches, particularly changes in beam sizes and floor openings, as well as certain aspects of mechanical design layout, are not reflected in the SASSI FEM used for SSI analyses. Inclusion of these details in the SASSI FEM are not expected to have any significant impact on the seismic forces used in the design of the EPGBs, but may impact the in-structure response spectra. Therefore, a subsequent analysis will be performed with these details in the FEM to confirm the seismic responses and in-structure response spectra presented in Section 3.7.2. The design of the EPGBs will conform to the structural acceptance criteria described in Section 3.8.4.5.

3.8.4.4.4 Essential Service Water Buildings

Reinforced concrete elements for the four ESWBs consist of slabs, beams, shear walls, and foundation basemat to transfer imposed loads to the supporting soil. Structural steel framing is used to support the missile barriers protecting the safety-related fans.

Similar to the EPGBs, the ESWBs are analyzed and designed using a 3D FEM representing the structure. The FEM is generated using the GT STRUDL computer code. The use of the model for both static and dynamic analyses, including extraction of results for design, is almost identical to the methods presented in Section 3.8.4.4.3. Similarly, the GT STRUDL model is used to provide an accurate representation of the structure for translation to an SSI model (SASSI 2000) for seismic analysis. As such, only model variations are addressed below.

In addition to structural dead loads, slab live loads, piping loads and equipment loads, the GT STRUDL FEM for the ESWBs includes the weight of non-structural fill, hydrostatic loads, hydrodynamic loads, and soil pressures (including surcharge pressures). The appropriate accelerations from the SSI analysis are applied to the tributary floor areas and walls to obtain the equivalent static seismic loads.

*[Dead load, live load, equipment loads, and piping loads are combined with the equivalent static seismic loads for structural design in accordance with the provisions of ACI 349-01, with supplemental guidance of RG 1.142, ACI 350-06, and ACI 350.3-06.]** The evaluation of walls and slabs for external hazards (e.g., hurricane- or tornado-generated missiles) is performed by local analyses, including ductility evaluations. The elastic solution methodology of ASCE 4-98, Section 3.5.3.2 is used for the dynamic soil pressures associated with the 21 feet embedment of the ESWBs.

Seismic induced lateral soil pressure on below grade walls are evaluated considering the following cases:

- The seismic soil pressure as equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2.
- The seismic soil pressure as equal to the passive earth pressure.

Additional information on the seismic analysis approach for the ESWBs is contained in Section 3.7.2.

3.8.4.4.5 Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts

The design of buried conduit and duct banks, and buried pipe and pipe ducts is site-specific. Buried Seismic Category I conduit, electrical duct banks, pipe, and pipe ducts will be analyzed and designed in accordance with the specific requirements of the systems. In addition, these items will be designed for the effects of soil overburden, surcharge, groundwater, flood, seismic soil interaction, and other effects of burial. *[Concrete components of buried items will be designed in accordance with ACI 349-2001, including the exceptions specified in RG 1.142. Steel components of buried items will be designed in accordance with ANSI/AISC N690-1994 (R2004), including Supplement 2.]**

Static and long-term analyses of buried items will be based on soil properties under consolidated drained conditions of the soil. Buried items will be designed for soil loads corresponding to the weight of the overlying soil prism.

Live loads will be applied, such as those imposed by truck and rail traffic and by construction equipment and activities. Where buried items are vulnerable to highway or railway traffic loads, the potential for fatigue-induced failure will be evaluated. The minimum burial depth of buried items will conform to guidance from ASME 31.4 and ASME 31.8.

The pressure transmitted to a buried component by surface traffic live load is determined according to Boussinesq's equation:

$$P_P = 0.48 \frac{P_S}{H^2 \left[1 + \left(\frac{d}{H}\right)^2\right]^{2.5}}$$

Where:

P_p = surface load transmitted to the buried item.

d = offset distance from the surface load to buried item.

H = thickness of soil cover above the item.

P_s = concentrated surface load.

The calculated value of P_p is multiplied by an impact factor which can be found in the AREVA NP Topical Report "U.S. EPR Piping Analysis and Pipe Support Design." For the U.S. EPR, specific live loads for trucks are based on the AASHTO H20 and HS20 trucks. In addition, rail loads are based on the Cooper E80 railway loads, but may be controlled by anticipated shipping weights. Recommended surface loads transmitted to the buried components (Pp) for the AASHTO H20/HS20 truck and Cooper E80 railway loads can be found in the Topical Report.

For the U.S. EPR, the load for buried components is considered an equivalent soil load. The same load factors are applied for live load and soil load in all applicable soil loads so the net effects for buried components are the same.

Buried items will be designed for freeze-thaw induced stresses and for other thermally induced stresses due to soil and ambient air temperatures. Interfacial longitudinal friction effects will be evaluated for buried pipes that are encased in larger pipes or in concrete trenches or boxes.

In cases where buried items are located below the ground water table or where seasonal change in the ground water table is significant, the effect of buoyancy and the increased weight of water will be evaluated. These evaluations will include the effects of fluctuations in ground water level and the effects of flood.

Seismic load effects on buried items will be evaluated using dynamic analyses or equivalent static load methods. For seismic-related and dynamic analyses, the shear strength of soil will be based on the consolidated-undrained triaxial stress conditions of the soil. The procedure for evaluating the structural integrity of buried items under seismic conditions will involve determination of the axial and bending strains in the system due to seismic wave propagation in the surrounding soil mass. The axial force and bending stresses will be computed using the buried item material properties (e.g., Young's modulus and pipe section modulus). Pipe ovalization will be computed based on the applicable static and dynamic loads imposed on the pipe from the soil and surcharge loads. Soil-structure interaction analyses will be performed for buried piping systems based on the free-field earthquake motion, considering the three orthogonal components of the motion. The influence of geotechnical properties such as strain rate and magnitude, confining stress, and relative density on pore pressure, damping, and shear modulus will be incorporated into analyses. Response of buried items to burial depth, groundwater, presence of adjacent structures, and soil heterogeneity will be evaluated in seismic analyses.

Buried items will be evaluated for the effects of settlement and ground movement, including potential damage related to compaction of soil during construction, long-term elastic and consolidation settlement (total and differential), freeze-thaw induced settlement, seismic-induced settlement, seismic wave propagation, and seismic-induced permanent ground deformation. The effects of differential settlement between buried pipes and the buildings or structures to which pipes are anchored will be evaluated. At site locations where differential settlement is significant, flexible anchors may be used in lieu of rigid anchors. Support structures will be designed to resist the resulting axial loads, bending stresses, and shear stresses imposed by buried items on the structure.

Refer to the AREVA NP Inc., U.S. Piping Analysis and Pipe Support Design Topical Report (Reference 37) for additional analysis and design procedures applicable to buried piping.

A COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures used for buried conduit and duct banks, and buried pipe and pipe ducts.

A COL applicant that references the U.S. EPR design certification will use results from site-specific investigations to determine the routing of buried pipe and pipe ducts.

A COL applicant that references the U.S. EPR design certification will perform geotechnical engineering analyses to determine if the surface load will cause lateral or vertical displacement of bearing soil for the buried pipe and pipe ducts and consider the effect of wide or extra heavy loads.

3.8.4.4.6 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.4, 2.5, 3.3, 3.5, 3.7, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections. A cross-reference between U.S. EPR FSAR sections and information required by SRP Section 3.8.4, Appendix C is provided in Table 3.8-17.

3.8.4.5 Structural Acceptance Criteria

[Limits for allowable stresses, strains, deformations and other design criteria for other Seismic Category I reinforced concrete structures are in accordance with ACI 349-2001 and its appendices (GDC 1, GDC 2, and GDC 4). Limits for concrete design include the exceptions specified in RG 1.142.

Limits for allowable loads on concrete embedments and anchors are in accordance with the requirements of ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1) and RG 1.199 (with exception described in Section 3.8.1.4.10).

*Limits for the allowable stresses, strains, deformations, and other design criteria for other structural steel Seismic Category I structures are in accordance with ANSI/AISC N690-1994 (R2004) including Supplement 2 (GDC 1, GDC 2, and GDC 4).]**

Allowable settlements for other Seismic Category I structures are described in Section 2.5.

The design of other Seismic Category I structures is generally controlled by load combinations containing SSE seismic loads. *[Stresses and strains are within the ACI 349-2001 limits, with the exceptions previously listed, and ANSI/AISC N690-1994 limits.]**

Appendix 3E provides design results for critical sections of other Seismic Category I structures.

An as-built report is prepared to summarize deviations from the approved design and confirm that the as-built other Seismic Category I structures (RSB, SB, FB, EPGB, and ESWB) are capable of withstanding the design basis loads described in Section 3.8.4.3 without loss of structural integrity or safety-related functions.

Structural acceptance criteria for buried Seismic Category I pipe are addressed in the AREVA NP Inc., U.S. Piping Analysis and Pipe Support Design Topical Report.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the criteria specified in Section 3.8.4.4.5 and those specified in AREVA

NP Topical Report ANP-10264NP-A.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control programs, and special construction techniques used in the fabrication and construction of concrete and steel Seismic Category I structures other than the RCB and the RB internal structures.

Construction of concrete radiation shielding structures and certain elements of design that relate to problems unique to this type of structure is in accordance to RG 1.69. The requirements and recommended practices contained in ANSI/ANS-6.4-2006, are generally acceptable for the construction of radiation shielding structures, as amended by the applicable exceptions noted in RG 1.69.

3.8.4.6.1 Materials

Concrete, reinforcing steel, and structural steel materials for other Seismic Category I structures are the same as described in Section 3.8.3.6 (GDC 1), except as follows:

Structural concrete used in the construction of other Seismic Category I structures has the following compressive strengths (f'_c) at 90 days.

- The NI Common Basemat Structures, including RSB, FB and SBs (except for foundation basemat): 6,000 psi minimum.
- The EPGBs: 5,000 psi minimum.
- The ESWBs: 5,000 psi minimum.
- Buried duct banks and pipe ducts: 4,000 psi minimum.

The use of epoxy coated reinforcing steel and waterproofing membranes for exterior walls and slabs will be evaluated on a site-specific basis as described in Section 3.8.5.6.

3.8.4.6.2 Quality Control

Quality control procedures for other Seismic Category I structures are the same as described in Section 3.8.3.6 (GDC 1).

3.8.4.6.3 Special Construction Techniques

Other Seismic Category I structures are constructed using proven methods common to heavy industrial construction. No special, new, or unique construction techniques are used.

Modular construction methods are used to the extent practical for pre-fabricating portions of the spent fuel pool liner, other tank liners, distribution system supports, reinforcing, concrete formwork, and other portions of other Seismic Category I structures. Such methods have been used extensively in the construction industry. Rigging is pre-engineered for heavy lifts of modular sections. Permanent and temporary stiffeners are used on liner plate sections and other modularized items to satisfy code requirements for structural integrity of the modular sections during rigging operations.

Steel decking and plates and supporting steel beams may be used to form concrete floors. In these instances, the decking thickness is in addition to the nominal floor thicknesses. The decking, plates, and beams may be left in place, in which case they are designed for applicable seismic loads and other loading conditions. Other types of formwork may also be used that are left in place and become a permanent part of the structure. Such items meet code requirements and are designed to prevent their failure from affecting Seismic Category I SSC.

3.8.4.7 Testing and Inservice Inspection Requirements

Monitoring and maintenance of other Seismic Category I structures is performed in accordance with the requirements of 10 CFR 50.65 and supplemented with the guidance in RG 1.160 (GDC 1).

Testing and inservice inspection of the spent fuel pool leak chase channels in the FB is addressed in Section 9.1.

Refer to Section 9.1.5 for testing and inservice inspection requirements applicable to cranes.

Physical access is provided to perform inservice inspections of exposed portions of other Seismic Category I structures.

Examination of inaccessible portions of below-grade concrete structures for degradation and monitoring of ground water chemistry are addressed in Section 3.8.5.7.

A COL applicant that references the U.S. EPR design certification will address examination of buried safety-related piping in accordance with ASME Code Section XI, IWA-5244, “Buried Components.”

3.8.5 Foundations

3.8.5.1 Description of the Foundations

Foundations for Seismic Category I structures are provided for the following buildings and structures:

- NI Common Basemat Structure foundation basemat.
- EPGB foundation basemats.
- ESWB foundation basemats. The ESWBs house the ESWCTs and the ESWPBs.

Foundations for buried items are included in Section 3.8.4. Section 3.7.2 addresses design requirements for Non-Seismic Category I structures to preclude adverse interaction effects on Seismic Category I structures.

Figure 3B-1 provides a site plan of the U.S. EPR standard plant showing the outline of the foundation basemats for the NI Common Basemat Structure, EPGBs, and ESWBs, along with the location of each foundation basemat.

Structures described within this section are not shared with any other power plant units (GDC 5).

A COL applicant that references the U.S. EPR design certification will describe site-specific foundations for Seismic Category I structures that are not described in this section.

3.8.5.1.1 Nuclear Island Common Basemat Structure Foundation Basemat

The NI Common Basemat Structure foundation basemat is a heavily reinforced concrete slab that supports the NI Common Basemat Structure Seismic Category I structures. The RCB and the RSB are located near the center of the NI Common Basemat Structure foundation basemat, and they are surrounded by the FB and the four SBs. The NI Common Basemat Structure foundation basemat is a cruciform shape that has outline dimensions of approximately 360 feet by 360 feet by 10 feet thick. The bottom of the NI Common Basemat Structure foundation basemat is founded approximately at elevation -41 feet and is embedded into the supporting soil approximately 40 feet. The NI Common Basemat Structure foundation basemat outline and section views are presented in Figures 3B-1, 3.8-11, 3.8-12, 3.8-13, 3.8-50, 3.8-51, 3.8-52, 3.8-63, 3.8-74, and 3.8-85.

The NI Common Basemat Structure foundation basemat provides anchorage of the vertical post-tensioning tendons in the RCB, which is described in Section 3.8.1. The portion of the NI Common Basemat Structure foundation basemat that is considered to provide support and anchorage for the RCB is the area under the circumference of the outer face of the RSB wall, as shown on Figure 3.8-11, Figure 3.8-12, Figure 3.8-13, and Figure 3.8-118. *[This portion of the NI Common Basemat Structure foundation basemat is designed in accordance with the ASME Code, Section III, Division 2.]** A circular gallery is provided beneath the NI Common Basemat Structure foundation basemat for maintenance access to the bottom of the vertical post-tensioning tendons provided in the RCB shell wall. The tendon access gallery is approximately 20 feet wide by 18 feet high, including an approximately 72 inch thick foundation slab under the gallery structure. The tendon gallery, which is integrally cast with the basemat, acts as a shear key and transfers lateral and vertical loads from the basemat into the soil. *[The walls and slab of the tendon access gallery are designed according to ACI 349.]**

Sections 3.8.1 and 3.8.3 describe the interface of the RCB containment liner plate and upper internal basemat above the liner for supporting the RB internal structures. Sections 3.8.4 describes the interface of the RSB, FB, and SBs with the NI Common Basemat Structure foundation basemat. Concrete walls and columns of these NI Common Basemat Structure Seismic Category I structures are anchored into the NI Common Basemat Structure foundation basemat with reinforcing bars to transmit vertical, horizontal, and bending moment loads into the basemat and to enhance the rigidity of the basemat.

Horizontal shear loads are transferred from the NI Common Basemat Structure foundation basemat to the underlying soil by friction between the bottom of the basemat, mud mat (or both), and the soil, and by passive earth pressure on the below-grade walls of the NI Common Basemat Structure Seismic Category I structures. In addition, the tendon gallery is classified as a Seismic Category I structure and analyzed as a shear key to transfer loads to the soil. Section 2.5.4.2 describes the friction coefficient properties of soil addressed for the U.S. EPR.

Buildings adjacent to the NI Common Basemat Structure are separated from the NI Common Basemat Structure foundation basemat to allow for differential seismic movements between buildings. Refer to Figure 3B-1, which illustrates the gaps between buildings.

3.8.5.1.2 Emergency Power Generating Buildings Foundation Basemats

Each EPGB foundation basemat supports a building superstructure and associated equipment. At the super-structure and foundation basemat interface, heavily reinforced concrete shear walls function as bearing walls to transfer loads from floors and the roof. Each foundation basemat is embedded approximately five feet into the supporting soil and has overall dimensions of approximately 178 feet long by 94.5 feet wide by 6 feet thick. Each foundation also has a system of shear keys as shown in Figure 3B-63—Emergency Power Generating Buildings Dimensional Plan Elevation 0 m (0 ft). In the areas of the two diesel fuel oil storage tanks, the foundation basemat reduces in width from 94.5 feet to 42 feet.

Figure 3.8-89 illustrates the general arrangement plan, which also shows the primary shear walls at column lines A, C, E, G and J in the east-west direction, and column lines 11, 13, 17 and 19 in the north-south direction. Additional figures, provided in

Appendix 3E, illustrate both the shear walls at the super-structure and foundation basemat interface and the foundation basemat reinforcement.

Figures 3.8-93 and 3.8-94 provide section views of the EPGB structure, which further clarify the relationship between the superstructure and the foundation basemat. Isometric views of the GT STRUDL model representing the overall structure are provided in Section 3.7.2.

3.8.5.1.3 Essential Service Water Buildings Foundation Basemats

The reinforced concrete foundation basemat for each ESWB supports the superstructure and water basin. At the super-structure and foundation basemat interface, heavily reinforced concrete shear walls function as bearing walls to transfer loads from the floors and the roof. Each foundation basemat is embedded approximately 21 feet into the supporting soil and has overall dimensions of approximately 164 feet by 108 feet wide by 6 feet thick.

Figures 3.8-101 and 3.8-102 provide cross-sections of the ESWB in each direction, illustrating the superstructure which bears on the foundation basemat. Figure 3.8-95 provides the general arrangement plan, which also illustrates the primary shear walls at column lines A, B, D and F in the east-west direction, and column lines 1, 2, 4 and 5 in the north-south direction. Additional figures provided in Appendix 3E illustrates both the shear walls at the super-structure and foundation basemat interface and the foundation basemat reinforcement. Isometric views of the GT STRUDL model representing the overall structure are provided in Section 3.7.2.

3.8.5.2 Applicable Codes, Standards, and Specifications

Applicable codes, standards, specifications, design criteria, regulations, and regulatory guides that are used for the design, fabrication, construction, testing, and inservice inspection of Seismic Category I foundations are the same as those in Section 3.8.4.2 (GDC 1, GDC 2, GDC 4 and GDC 5).

In addition, *[the portion of the NI Common Basemat Structure foundation basemat under the RCB/RSB is designed in accordance with the ASME Code, Section III, Division 2 for support and anchorage of the concrete RCB.]**

3.8.5.3 Loads and Load Combinations

Loads and load combinations for Seismic Category I foundations are the same as those in Section 3.8.4.3.

In addition to the loads addressed in Section 3.8.4.3, the NI Common Basemat Structure foundation basemat is designed for the loads and load combinations from the RCB as described in Section 3.8.1.3. The NI Common Basemat Structure foundation basemat provides for anchorage of the RCB vertical post-tensioning tendons, and the portion of the basemat under the RCB/RSB is designed to accommodate loads from containment.

[Loads and load combinations on Seismic Category I foundations are in accordance with ACI 349-01, RG 1.142, RG 1.199, and ANSI/AISC N690-1994, including Supplement 2 (2004) for steel structures (GDC 1, GDC 2, GDC 4 and GDC 5).] [Loads and load combinations on the portion of the NI Common Basemat Structure foundation basemat that supports the RCB/RSB are in accordance with the ASME Code, Section III, Division 2 and RG 1.136 (Exception: RG 1.136 endorses the 2001 Edition of the ASME Code with the 2003 addenda (including exceptions taken in RG 1.136). The U.S. EPR standard plant design is based on the 2004 Edition of the ASME Code, Section III, Division 2, inclusive of the exceptions taken in RG 1.136).]**

The NI Common Basemat Structure is a monolithic concrete structure. However, various portions of the structure have different classifications (i.e., RCB, RB internal structures, and other Seismic Category I structures) and correspondingly different design requirements, as shown in Figure 3.8-118. In some instances, the load combinations identified in SRP Section 3.8.5 do not include certain independent loadings which should be considered to account for potential structure-to-structure effects (i.e., the effect on one structure resulting from loadings applied to a separate, but monolithically connected, structure). To account for potential structure-to-structure effects, the loading combinations from SRP Section 3.8.5 are adjusted by including the necessary additional independent loadings. All load combinations include an additional hydrostatic load (F) while all sliding and overturning load combinations include an additional buoyant force (F_b). The load factors for hydrostatic load (F) and buoyant force (F_b) are matched to that of the dead load (D) for each loading combination.

In addition to the load combinations specified above, the following load combinations are applied for Seismic Category I foundations to consider sliding and overturning due to earthquakes, winds, hurricanes, and tornados and against flotation due to floods:

$D + H + W + F + F_b$ $\qquad$ $D + H + E' + F + F_b$

$D + H + W_t + F + F_b$ $\qquad$ $D + F_b + F$

where:

> F_b = the buoyant force of the design basis flood at maximum site water level. Refer to Section 3.8.4.3.1 for definitions of the other load parameters.

The U.S. EPR Seismic Category I foundations are also designed for the effects of short term and long term settlements. The settlement analysis is described in Section 3.8.5.4. Section 2.5 and Section 3.8.5.5 provide the settlement limits considered for the U.S. EPR.

There are no OBE loads applicable to the design of Seismic Category I foundations, since an OBE level of one-third the SSE has been selected. See Section 3.7 for a description of the OBE.

3.8.5.4 Design and Analysis Procedures

Design and analysis procedures are similar for the various Seismic Category I foundations but vary somewhat from structure to structure. The general analysis and design procedures applicable to Seismic Category I foundations are provided in the following sections. Procedures specific to the following Seismic Category I foundations also are described.

- NI Common Basemat Structure foundation basemat.
- EPGBs foundation basemats.
- ESWBs foundation basemats.

3.8.5.4.1 General Procedures Applicable to Seismic Category I Foundations

Concrete foundation basemats for Seismic Category I structures are analyzed as flat slabs on elastic supports to represent the underlying soil. Loads are applied to the foundation basemats by the interfacing reinforced concrete walls and structural steel columns that comprise the building structures being supported, as well as by equipment supported directly on the foundations. Intersecting concrete walls also serve to stiffen the foundation basemat slabs to increase resistance to bending moments resulting from soil pressures under the slabs. Foundations are analyzed for the various factored loads and load combinations identified in Section 3.8.5.3.

Seismic Category I foundation basemat structures transfer vertical loads from the buildings to the subgrade by direct bearing of the basemats on the subgrade. Horizontal shears, such as those produced by wind, hurricanes, tornados, and earthquakes are transferred to the subgrade by friction along the bottom of the foundation basemat, shear key, or by passive earth pressure.

The stability evaluations for the NI, EPGB, and ESWB are based on SSI analysis results, as described in Section 3.7.2.3. The coefficient of passive soil pressure corresponding to the sidewall movements into the soil are estimated from the SSI analysis and are used to calculate the passive soil pressure resisting sidewall movement.

Passive soil pressure capacities are based on constitutive models, typically used for granular media, such as Drucker-Prager or Coulomb-Mohr. For soil sites, a granular backfill material is used against side walls and underneath the structures, if needed. Backfill shall be installed to meet 95 percent of the Modified Proctor density (ASTM

D-1557 (Reference 66)). For rock sites, controlled low strength material, as described by ACI-229R (Reference 65), is specified on the faces of below grade walls. The tendon gallery acting as a shear key is backfilled with lean concrete. Cohesive materials will be addressed on a site-specific basis.

The wall pressures calculated from SSI analysis, elastic solution by Wood, and those required for sliding stability are considered in the design of embedded walls. Each soil case is analyzed, dynamically and statically, and design loads and controlling loads for each wall are used in the design.

The estimated maximum sidewall movement into the soil that results in the highest K_p value may not necessarily occur when the minimum factor of safety is calculated. Therefore, the minimum factor of safety is investigated using appropriate sidewall movements (using corresponding K_p) at the time of minimum sliding factor of safety.

Design and analysis procedures for Seismic Category I foundations are the same as those described in Sections 3.8.1.4 and 3.8.4.4 for the respective structures that apply loads on the foundations.

*[Seismic Category I concrete foundations are designed in accordance with ACI 349-01 and its appendices]** (GDC 1). Exceptions to code requirements specified in RG 1.142 are incorporated into the design and are accommodated in the loading combinations described in Section 3.8.5.3. *[In addition, the portion of the NI Common Basemat Structure foundation basemat that supports the RCB/RSB is designed in accordance with the ASME Code, Section III, Division 2 for support and anchorage of the concrete RCB]** as described in Section 3.8.1.

*[The design of concrete foundations for Seismic Category I structures is performed using the strength-design methods described in ACI 349-01.]** The ductility provisions of ACI 349-01 are satisfied to provide a steel reinforcing failure mode and to prevent concrete failure for design basis loadings.

Foundation design is performed for the spectrum of soil cases described in Section 3.7.1. Section 2.5 and Section 3.7 describe seismic parameters and design methods used for analyzing and designing Seismic Category I structures.

Soil-structure interaction and structure-soil-structure interaction effects are considered in the seismic analyses of Seismic Category I structures as described in Section 3.7.2. Figure 3B-1 illustrates separation distances between Seismic Category I structures upon which these interaction evaluations are based.

Dynamic bearing pressures are obtained from the SASSI analysis. Dynamic bearing pressures are calculated at each time step in the x and y input motion to satisfy the equilibrium of vertical forces and moments as a result of dead, live, buoyancy, precipitation, and seismic loads acting on the foundation for each soil case. Dead load pressures account for the deformation effect at the center and/or edges of the basemat.

The NI Common Basemat Structure and EPGB are designed for the static soil bearing pressures and dynamic bearing pressures in Sections 3.8.5.5.1 and 3.8.5.5.2. Accordingly, Seismic Category I foundations are sized and reinforced to accommodate these bearing pressure values.

The following criteria apply for load combinations for concrete and steel Seismic Category I foundations:

- Where any load reduces the effects of other loads, the corresponding coefficient for that load is 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads.
- For load combinations in which a reduction of the maximum design live load (L) has the potential to produce higher member loads and stresses, multiple cases are considered where the live load (L) is varied between its maximum design value and zero.
- For load combinations that include a tornado load (W_t), the tornado load parameter combinations described in Section 3.3 are used.

Loads and load combinations defined in Section 3.8.5.3 are used to determine strength requirements of members and elements of Seismic Category I foundations. Concrete and steel structural elements and members are designed for axial tension and compression forces, bending moments, torsion, and in-plane and out-of-plane shear forces for the controlling loading combinations that are determined from analysis. Concrete and steel members and elements remain elastic for loadings other than impact. Local yielding is permitted for localized areas subjected to hurricane- or tornado-generated missile loads, pipe break accident loadings, and beyond design basis loadings. The structural integrity of members and elements is maintained for the loading combinations described in Section 3.8.5.3.

For the loading combinations identified in Section 3.8.5.3, the minimum factors of safety required to prevent sliding and overturning are specified in Table 3.8-11—Minimum Required Factors of Safety Against Overturning, Sliding, and Flotation for Foundations.

Normal lateral earth pressure loads consider saturated soil up to a groundwater elevation of -3.3 feet relative to site finished grade. Lateral soil loads due to external floods consider saturated soil up to elevation -1.0 feet relative to site finished grade. Seismic loads from all three components of the earthquake motion are combined using the SRSS method. The SSE components of soil loads are determined using densities for saturated soil to account for the weight of the soil plus the weight of either normal or flood water levels. Earthquake-induced lateral soil pressures are obtained from SSI analyses for NI common basemat structures, EPGB, and ESWB. The design of embedded elements, such as embedded walls on basemats, assumes that the lateral pressure due to the SSE is in phase with the inertial loads. In cases where passive pressure is assumed to act on embedded structures in the stability check against sliding, the walls of the structure are evaluated to withstand such earth pressure. Section 3.8.4.4.2, Section 3.8.4.4.3, and Section 3.8.4.4.4 provide further information on how seismic-induced lateral earth pressures are determined for the NI Common Basemat Structure, EPGB, and ESWB, respectively. These lateral load effects are considered in structure sliding and overturning analyses. Refer to Section 2.5.4.2 for the soil parameters used to determine soil loads and lateral earth pressure.

When the effects of vertical seismic acceleration are included in the stability check against sliding, the unfactored dead weight of the structure is used to calculate the resistance to sliding due to friction.

Buoyancy effects of saturated soil due to a groundwater level of elevation -3.3 feet below finished grade or to a flood water level of elevation -1.0 feet below finished grade are considered when performing sliding and overturning analyses. For uplift evaluations (i.e., flotation and seismic overturning), dead load includes the weight of water permanently stored in pools and tanks.

A differential settlement evaluation is performed for the Seismic Category I structures considering both short term (elastic) and long term (heave and consolidation) effects. The effects of differential foundation settlements are applied concurrently with the dead load using the same load factors. The U.S. EPR design requires separate Seismic Category I structures to be connected by site-specific designed Seismic Category I umbilicals (i.e., ductbank, embedded piping, and/or structural galleries containing piping, cable tray, and/or ductwork). The effects of site-specific differential settlement between the individual U.S. EPR Seismic Category I structures and the site-specific Seismic Category I umbilicals will be considered in the design of the connections and the construction sequence. See Section 3.8.4.4.5 for analysis and design procedures for Seismic Category I buried items that interface with structures on separate foundations.

3.8.5.4.2 Nuclear Island Common Basemat Structure Foundation Basemat

Model

The details of the 3D FEM NI Common Basemat Foundation Models (dynamic and static) including the types of finite elements used are described in Section 3.7.2.3.1.4. The NI Common Basemat Structure Foundation Model includes the RCB, RB internal structures, RSB, FB, and SBs, as well as the NI Common Basemat Structure foundation basemat and subgrade.

Figure 3.7.2-151—Solid Element Basemat shows the basemat portion of the full model, and Figure 3.7.2-152—Foundation Basemat Model with Solid Element Basemat shows the full model, including the NI superstructure.

Springs are used to represent soil that provides support for the concrete foundation basemat in the ANSYS model. These springs represent the compressibility of the soil and were developed to reflect the pressure distribution under the NI Common Basemat Structure. Springs values vary for each soil case and are based on the soil properties delineated in Section 2.5 and Table 3.7.1-6. The distribution used is elliptical in nature and takes the form of:

$$K(x,y) = K_o[A - B^*(1 - x^2/2l^2 - y^2/2b^2)^{1/2}]$$

where:

$K(x, y)$ is the subgrade modulus at x, y corrected for mat stiffness (pounds/ft^2 per foot)

K_o is the weighted average subgrade modulus (pounds/ft^2 per foot)

A & B are constants for a soil type based on its properties, bearing pressure distribution and shape of the foundation.

x = is the coordinate in the length direction of the Foundation Mat (feet)

y = is the coordinate in the width direction of the Foundation Mat (feet)

b = half width of foundation

l = half length of foundation.

The Gazetas equation (Reference 57) was used to evaluate the total soil spring (K_o) for the design of the foundation basemat of the NI Common Basemat Structure. Although Gazetas addresses the dynamic stiffness of the foundation basemat, the use of one-half the dynamic shear modulus in the equation approximates the total stiffness of the supporting soil medium under static conditions. Table 3.8-13—Static Spring Distribution provides the distribution equations and K_o values for each soil case.

The pressure on the buried outer walls that are in contact with soil/rock is a function of the relative movement between the structure and the surrounding soil. The sidewall pressures are idealized by nonlinear sidewall soil/rock springs that represent the following three states: active, passive and at-rest. In ANSYS, the sidewall springs are modeled using a combination of two elements: a linear 3-D truss element (LINK8) and a non-linear spring element (COMBIN39). The at-rest earth pressure is applied as a preload to the 3-D truss element and the forces developed due to the wall movement (towards and away from the soil mass) are modeled using the nonlinear spring element which is capable of idealizing the different force-deflection curves on the active and passive side. The pressures at a sidewall node are multiplied by the tributary area of the sidewall node to define the sidewall force versus the deflection behavior of a particular sidewall spring.

Linear bi-directional static springs that have a stiffness of one-half the dynamic springs as specified in Table 3.8-13 are applied to the base of the NI Common Basemat Structure. Sliding and uplift are not modeled in the static analysis. Figure 3.8-108—Elastic Displacement for Soil Case 1n2u, Figure 3.8-110—Elastic Displacement for Soil Case 4u, Figure 3.8-111—Elastic Displacement for Soil Case 5a, Figure 3.8-112—Elastic Displacement for Soil Case 1n5a, and Figure 3.8-113—Elastic Displacement for Soil Case 2sn4u illustrate elastic displacements, from dead load + 0.25* live load + 0.75* precipitation load + hydrostatic forces and at-rest earth pressure, using the springs listed in Table 3.8-13.

Analysis

The ANSYS basemat model is loaded statically by accelerating the lumped and distributed masses described in Section 3.7.2.3.1.2 before a nonlinear time-history analysis is performed. The initial conditions (dead load, 25% live load, 75% precipitation load, hydrostatic forces and at-rest earth pressures) to the basemat foundation model (nonlinear) are input by performing multiple static analysis load steps prior to the start of the dynamic load. Static load steps are performed in a transient analysis by turning off the transient time integration effects. The static analysis time-steps are performed at solution times less than 0.005sec. The transient itself is started by turning on the time integration effects at time = 0.005sec to the end of the acceleration time-history input.

The seismic input motions are in-column ground motions obtained from SHAKE91 analysis runs at the bottom of the NI Common Basemat foundation level in the three translational directions derived using the NEI approach in Section 2.5.2.6.

The seismic time-history analysis starts from time = 0.005 sec. Thus, effects of the seismic loads are obtained by subtracting the results at time-history data points with the static analysis baseline results. The maximum seismic loads are obtained by determining the maximum/minimum design load values for basemat and tendon gallery for each of the elements/nodes over all time points of the transient analysis.

In addition to the seismic load, the basemat foundation model is analyzed (with static soil springs) for various static load cases: normal loads (e.g., dead, live, soil/lateral earth pressure, thermal load, pipe reaction, post-tension loads, relief valve loads), construction loads, test loads for reactor containment building, severe environmental loads (e.g., wind), extreme environmental loads (e.g., tornado and hurricane), abnormal loads (e.g., internal flood, buoyant pressure, accident pressure).

Design Considerations

Section 3.8.1, Section 3.8.3, and Section 3.8.4 provide descriptions of interfacing structures that induce loads on the NI Common Basemat Structure foundation basemat. The figures in those sections illustrate the concrete shear walls and columns that transfer loads to the NI Common Basemat Structure foundation basemat. The tendon gallery beneath the NI Common Basemat Structure foundation basemat is relied upon as a shear key to aid in resisting lateral forces on the basemat.

The SSI analysis, described in Section 3.7.2.4, is a frequency domain linear seismic analysis. The additional loads due to the nonlinearities of basemat uplift and sliding obtained in the 3D basemat FEM is considered for the design of the tendon gallery and NI embedded walls. The additional (delta) loads, generated on the tendon gallery walls due to sliding, are calculated by performing additional analyses without allowing for sliding and uplift behavior and comparing the results (sidewall pressures and design forces and moments) to the analysis that includes the nonlinear effects. When nonlinear responses are observed in the model, the increase in loading is added to the SSI results described in Section 3.7.2.4 for the design of tendon gallery and NI embedded walls.

In the design of the NI embedded walls and tendon gallery, the static soil pressure (earth pressure at rest) and effects of surcharge due to the weight of adjacent buildings (NI for the case of the tendon gallery) are applied as a separate load case. The dynamic load case corresponds to the passive pressures generated on the walls during the SSE condition.

The passive soil pressures on the NI embedded walls (excluding the tendon gallery) are calculated using the results from the SSI analysis (see Section 3.7.2.4). The SSE wall pressures are scaled up such that the maximum pressure on each wall is, at least, equal to the passive earth pressure obtained with $K_p = 3$. The dynamic load case corresponding to scaled SSI pressures and delta pressures due to uplift and sliding of the basemat is applied as a separate load case. The static and dynamic load cases are then combined in the appropriate load combinations to arrive at the design forces and moments of the walls. The above procedure is used for all soil cases except 5ae. For 5ae (rock case), the nodes in contact with the excavation are laterally constrained to obtain design forces and moments of the walls.

The passive soil pressures and seismic design loads on the tendon gallery walls for all cases including the 5ae case are directly obtained from the nonlinear analysis of finite element model for NI Common Basemat Foundation described in Section 3.7.2.3.1.4. These loads include the sidewall and delta pressures due to uplifting and sliding of the basemat. The seismic loads are combined with other static analysis load cases as described in Section 3.8.1 through Section 3.8.4 to obtain the design forces and moments for the tendon gallery.

Based on the results (shears and moments) from the static and dynamic analysis of the basemat foundation model described above, the basemat is designed for the combined effect of the various load cases. Section 3.8.1 through Section 3.8.4 list the appropriate load combinations to be used for the Seismic Category I structures.

A differential settlement evaluation is performed for the NI common basemat structure considering both short term (elastic) and long term (heave and consolidation) effects. The evaluation accounts for the construction sequence, building stiffness, and time duration for loading the NI common basemat structure. The evaluation considers a soft soil site consistent with the soft soil case, 1n2ue, addressed in Table 3.7.1-6. A comparison of the angular distortion (measure of curvature) of the basemat for various soil cases demonstrates that the soft soil site will control the design for settlement.

The resulting forces and moments throughout the structure are captured by applying soil springs to the 3D finite element structural model of the basemat and superstructure used for designing the basemat. The soil springs are developed to capture the short and long term responses of the soil.

A construction sequence is evaluated for the NI common basemat structure, which assumes that the concrete for the mat foundation is in a single placement prior to the start of placement of concrete for the superstructure. It is assumed that concrete placement for the superstructure continues so that the superstructure is erected uniformly.

The construction sequence considers 11 steps for the NI common basemat structure:

1. Basemat only.
2. Walls up to elevation -16 ft.
3. Floor slabs at elevation -16 ft.
4. Walls up to grade elevation.
5. Floor slabs at grade elevation.
6. Walls up to elevation 55 ft.
7. Floor slabs at elevation 55 ft.
8. Walls up to elevation 96 ft.
9. Floor slabs at elevation 96 ft.
10. Walls up to elevation 144 ft.

11. Remaining structure up to elevation 204 ft.

Soil springs are applied to the 3D finite element superstructure and basemat structural models to determine the displacement of the basemat and capture the resulting locked-in forces and moments throughout the structure at each construction step. The soil springs are developed using the PLAXIS 3D foundation (Plaxis 3D) software. The Plaxis 3D subgrade modulus K is determined using the following equation:

$$K = \sigma'_{yy} / \delta$$

where σ'_{yy} is the vertical effective stress, and δ is the vertical deformation.

Two sets of soil springs are developed using Plaxis 3D. The first set of soil springs is developed with the geometry and loading of the basemat only. The second set of soil springs is developed with the geometry and loading of the full NI superstructure. Each set of soil springs is developed by iterating on settlement results between a full 3D finite element structural model of the NI common basemat structure with Winkler springs and results from the Plaxis 3D model. The Plaxis 3D model plate thicknesses are adjusted and soil springs are developed for each iteration as previously described. The distribution of the soil springs matches the distribution with the stiffness of the NI common basemat completed structure considering the full concrete elastic modulus, Ec. The soil springs are applied to the 3D finite element structural model until a good fit (less than 10 percent difference) is observed between settlements generated by both the 3D finite element structural model and Plaxis 3D model.

The Plaxis 3D model assumes a sandy material with laterally uniform soil stiffness. The effects from the adjacent structures are considered in the development of the second set of soil springs. The Plaxis 3D analysis also includes the settlement effects due to consolidation. Beyond construction, the long term settlements due to rewatering, creep, and dissipation of any excess remaining pore pressure is assumed to be negligible.

The 11 steps in the construction sequence are evaluated for each set of soil springs. At each construction step in the 3D FEM structural evaluation, 100 percent of the dead load, 25 percent of the live load, and 75 percent of the precipitation loads are applied to determine locked-in forces and moments for structural elements.

The full Ec and section modulus are used for hardened concrete. In the basemat evaluation, the soil material will experience initial displacement; however, the basemat will not initially experience the assumed linear stress increase because the concrete is still plastic. Therefore, using the full Ec value is considered conservative when calculating stresses for the initial basemat evaluations.

For the superstructure elements, the walls and slabs are added in a stepwise manner as wet concrete. At each step, the effects of the added mass are considered by reducing

Ec to 0.1 x Ec for the superstructure elements. The section properties are converted back to the full Ec prior to evaluating the next step.

The basemat and superstructure forces and moments are captured at each construction step and an enveloping settlement load file is developed. A comparison is made of the enveloping settlement load file results from each set of soil springs. The set of soil springs which control the design of forces and moments due to settlement are used on the design.

The NI superstructure design is performed with fixed base models. The additional forces and moments due to settlement are added to each of the design soil cases. The fixed base models already include the results from dead weight, live load, and precipitation loads. To capture the effects of only the differential settlement, a comparison is made between the settlement load results and the fixed base results with the same load combination (i.e., 100 percent of the dead load, 25 percent of the live load, and 75 percent of the precipitation loads). A single and separate load file is developed and added to each NI superstructure fixed base static model analyses in the load combinations with a dead load (i.e., the load factor used corresponds to the dead load factor).

The forces and moments due to settlement in the basemat model are determined similar to the approach used for the NI superstructure. The basemat model node numbering and meshing is different from the NI superstructure model. The settlement analysis is performed with the basemat model to allow mapping results directly to the model using the same nodal and element geometry. The basemat model settlement analysis is performed by applying the settlement soil springs to the basemat model for each of the 11 construction sequence steps. The basemat model settlement analysis is then performed for each of the 11 construction steps applying the elastic soil springs developed for each generic FSAR soil case.

For a given soil case, a comparison is made with the basemat forces and moments from the elastic soil spring case, and the forces and moments from the settlement springs at each of the 11 construction steps to develop a differential set of forces and moments in the basemat for each step. An enveloping differential load file is created which consists of the maximum differential forces and moments in each basemat element from each construction steps.

Following this same approach, an enveloping differential load file is created for each soil case and added to the elastic soil spring analysis results in the load combinations with a dead load (i.e., the load factor used corresponds to the dead load factor).

The basemat design includes symmetrical main reinforcing steel in each direction and on each face to control development of any large cracks in the basemat.

Relative differential settlement contours are developed for each construction step using the second set of soil springs. The contours are relative to the minimum settlement value determined under the NI common basemat structure, and are shown in Figure 3.8-124 through Figure 3.8-134.

Detailed analysis and design procedures are described in the critical sections presented in Appendix 3E.

Section 3.8.3 provides a description of analysis and design of the RB internal structures basemat, which is located above the containment liner plate.

Stability Evaluation

The NI stability analysis using seismic reaction forces from the SSI model addressed in Section 3.7.2 considers the soil cases in Table 3.7.1-6. The soil bearing pressures directly beneath the foundation basemat are based on the SASSI analysis described in Section 3.7.2.4 and reported in Appendix 3E Table 3E.1-5.

3.8.5.4.3 Emergency Power Generating Buildings Foundation Basemats

Shear loads are transferred from the EPGB foundation basemat to the underlying soil by friction between the bottom of the basemat, mud mat, and the soil, and by passive earth pressure.

The EPGB foundation basemat is analyzed and designed using the GT STRUDL v.29.1 finite element analysis code. The FEM contains both the building superstructure (i.e., reinforced concrete walls and elevated slabs) as well as the foundation basemat. Analysis of the EPGB includes all applicable design loads and design load combinations described in Section 3.8.4.3. Figure 3.8-104—Emergency Power Generating Building Foundation Basemat Model illustrates the foundation basemat portion of the overall EPGB FEM.

The GT STRUDL FEM representing the EPGB foundation basemat consists of SBHQ6 rectangular elements, each with six degrees of freedom. This element type is capable of capturing both in-plane and out-of-plane behavior. Elastic boundary conditions are included in the FEM in order to simulate the stiffness of the supporting soil. Basemat flexibility and SSI are addressed by inclusion of the basemat section properties and aforementioned soil spring boundary conditions in the FEM.

The foundation basemat is included in the overall GT STRUDL FEM used for static analysis of the foundation basemat, along with compression-only soil springs representing static soil stiffness properties in Table 3.8-19. Soil spring development and distribution methodologies are the same as those used for the NI soil cases and are described in Section 3.8.5.4.2. Compression-only effects are included in the boundary conditions in order to capture uplift effects induced by extreme event loading (e.g., SSE). Illustrations of the complete FEM representing the EPGB are provided in Section 3.7.2.

The effect of settlement on the EPGB considers a soft soil site consistent with a soft soil case as shown in Table 3.7.1-6. Soil springs are developed to consider both short term (elastic) and long term (heave and consolidation) effects. The 3D finite element models of the EPGB basemat and superstructure are used in a static structural analysis with elastic soil springs applied in an elliptical distribution. The consolidation effects are approximated by further softening the elastic soil spring stiffness by a factor of two. A settlement load file is created considering 100 percent of the dead load, 25 percent of the live load, and 75 percent of the precipitation loads to determine locked-in forces and moments for all structural elements. The full Ec and section modulus is used in the EPGB settlement analysis. A check is conducted to determine if the basemat concrete has cracked during development of the load file. If the basemat concrete has cracked, a cracked section modulus is used to develop the forces and moments. The basemat design includes symmetrical main reinforcing steel in each direction and on each face to account for any additional lateral variability in the soil properties and to control development of any large cracks in the basemat.

The total differential settlement contour is developed for the EPGB as shown in Figure 3.8-135.

Detailed analysis and design procedures are described in the critical sections presented in Appendix 3E for the EPGBs.

3.8.5.4.4 Essential Service Water Building Foundation Basemats

Horizontal shear loads are transferred from the ESWB foundation basemat to the underlying soil by friction between the bottom of the basemat, mud mat, and the soil. In addition, dynamic soil pressure and passive earth pressure have been considered for the below-grade walls, reflecting the total embedment depth of nominally 21 feet.

Similar to the approach for the EPGB, the foundation basemat is analyzed and designed using the GT STRUDL v.29.1 finite element analysis code. The FEM contains both the building superstructure (i.e., reinforced concrete walls, slabs, and beams) and the foundation basemat. Analysis of the ESWB includes all applicable design loads and design load combinations described in Section 3.8.4.3. Figure 3.8-105—Essential Service Water Building Foundation Basemat Model illustrates the foundation basemat portion of the overall ESWB FEM.

The GT STRUDL FEM representing the ESWB foundation basemat consists of SBHQ6 rectangular elements, each with six degrees of freedom. This element type is capable of capturing both in-plane and out-of-plane behavior. Elastic boundary conditions are included in the FEM in order to simulate the stiffness of the supporting soil. Basemat flexibility and SSI are addressed by inclusion of the basemat section properties and aforementioned soil spring boundary conditions in the FEM. Illustrations of the complete FEM representing the ESWB are provided in Section 3.7.2.

The effect of settlement on the ESWB structure considers a soft soil site consistent with a soft soil case as shown in Table 3.7.1-6. Soil springs are developed to consider both short term (elastic) and long term (heave and consolidation) effects. The 3D FEM of the ESWB basemat and superstructure are used in a static structural analysis with elastic soil springs applied in an elliptical distribution. The consolidation effects are approximated by further softening the elastic soil spring stiffness by a factor of two. A settlement load file is created considering 100 percent of the dead load, 25 percent of the live load, and 75 percent of the precipitation loads to determine locked-in forces and moments for all structural elements. The full Ec and section modulus is used in the ESWB settlement analysis. A check is conducted to determine if the basemat concrete has cracked during development of the load file. If the basemat concrete has cracked, a cracked section modulus is used to develop the forces and moments. The basemat design includes symmetrical main reinforcing steel in each direction and on each face to account for any additional lateral variability in the soil properties and to control development of any large cracks in the basemat.

The total differential settlement contour is developed for the ESWB as shown in Figure 3.8-136.

Detailed analysis and design procedures are described in the critical sections presented in Appendix 3E for the ESWBs.

3.8.5.4.5 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.4, 2.5, 3.3, 3.5, 3.7, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections. A cross-reference between U.S. EPR FSAR sections and information required by SRP Section 3.8.4 Appendix C is provided in Table 3.8-17.

3.8.5.5 Structural Acceptance Criteria

*[Limits for allowable stresses, strains, deformations, and other design criteria for Seismic Category I concrete foundations are in accordance with ACI 349-01 and its appendices]** (GDC 1, GDC 2 and GDC 4). Limits for concrete design include the exceptions specified in RG 1.142. In addition, the portion of the NI Common Basemat Structure foundation basemat that supports the RCB/RSB is in accordance with the ASME Code and RG 1.136 for containment loadings as described in Section 3.8.1.

[Limits for the allowable stresses, strains, deformations, and other design criteria for structural steel elements of Seismic Category I foundations are in accordance with

*ANSI/AISC N690-1994 (R2004), including Supplement 2]** (GDC 1, GDC 2 and GDC 4).

The design of Seismic Category I foundations is generally controlled by load combinations containing SSE seismic loads. Stresses and strains are within the ACI 349-01 limits, with the exceptions previously listed. *[Limits for allowable loads on concrete embedments and anchors are in accordance with Appendix D of ACI 349-06 (Appendix D with exception stated in Section 3.8.1.2.1) and guidance given in RG 1.199 (with exception described in Section 3.8.1.4.10).]** Portions of the NI Common Basemat Structure foundation basemat that support the RCB/RSB are within the limits in accordance with ASME Code, Section III, Division 2.

Seismic Category I foundations are required to satisfy the factors of safety against overturning, sliding, and flotation defined in Table 3.8-11. The calculated minimum factors of safety for the NI Common Basemat Structure are provided in Table 3E.1-4—Minimum Factors of Safety for the Nuclear Island Common Basemat Structure.

Acceptance criteria for soil conditions for the media supporting Seismic Category I foundations are addressed in Section 2.5.

Acceptance criteria for tilt settlement for Seismic Category I foundations are addressed in Section 2.5.

The acceptance criteria for differential settlement of Seismic Category I foundations are based on the site- specific predicted angular distortion, as described in U.S. Army Engineering Manual 1110-1-1904 (Reference 67). Predicted angular distortion is compared to the angular distortion throughout the basemat in both the east-west and north-south directions in the differential settlement contours. If the predicted angular distortion of the basemat of Seismic Category I structures is less than the angular distortion shown, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

Additional acceptance criteria for critical areas of these structures are described in Appendix 3E.

An as-built report is prepared to summarize deviations from the approved design and confirm that the as-built Seismic Category I foundations are capable of withstanding the design basis loads described in Section 3.8.5.3 without loss of structural integrity or safety-related functions.

A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for site-specific soil characteristics that are not within the envelope of the soil parameters specified in Section 2.5.4.2.

3.8.5.5.1 Nuclear Island Common Basemat Structure Foundation Basemat

Appendix 3E.1 provides details of the design of the NI Common Basemat Structure foundation basemat critical areas.

The static and dynamic bearing pressures for the NI Common Basemat Structure foundation basemat are specified in Table 3E.1-5. The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1. The factors of safety for each analysis case are provided in Table 3E.1-4

A differential settlement evaluation is performed for the NI common basemat structure considering both short term (elastic) and long term (heave and consolidation) effects. The evaluation accounts for the construction sequence, building stiffness, and time duration of loading of the NI common basemat structure. The resulting forces and moments throughout the structure are captured by application of soil springs to the same 3D finite element structural model of the basemat and superstructure that is used for design of the basemat. The total settlement for the NI common basemat is expected to be between 0 inches for a hard rock site and up to 5 inches for a soft soil site.

A COL applicant that references the U.S. EPR design certification will compare the NI common basemat site-specific predicted angular distortion to the angular distortion in the relative differential settlement contours in Figure 3.8-124 through Figure 3.8-134, using methods described in U.S. Army Engineering Manual 1110-1-1904. The comparison is made throughout the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of NI common basemat structure is less than the angular distortion shown for each of the construction steps, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

For worst-case loading combinations on the RB internal structures basemat above the containment liner, the minimum safety factor against sliding is 2.8 and the minimum safety factor against overturning is 1.9.

3.8.5.5.2 Emergency Power Generating Buildings Foundation Basemats

Appendix 3E.2 provides details of the design of the EPGB foundation basemats critical sections.

The static and dynamic bearing pressures for the EPGB foundation basemat are provided in Table 3E.2-2. The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1. The factors of safety for each analysis case are provided in Table 3E.2-1.

A differential settlement evaluation is performed for the EPGB structure considering both short term (elastic) and long term (heave and consolidation) effects.

A COL applicant that references the U.S. EPR design certification will compare the EPGB site-specific predicted angular distortion to the angular distortion in the total differential settlement contours in Figure 3.8-135, using methods described in U.S. Army Engineering Manual 1110-1-1904. The comparison is made throughout the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of EPGB structures is less than the angular distortion shown, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

3.8.5.5.3 Essential Service Water Building Foundation Basemats

Appendix 3E provides details of the design of the ESWB foundation basemats critical sections.

Maximum soil bearing pressures under the ESWB foundation basemat are 17,800 pounds per square foot for static loading conditions, and 28,200 pounds per square foot for dynamic loading conditions. For uniformity of site characteristics, the required bearing capacity will be the same as for the NI. The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1.

A differential settlement evaluation is performed for the ESWB structure considering both short term (elastic) and long term (heave and consolidation) effects.

A COL applicant that references the U.S. EPR design certification will compare the ESWB site-specific predicted angular distortion to the angular distortion in the total differential settlement contours in Figure 3.8-136, using methods described in U.S. Army Engineering Manual 1110-1-1904. The comparison is made throughout the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of ESWB structures is less than the angular distortion shown, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control programs and special construction techniques used in the fabrication and construction of Seismic Category I foundations.

3.8.5.6.1 Materials

Concrete, reinforcing steel, and structural steel materials for Seismic Category I foundations have been used in other nuclear facilities and are the same as described in

Section 3.8.3.6 (GDC 1), except as follows:

- Materials for the portion of the foundation basemat that supports the RCB/RSB are the same as described in Section 3.8.1.6.
- Structural concrete used in the construction of Seismic Category I foundations has a minimum compressive strength of 4000 psi (f'_c) at 90 days.
- Waterproofing and dampproofing systems are addressed in Section 3.4.2.
- *[Concrete exposed to aggressive environments, as defined in ACI 349-01, Chapter 4, shall meet the durability requirements of ACI 349-01 Chapter 4 or ASME Section III, Division 2, Article CC-2231.7, as applicable. In addition, epoxy coated reinforcing steel will be considered, on a site specific basis, for use in foundations subjected to aggressive environments. For epoxy coated reinforcing steel, the required splice length is increased in accordance with ACI 349-01 specifications.*
- *The waterproofing and dampproofing system of all below-grade Seismic Category I structures subjected to aggressive environments, as defined according to ACI 349-01, Chapter 4, shall be evaluated for use in such environments.]**

The waterproofing and dampproofing system will provide adequate frictional characteristics, as specified inTable 2.1-1. This characteristic will be demonstrated by vendor testing. The contact surface between the waterproofing or dampproofing system and the concrete will be finished in accordance with manufacturer recommendations.

A COL applicant that references the U.S. EPR design certification will evaluate the use of epoxy coated rebar for foundations subjected to aggressive environments, as defined in ACI 349-01, Chapter 4. In addition, waterproofing and dampproofing systems of Seismic Category I foundations subjected to aggressive environments will be evaluated for use in aggressive environments. Also, the concrete of Seismic Category I foundations subjected to aggressive environments will meet the durability requirements of ACI 349-01, Chapter 4 or ASME, Section III, Division 2, Article CC-2231.7, as applicable.

3.8.5.6.2 Quality Control

Quality control procedures for Seismic Category I foundations are the same as described in Section 3.8.3.6 (GDC 1).

3.8.5.6.3 Special Construction Techniques

Seismic Category I foundations are constructed using proven methods common to heavy industrial construction. No special, new, or unique construction techniques are used.

Modular construction methods are used to the extent practical for prefabricating portions of reinforcing and concrete formwork. Such methods have been used extensively in the construction industry. Rigging is pre-engineered for heavy lifts of modular sections.

3.8.5.7 Testing and Inservice Inspection Requirements

Monitoring and maintenance of Seismic Category I foundations is performed in accordance with 10 CFR 50.65 and supplemented with the guidance in RG 1.160 (GDC 1).

Additional testing and surveillance requirements for the portion of the foundation basemat that supports the RCB/RSB are the same as described in Section 3.8.1.7.2.

Physical access is provided to perform inservice inspections of exposed portions of Seismic Category I foundations.

A COL applicant that references the U.S. EPR design certification will identify site-specific settlement monitoring requirements for Seismic Category I foundations based on site-specific soil conditions.

If the monitoring program indicates actual settlement values are not following predicted settlement values during construction, condition specific evaluations or actions will be required. This may include adjusting the construction sequence or schedule, or evaluation of the existing conditions to demonstrate that the resulting moments and forces imposed on the structure are acceptable.

A COL applicant that references the U.S. EPR design certification will describe the program to examine inaccessible portions of below-grade concrete structures for degradation and monitoring of groundwater chemistry.

3.8.6 References

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Table 3.8-1—Thermal Properties for Heat Transfer Analysis-Reactor Containment Building

Material Property	Concrete	Steel
Thermal conductivity (kW/m*C)	0.0023	0.041
Specific heat (J/kg*C)	1000	434

Table 3.8-2—Material Properties – Reactor Containment Building

Material	Modulus of Elasticity (ksi)	Poisson Ratio	Nominal Strength (ksi)	Unit Weight (lb/ft^3)
Concrete	4,769	0.17	f'_c=7.0	150
Post Tensioning Cable	28,000	0.30	F_{pu} =270	490
Reinforcing Bar	29,000	0.30	F_y = 60	490

Table 3.8-3—Tendon Frictional Losses

Tendon	K (per foot) Wobble Loss	μ (per radian) Curvature Loss
Hoop	0.00050	0.18
Vertical	0.00025	0.16
Gamma	0.00037	0.16

Table 3.8-4—Thermal Properties – Reactor Containment Building

Specific Heat (Btu/lb$_m$*°F)	Thermal Conductivity (BTU/hr*ft*°F)	Film Coefficient (BTU/hr*ft^2*°F)	Thermal Diffusivity (ft^2/hr)
0.24	1.33	∞ (Inside Containment) 1.41 (Outside Containment)	0.037

Table 3.8-5—Tendon Losses and Effective Forces with Time

		Hoop Tendons				Vertical Tendons				Gamma Tendons			
		T = zero[1] (plant startup)		T = 60 years[1] (plant shutdown)		T = zero[1] (plant startup)		T = 60 years[1] (plant shutdown)		T = zero[1] (plant startup)		T = 60 years[1] (plant shutdown)	
Description		Value (ksi)	% loss[2]	Value (ksi)	% loss[2]	Value (ksi)	% loss[2]	Value (ksi)	% loss[2]	Value (ksi)	% loss[2]	Value (ksi)	% loss[2]
Tension at lock off	$f_{p\text{-}ini}$	197.10	N/A	197.10	N/A	197.10	N/A	197.10	N/A	197.10	N/A	197.10	N/A
Loss due to friction	$\Delta f_{p\text{-}fr}$	-42.85	-21.7%	-42.85	-21.7%	-5.54	-2.8%	-5.54	-2.8%	-4.95	-2.5%	-4.95	-2.5%
Loss due to elastic shortening	$\Delta f_{p\text{-}es}$	-7.46	-3.8%	-7.46	-3.8%	-3.35	-1.7%	-3.35	-1.7%	-4.59	-2.3%	-4.59	-2.3%
Loss due to creep	$\Delta f_{p\text{-}cr}$	-16.24	-8.2%	-17.92	-9.1%	-9.80	-5.0%	-10.92	-5.5%	-15.05	-7.6%	-16.71	-8.5%
Loss due to shrinkage	$\Delta f_{p\text{-}sh}$	-5.60	-2.8%	-5.60	-2.8%	-5.60	-2.8%	-5.60	-2.8%	-5.47	-2.8%	-5.60	-2.8%
Loss due to steel relaxation	$\Delta f_{p\text{-}sr}$	-9.45	-4.8%	-11.96	-6.1%	-12.12	-6.2%	-15.34	-7.8%	-12.08	-6.1%	-15.29	-7.8%
Sum of all losses	$\Sigma\Delta f_p$	-81.60	-41.3%	-85.79	-43.5%	-36.41	-18.5%	-40.75	-20.7%	-42.14	-21.4%	-47.14	-23.9%
Effective Post Tension	$f_{p\text{-}eff}$	115.50	N/A	111.31	N/A	160.69	N/A	156.35	N/A	154.96	N/A	149.96	N/A

Notes:

1. To account for the length of the construction period and the relative ages of the RCB wall, dome and tendons, the tendon losses above assumed that at "T=zero (plant startup)," the age of the tendons is 4.0 years, the age of the RCB dome is 4.1 years, and the age of the RCB wall is 5.0 years.

2. The "% loss" values are calculated relative to the tension at lock off (197.10 ksi).

Table 3.8-6—Containment Ultimate Pressure Capacity (P_u) at Accident Temperature of 309°F

Sections	Pressure Capacity (P_u)		Controlling Mode/Location
	Pu (psi)	Minimum Ratio P_u/P_d (2)	
Cylinder (Hoop)	272	4.39	Maximum allowable membrane strains away from structural discontinuities.
Dome	208	3.35	Maximum allowable membrane strains away from structural discontinuities.
Dome Belt	211	3.40	Maximum allowable flexural strains at structural discontinuities.
Gusset Base	316	5.10	Maximum allowable flexural strains at structural discontinuities.
Equipment Hatch	128.5	2.07	ASME Code Level D Stability/ Buckling limit in the equipment hatch cover.
Construction Opening Closure	118.5	1.91	ASME Code Level D allowable pressure to ensure no stability/ buckling of the opening cover.
Personnel Airlocks	119.4	1.93	ASME Code Level D allowable pressure to ensure no stability/ buckling of the airlock hatch cover.
Fuel Transfer Tube	155(3)	2.5(3)	High strains in the containment sleeve portions not backed by concrete.
Main Steam and Feedwater Line Penetrations	155(3)	2.5(3)	High strains in the containment sleeve portions not backed by concrete.

Notes:

1. Deleted.
2. P_d – design pressure.
3. The ultimate pressure capacity is reported at 2.5 times the design pressure.

Table 3.8-7—ISI Schedule for the U.S. EPR

	Test Pressure	
Year	**U.S. EPR ISI**	**RG 1.90**
0	$1.15*P_d$	$1.15*P_d$
1	P_N	P_N
3	$1.15*P_d$	$1.15*P_d$
7	$1.15*P_d$	$1.15*P_d$
Thereafter	P_a	P_a

Notes:

P_N – Normal operating pressure or zero.

P_d – Containment design pressure, Pd = 62 psig.

Pa – Maximum calculated DBA pressure, Pa = 55 psig.

Table 3.8-8—Materials for Structural Steel Shapes and Plates
Sheet 1 of 2

ASTM Designation	F_y	F_u
A36	36 ksi	58 to 80 ksi
A53 (Type E or S) (Gr. B)	35 ksi	60 ksi
A106		
Grade A	30 ksi	48 ksi
Grade B	35 ksi	60 ksi
Grade C	40 ksi	70 ksi
A167	27 to 39 ksi	73 to 94 ksi
A240		
Austenitic	25 to 70 ksi	70 to 125 ksi
Duplex	58 to 80 ksi	87 to 116 ksi
Ferritic or Martensitic	25 to 90 ksi	55 to 115 ksi
A242	42 to 50 ksi	63 to 70 ksi
A276		
Austenitic	25 to 125 ksi	70 to 145 ksi
Austenitic-ferritic	65 to 105 ksi	90 to 125 ksi
Ferritic	30 to 60 ksi	60 to 75 ksi
Martensitic	30 to 100 ksi	60 to 125 ksi
A312	25 to 62 ksi	70 to 115 ksi
A441	40 to 50 ksi	60 to 70 ksi
A479		
Austenitic	25 to 125 ksi	70 to 145 ksi
Austenitic-ferritic	65 to 85 ksi	90 to 118 ksi
Ferritic	25 to 55 ksi	60 to 70 ksi
Martensitic	40 to 100 ksi	70 to 130 ksi
A500 (round)		
Grade A	33 ksi	45 ksi
Grade B	42 ksi	58 ksi
Grade C	46 ksi	62 ksi
Grade D	36 ksi	58 ksi
A500 (square & rectangular)		
Grade A	39 ksi	45 ksi
Grade B	46 ksi	58 ksi
Grade C	50 ksi	62 ksi
Grade D	36 ksi	58 ksi
A501	36 ksi	58 ksi
A514	90 to 100 ksi	100 to 130 ksi
A515	32 to 38 ksi	60 to 90 ksi
A516	30 to 38 ksi	55 to 90 ksi

Table 3.8-8—Materials for Structural Steel Shapes and Plates
Sheet 2 of 2

ASTM Designation	F_y	F_u
A570	30 to 55 ksi	49 to 70 ksi
A572	42 to 65 ksi	60 to 80 ksi
A588	42 to 50 ksi	63 to 70 ksi
A607		
Class I	45 to 70 ksi	60 to 85 ksi
Class II	45 to 70 ksi	55 to 80 ksi
A618		
Grade Ia, Ib & II	46 to 50 ksi	67 to 70 ksi
Grade III	50 ksi	65 ksi
A709	36 to 50 ksi	58 to 80 ksi
A913	50 to 70 ksi	65 to 90 ksi
A992	50 to 65 ksi	65 ksi

Notes:

1. The design of structural steel members is based on the conservative use of the minimum allowable material stress values provided in Table 3.8-8. The design specifies a particular minimum value to be used for the fabrication of the component, and the stress values of the materials actually used in fabrication will be confirmed by certified material test reports and certificates of conformance.

Table 3.8-9—Structural Bolting Materials

ASTM Designation	Minimum F_y (ksi)	Minimum F_u (ksi)
A193		
Grade B5	80 ksi	100 ksi
Grade B6	85 ksi	110 ksi
Grade B6X	70 ksi	90 ksi
Grade B7	75 to 105 ksi	100 to 125 ksi
Grade B7M	75 to 80 ksi	100 ksi
Grade B16	85 to 105 ksi	100 to 125 ksi
A307		
Grade A	-	60 ksi
Grade B	-	60 to 100 ksi
Grade C	36 ksi	58 to 80 ksi
A320		
Class 1 & 1A	30 ksi	75 ksi
Class 2	50 to 100 ksi	90 to 125 ksi
Grade L7M	80 ksi	100 ksi
Other grades	105 ksi	125 ksi
A325	81 to 92 ksi	105 to 120 ksi
A354		
Grade BC	99 to 109 ksi	115 to 125 ksi
Grade BD	115 to 130 ksi	140 to 150 ksi
A449	58 to 92 ksi	90 to 120 ksi
A490	130 ksi	150 to 170 ksi
A540		
Grade B21	100 to 150 ksi	115 to 165 ksi
Grade B22	100 to 150 ksi	115 to 165 ksi
Grade B23	100 to 150 ksi	115 to 165 ksi
Grade B24	100 to 150 ksi	115 to 165 ksi
Grade B24V	130 to 150 ksi	145 to 165 ksi
A564		
Type 630	75 to 170 ksi	115 to 190 ksi
Type 631	140 to 150 ksi	170 to 185 ksi
Type 632	160 to 175 ksi	180 to 200 ksi
Type 634	155 ksi	170 ksi
Type 635	150 to 170 ksi	170 to 190 ksi
Type XM-12	75 to 170 ksi	115 to 190 ksi
Type XM-13	85 to 205 ksi	125 to 220 ksi
Type XM-16	185 to 220 ksi	205 to 235 ksi
Type S45503	185 to 220 ksi	205 to 235 ksi
Type XM-25	75 to 170 ksi	125 to 180 ksi

Table 3.8-10—Structural Bolting Nut and Washer Materials

Item	ASTM Designation
Nuts	A194 (all grades) A563 (all grades) F1852
Washers	F436

Note:

1. Use of A563 nuts conforms to Appendix X1 of ASTM A563, which provides guidance on the suitability of A563 nuts for specific bolting materials.

Table 3.8-11—Minimum Required Factors of Safety Against Overturning, Sliding, and Flotation for Foundations

Load Combination	Minimum Factors of Safety		
	Overturning	Sliding	Flotation
D + H + W	1.5	1.5	-
D + H + Wt	1.1	1.1	-
D + H + E’	1.1	1.1	-
D + Fb	-	-	1.1

Table 3.8-12—Deleted

Table 3.8-13—Static Spring Distribution

Soil Case	K_o (k/ft^3)		Recommended Springs and Distribution	
	Static	Dynamic	Distribution (b = l = ~52.4m)	Min/Max Spring
1n2ue	36	72	$K(x,y)=K_o\ [3.74 - 3.12*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.62 K_o, 1.99 K_o
4ue	372	743	$K(x,y)=K_o\ [3.12 - 2.42*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.70 K_o, 1.77 K_o
5ae	5402	10804	$K(x,y)=K_o\ [2.01 - 1.15*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.86 K_o, 1.37 K_o
2sn4ue	162	324	$K(x,y)=K_o\ [3.33 - 2.65*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.68 K_o, 1.85 K_o
1n5ae	918	1837	$K(x,y)=K_o\ [2.48 - 1.69*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.79 K_o, 1.53 K_o
hf-lb	1578	3157	$K(x,y)=K_o\ [2.53 - 1.75*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.78 K_o, 1.55 K_o
hf-be	2341	4682	$K(x,y)=K_o\ [2.44 - 1.64*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.80 K_o, 1.52 K_o
hf-ub	3634	7267	$K(x,y)=K_o\ [2.32 - 1.50*sqrt(1 - x^2/2l^2 - y^2/2b^2)]$	0.82 K_o, 1.48 K_o

Table 3.8-14—Deleted

Table 3.8-15—Deleted

Table 3.8-16—Deleted

Table 3.8-17—Design Report Cross-Reference Table
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SRP Section 3.8.4, Appendix C	U.S. EPR FSAR Sections			
	Concrete Containment (Section 3.8.1)	Steel Containment (Section 3.8.2) & Concrete and Steel Internal Structures (Section 3.8.3)	Other Seismic Category I Structures (Section 3.8.4)	Foundations (Section 3.8.5)
II. Structural Description and Geometry				
1. Structural Geometry and Dimensions	Section 3.8.1.1	Sections 3.8.2.1, 3.8.3.1	Section 3.8.4.1	Section 3.8.5.1
2. Key Structural Elements and Description	Sections 3.8.1.1	Sections 3.8.2.1, 3.8.3.1	Section 3.8.4.1	Section 3.8.5.1
3. Floor Layout and Elevations	Figures 3.8-1 - 3.8-13	Figures 3.8-3 - 3.8-13, 3.8-32 - 3.8-37	Figures 3.8-3 - 3.8-13 (Reactor Shield Building); Figures 3.8-38 - 3.8-85 (Fuel Building and Safeguard Buildings); and Figures 3.8-89 - 3.8-102 (Emergency Power Generating Buildings and Essential Service Water Buildings)	Figures 3.8-11, 3.8-12, 3.8-13, 3.8-50, 3.8-51, 3.8-52, 3.8-63, 3.8-74, 3.8-85, 3.8-93, 3.8-94, 3.8-101, 3.8-102, 3.8-103 and 3.8-118
4. Conditions of Vicinity and Supports	Sections 3.8.1.1	Sections 3.8.2.1, 3.8.3.1	Section 3.8.4.1	Section 3.8.5.1
5. Special Structural Features	Section 3.8.1.1.2-Grouted Tendon System	Sections 3.8.3.1.9-Core Melt Retention Area, IRWST	Sections 3.8.4.1.1-Reactor Shield Building, Section 3.8.4.1.2-Fuel Building dual walls, Section 3.8.4.1.3-Safeguard Buildings 2 and 3 dual walls	None

Table 3.8-17—Design Report Cross-Reference Table
Sheet 2 of 5

SRP Section 3.8.4, Appendix C		U.S. EPR FSAR Sections			
		Concrete Containment (Section 3.8.1)	Steel Containment (Section 3.8.2) & Concrete and Steel Internal Structures (Section 3.8.3)	Other Seismic Category I Structures (Section 3.8.4)	Foundations (Section 3.8.5)
III. Structural Material Requirements					
1. Concrete	A. Compressive Strength	Section 3.8.1.6.1, Table 3.8-2, Appendix 3E.1	Section 3.8.3.6.1, Appendix 3E.1	Section 3.8.4.6.1, Appendix 3E.1	Section 3.8.5.6.1, Appendices 3E.1
	B. Modulus of Elasticity	Table 3.8-2	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendix 3E.1
	C. Shear Modulus	Section 3.8.1.6.1	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendix 3E.1
	D. Poisson's Ratio	Table 3.8-2	Section 3.8.3.6.1	Section 3.8.4.6.1, same as 3.8.3.6.1	Section 3.8.5.6.1, same as 3.8.3.6.1
2. Reinforcement	A. Yield Stress	Table 3.8-2	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendix 3E.1
	B. Tensile Strength	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendix 3E.1
	C. Elongation	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendices 3E.1
3. Structural Steel	A. Grade	Section 3.8.1.6.4	Section 3.8.3.6.3, Table 3.8-8	Section 3.8.4.6.1, Table 3.8-8	Section 3.8.5.6.1, Table 3.8-8
	B. Ultimate Tensile Strength	Appendix 3E.1	Table 3.8-8	Table 3.8-8	Table 3.8-8
	C. Yield Stress	Appendix 3E.1	Table 3.8-8	Table 3.8-8	Table 3.8-8

Table 3.8-17—Design Report Cross-Reference Table
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SRP Section 3.8.4, Appendix C		U.S. EPR FSAR Sections: Concrete Containment (Section 3.8.1)	U.S. EPR FSAR Sections: Steel Containment (Section 3.8.2) & Concrete and Steel Internal Structures (Section 3.8.3)	U.S. EPR FSAR Sections: Other Seismic Category I Structures (Section 3.8.4)	U.S. EPR FSAR Sections: Foundations (Section 3.8.5)
4. Prestressing Stage	A. Type of System	Section 3.8.1.1.2	N/A	N/A	N/A
	B. Description of Tendons	Sections 3.8.1.1.2, 3.8.1.6.3	N/A	N/A	N/A
	C. Description of Surcharge	Table 3.8-5	N/A	N/A	N/A
	D. Tendons and Sheeting Layout	Section 3.8.1.1.2, 3.8.1.6.3, Figures 3.8-18 and 3.8-19	N/A	N/A	N/A
	E. Dome Prestressing	Section 3.8.1.1.2	N/A	N/A	N/A
5. Foundation Media	A. General Description	Section 2.5, 3.7.1.3, Table 3.7.2-9			
	B. Unit Weight	Section 2.5.4.2; Table 2.1-1; Table 3.7.2-9			
	C. Shear Modulus	Table 3.7.2-9			
	D. Angle of Internal Friction	Section 2.5.4.2, Table 2.1-1			
	E. Cohesion	Section 2.5.4.2			
	F. Bearing Demand	Section 2.5.4.10.1, Table 2.1-1			
6. Special Considerations		None	None	None	None

Table 3.8-17—Design Report Cross-Reference Table
Sheet 4 of 5

SRP Section 3.8.4, Appendix C	U.S. EPR FSAR Sections			
	Concrete Containment (Section 3.8.1)	Steel Containment (Section 3.8.2) & Concrete and Steel Internal Structures (Section 3.8.3)	Other Seismic Category I Structures (Section 3.8.4)	Foundations (Section 3.8.5)
IV. Structural Loads				
1. Live and Dead Load Floor Plans	Appendix 3E.1	Appendix 3E.1	Section 3.8.4.3.1, Appendices 3E.1, 3E.2 and 3E.3	Appendices 3E.1, 3E.2, 3E.3
2. Determination of Transient and Dynamic Loads	Section 3.8.1.3.1, Figures 3.8-20 - 3.8-23 (accident thermal and pressure)	Section 3.8.3.3.1, Figures 3.8-20 and 3.8-21 (accident thermal and pressure)	Section 3.8.4.3.1 (thermal), Table 3.5-1 (missiles)	Section 3.8.5.3, same as 3.8.4.3.1
3. Manufacturer's Data on Equipment Loads	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2 and 3E.3	Appendices 3E.1, 3E.2, 3E.3
4. Environmental Loads	Sections 3.3, 3.7.1, 3.8.1.3.1, Figure 3.7.1-1, Tables 2.1-1, 3.7.2-10, Table 3.7.2-20	Sections 3.8.2.3.1, 3.8.3.3.1, Figure 3.7.1-1, Tables 2.1-1, 3.7.2-11, 3.7.2-19	Sections 2.4, 3.3, 3.8.4.3.1, Tables 2.1-1, 3.7.2-12 - 3.7.2-17, 3.7.2-21 - 3.7.2-25, 3.7.2-27, 3.7.2-28, Figures 3.7.1-1, 3.7.1-33 and 3.7-34	Sections 2.4, 3.3, Figure 3.7.1-1, Tables 2.1-1, 3.7.2-18
5. Torsional Effects	Table 3.7.2-20	Table 3.7.2-19	Tables 3.7.2-21 - 3.7.2-25	Table 3.7.2-26
V. Structural Analysis and Design				
1. Design Computations of Critical Elements	Appendix 3E.1.1, 3E.1.2 and 3E.1.3	Appendix 3E.1.4, 3E.1.5 and 3E.1.6	Appendix 3E.1.7, 3E.1.8, 3E.2 and 3E.3	Appendix 3E.1.9
2. Stability Calculations	Appendices 3E.1, 3E.2 and 3E.3			
3. Engineering Drawings Including Details of Connections and Joints	Figures 3E.1-3 - 3E.1-25	Figures 3E.1-26 - 3E.1-57	Figures 3E.1-58 - 3E.1-70, 3E.2-3 - 3E.2-12, 3E.3-3 - 3E.3-9	Figures 3E.1-71 - 3E.1-76

Table 3.8-17—Design Report Cross-Reference Table
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SRP Section 3.8.4, Appendix C	U.S. EPR FSAR Sections			
	Concrete Containment (Section 3.8.1)	Steel Containment (Section 3.8.2) & Concrete and Steel Internal Structures (Section 3.8.3)	Other Seismic Category I Structures (Section 3.8.4)	Foundations (Section 3.8.5)
4. Discussion of Unique Features and Problem Resolution	None	None	None	None
VI. Summary of Results				
1. The Required Sections	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2, 3E.3	Appendix 3E.1
2. The Provided Sections	Appendix 3E.1.1, 3E.1.2 and 3E.1.3	Appendix 3E.1.4, 3E.1.5 and 3E.1.6	Appendix 3E.1.7, 3E.1.8, 3E.2 and 3E.3	Appendix 3E.1.9
3. Breakdown of Individual Load Combinations	Appendix 3E.1.1, 3E.1.2 and 3E.1.3	Appendix 3E.1.4, 3E.1.5 and 3E.1.6	Appendix 3E.1.7, 3E.1.8, 3E.2 and 3E.3	Appendix 3E.1.9
4. Tabulation of Capacities of the Section Versus Capacities Required for Different Failure Modes (Bending, Shear, Axial Load)	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2 and 3E.3	Appendix 3E.1
5. Margin of Safety Provided	Appendix 3E.1	Appendix 3E.1	Appendices 3E.1, 3E.2 and 3E.3	Appendix 3E.1

Table 3.8-18—Reactor Containment Building Doors
Sheet 1 of 7

RCB Elevation and Door No.	Vent Area (ft²)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
-8 ft Door 1	30.89	-8 ft Room 3	-8 ft Room 2	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
-8 ft Door 2	30.89	-8 ft Room 4	-8 ft Room 2	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
-8 ft Door 3	5.92	IRWST	-8 ft Room 3	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
-8 ft Door 4	5.92	-8 ft Room 7	-8 ft Room 2	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 5	5.92	IRWST	-8 ft Room 4	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
-8 ft Door 6	13.24	-8 ft Room 8	-8 ft Room 7	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
-8 ft Door 7	5.92	-8 ft Room 16	-8 ft Room 13	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 8	21.85	-8 ft Room 6	-8 ft Room 9	2.90 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
-8 ft Door 9	5.92	-8 ft Room 5	-8 ft Room 3	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
-8 ft Door 10	5.92	-8 ft Room 15	-8 ft Room 11	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 11	5.92	-8 ft Room 14	-8 ft Room 9	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 12	5.92	-8 ft Room 5	-8 ft Room 4	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
-8 ft Door 13	5.92	-8 ft Room 11	-8 ft Room 5	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 14	5.92	-8 ft Room 9	-8 ft Room 5	1.45 + 20%	Pressure Relief Blowout Panel	S	I
-8 ft Door 15	13.89	-8 ft Room 10	-8 ft Room 5	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+5 ft Door 1	5.92	+5 ft Room 13	+5 ft Room 18	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+5 ft Door 2	5.92	+5 ft Room 14	+5 ft Room 18	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

Table 3.8-18—Reactor Containment Building Doors
Sheet 2 of 7

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
+5 ft Door 4	28.63	+5 ft Room 16	+5 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+5 ft Door 5	20.77	+5 ft Room 17	+5 ft Room 16	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+5 ft Door 6	26.48	+5 ft Room 27	+5 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+5 ft Door 7	26.48	+5 ft Room 28	+5 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+5 ft Door 8	27.12	+5 ft Room 4	+5 ft Room 27	N/A[1]	Radiation Door	NS-AQ	II
+5 ft Door 9	27.12	+5 ft Room 7	+5 ft Room 28	N/A[1]	Radiation Door	NS-AQ	II
+5 ft Door 10	5.92	+5 ft Room 12	+5 ft Room 10	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+5 ft Door 12	20.88	+5 ft Room 19	+5 ft Room 12	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+5 ft Door 13	19.91	+5 ft Room 21	+5 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+5 ft Door 14	13.89	+5 ft Room 16	+5 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+5 ft Door 20	5.92	+5 ft Room 24	+5 ft Room 27	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+5 ft Door 21	5.92	+5 ft Room 23	+5 ft Room 27	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+5 ft Door 22	5.92	+5 ft Room 25	+5 ft Room 28	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+5 ft Door 23	5.92	+5 ft Room 26	+5 ft Room 28	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+17 ft Door 1	5.92	+17 ft Room 14	+17 ft Room 11	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

Table 3.8-18—Reactor Containment Building Doors
Sheet 3 of 7

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
+17 ft Door 2	22.71	+17 ft Room 6	+17 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 3	22.93	+17 ft Room 5	+17 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 4	22.82	+17 ft Room 23	+17 ft Room 18	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 5	5.92	+17 ft Room 26	+17 ft Room 15	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+17 ft Door 6	5.92	+17 ft Room 25	+17 ft Room 14	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+17 ft Door 7	22.06	+17 ft Room 2	+17 ft Room 24	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 8	26.48	+17 ft Room 4	+17 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 9	26.48	+17 ft Room 7	+17 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 10	26.48	+17 ft Room 3	+17 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 11	26.48	+17 ft Room 8	+17 ft Room 16	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 12	5.92	+17 ft Room 13	+17 ft Room 10	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+17 ft Door 13	23.35	+17 ft Room 9	+17 ft Room 16	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 14	22.06	+17 ft Room 24	+17 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+17 ft Door 15	5.92	+17 ft Room 20	+17 ft Room 13	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

**Table 3.8-18—Reactor Containment Building Doors
Sheet 4 of 7**

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
+17 ft Door *1 (RV Cavity Entrance)	N/A	+17 ft Room 23	+17 ft Room 1	None[2]	Water Tight[3]	NS-AQ	II
+17 ft Door *2 (Fuel Transfer Canal Entrance)	N/A	+17 ft Room 21	+17 ft Room 20	None[2]	Water Tight[3]	NS-AQ	II
+29 ft Door 1	5.92	+29 ft Room 15	+29 ft Room 12	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+29 ft Door 2	21.64	+29 ft Room 29	+29 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+29 ft Door 3	26.26	+29 ft Room 7	+29 ft Room 16	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 4	26.26	+29 ft Room 6	+29 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 5	26.26	+29 ft Room 8	+29 ft Room 16	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 6	30.57	+29 ft Room 5	+29 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 7	24.00	+29 ft Room 10	+29 ft Room 17	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 8	5.92	+29 ft Room 14	+29 ft Room 11	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+29 ft Door 9	26.48	+29 ft Room 20	+29 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 12	26.26	+29 ft Room 3	+29 ft Room 20	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+29 ft Door 13	5.92	+29 ft Room 18	+29 ft Room 19	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

Table 3.8-18—Reactor Containment Building Doors
Sheet 5 of 7

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4]		Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
		To Room	From Room				
+45 ft Door 1	5.92	+45 ft Room 14	+45 ft Room 10	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 2	21.85	+45 ft Room 18	+45 ft Room 22	2.90 + 20%	Pressure Relief Radiation Swing Door	S	I
+45 ft Door 3	24.22	+45 ft Room 5	+45 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+45 ft Door 4	24.22	+45 ft Room 4	+45 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+45 ft Door 5	21.85	+45 ft Room 21	+45 ft Room 22	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+45 ft Door 6	5.92	+45 ft Room 23	+45 ft Room 13	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 7	5.92	+45 ft Room 24	+45 ft Room 14	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 8	11.84	+45 ft Room 17	+45 ft Room 15	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 9	11.84	+45 ft Room 16	+45 ft Room 12	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 10	5.92	+45 ft Room 17	+45 ft Room 15	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 11	5.92	+45 ft Room 16	+45 ft Room 12	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 12	19.91	+45 ft Room 20	+45 ft Room 15	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+45 ft Door 13	5.92	+45 ft Room 12	+45 ft Room 9	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+45 ft Door 14	21.85	+45 ft Room 19	+45 ft Room 12	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+45 ft Door 15	5.92	+45 ft Room 13	+45 ft Room 22	1.45 + 20%	Pressure Relief Blowout Panel	S	I
+64 ft Door 1	5.92	+64 ft Room 10	+64 ft Room 7	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

Table 3.8-18—Reactor Containment Building Doors
Sheet 6 of 7

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
+64 ft Door 2	21.64	+64 ft Room 11	+64 ft Room 14	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+64 ft Door 3	19.96	+64 ft Room 3	+64 ft Room 13	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+64 ft Door 4	21.64	+64 ft Room 4	+64 ft Room 11	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+64 ft Door 5	5.92	+64 ft Room 15	+64 ft Room 11	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+64 ft Door 6	5.92	+64 ft Room 10	+64 ft Room 16	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+64 ft Door 7	5.92	+64 ft Room 9	+64 ft Room 16	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+79 ft Door 1	5.92	+79 ft Room 7	+79 ft Room 7	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+79 ft Door 2	5.92	+79 ft Room 7	+79 ft Room 20	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+79 ft Door 3	48.44	+79 ft Room 11	+79 ft Room 9	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+79 ft Door 4	24.22	+79 ft Room 20	+79 ft Room 12	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+79 ft Door 5	21.85	+79 ft Room 3	+79 ft Room 11	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+79 ft Door 6	21.85	+79 ft Room 4	+79 ft Room 10	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+79 ft Door 9	24.33	+79 ft Room 10	+79 ft Room 20	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+79 ft Door 10	5.92	+79 ft Room 19	+79 ft Room 10	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II
+79 ft Door 13	5.92	+79 ft Room 18	+79 ft Room 9	1.45 + 20%	Pressure Relief Blowout Panel	NS-AQ	II

Table 3.8-18—Reactor Containment Building Doors
Sheet 7 of 7

RCB Elevation and Door No.	Vent Area (ft^2)	Opening Direction[4] To Room	From Room	Burst Pressure (psid)	Door Function	Safety Class	Seismic Category
+94 ft Door 11	19.91	+94 ft Room 3	+94 ft Room 1	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II
+94 ft Door 12	19.91	+94 ft Room 4	+94 ft Room 11	2.90 + 20%	Pressure Relief Radiation Swing Door	NS-AQ	II

Notes:

1. This door has no pressure relief function and is a motor operated rolling door without hinges.
2. This door is part of the pool liner and has no pressure relief function.
3. The water tight function only applies when the refueling cavity is flooded for refueling activities.
4. The observer stands in the "From" room and the door swings open towards the observer. The observer is looking at the hinges and enters the "To" room.

Table 3.8-19—Static Foundation Modulus Values for EPGB Soil Cases

Soil Case	K_o (k/ft^3)	Springs and Distribution Distribution (b=l=88 ft)	Min/Max Springs
4u	894	$K(x,y) = K_o\left[5.69 - 4.94\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.75 K_o, 2.29 K_o
5a	12175	$K(x,y) = K_o\left[4.93 - 4.14\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.79 K_o, 2.08 K_o
1n5a	3044	$K(x,y) = K_o\left[5.31 - 4.54\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.77 K_o, 2.18 K_o
1n2u	89	$K(x,y) = K_o\left[5.27 - 4.49\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.78 K_o, 2.18 K_o
2sn4u	359	$K(x,y) = K_o\left[4.58 - 3.77\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.81 K_o, 1.98 K_o
hf_c[2]	149	$K(x,y) = K_o\left[4.83 - 4.03\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.8 K_o, 2.05 K_o
hf_s[1,2]	370	$K(x,y) = K_o\left[2.60 - 1.69\sqrt{1 - \frac{x^2}{2l^2} - \frac{y^2}{2b^2}}\right]$	0.91 K_o, 1.44 K_o

Notes:

1. Soil profile does not consider concrete fill on top of the rock for static foundation modulus.

2. hf_c is a high frequency profile with concrete and
   hf_s is a high frequency profile with soil

Next File