3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

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3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

This chapter of the Final Safety Analysis Report (FSAR) discusses the principal architectural and engineering design of those structures, systems, components, (SSCs) and equipment that are important to safety. It also provides information regarding the design, fabrication, erection, and testing to quality standards commensurate with the importance of their safety functions to be performed during the life of the plant. Recognized industry codes and standards are applied per the safety classifications to ensure that they meet the required safety-related function.

3.7 Seismic Design

Safety-related SSCs are designed to withstand safe-shutdown earthquake (SSE) loads and other dynamic loads, including those due to reactor building vibration caused by suppression pool dynamics. This section addresses seismic aspects of the design and analysis in accordance with Regulatory Guide (RG) 1.70, Revision 3, “Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition).”

3.7.1 Seismic Design Parameters

3.7.1.1 *Introduction*

This FSAR section addresses the design earthquake ground motion used for seismic analysis and design of the Category I structures. The design earthquake ground motion is based on the seismic and geologic characteristics at the site and established in terms of a set of idealized and smooth curves called the design response spectra. Advanced Boiling-Water Reactor (ABWR) Design Control Document (DCD) Figures 3.7-1 and 3.7-2 show the standard ABWR design values of the horizontal and vertical SSE spectra, respectively, applied to the finished grade in the free field for damping values of 2, 3, 4, 5, and 7 percent of critical damping. The maximum ground acceleration for both the horizontal and vertical motions is 0.30 g at 33 Hertz (Hz) for the standard plant structures. For site-specific structures, the maximum ground acceleration for both the horizontal and vertical motions is 0.13 g.

The bases for the seismic design of safety-related SSCs and equipment include the following:

- Design ground motion response spectra (GMRS)
- Design ground motion time history
- Percentage of the critical damping values
- Supporting media for seismic Category I structures

3.7.1.2 *Summary of Application*

Section 3.7.1 of the STP Units 3 and 4 FSAR, Revision 9, incorporates by reference Section 3.7.1 of the certified ABWR DCD, Revision 4, referenced in Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52, Appendix A. In addition, in FSAR Section 3.7.1, the applicant provides the following:

Tier 1 Departures

- STP DEP T1 5.0-1 — Site Parameters (shear wave velocity departure)

This departure states that the shear wave velocity at the STP site for Units 3 and 4 varies both horizontally within a soil stratum and vertically with depth and does not meet the minimum shear wave velocity requirements of 305 m/s (1,000 ft/s) of the ABWR DCD Tier 1, Table 5.0. The applicant performed a site-specific soil-structure interaction (SSI) analysis, as discussed in Appendix 3A of the FSAR, to confirm that the standard plant results included in the ABWR DCD will envelop the results of the site-specific SSI analysis. This analysis used measured shear wave velocities with appropriate variations to represent the variability at the site.

- STD DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category I to Non-Seismic

The referenced ABWR DCD Tier 1, Section 2.15.13 states that the below grade exterior walls of the radwaste building (RWB) and the basemat are classified as seismic Category I. This departure revises the seismic category of the RWB substructure from Category I to non-seismic. The applicant also references NUREG–1503, Section 3.8.4, which states that the RWB is not a seismic Category I structure, because the RWB does not house any safety-related systems or components. The detailed guidance for the design of the radwaste processing SSCs is in RG 1.143, Revision 2, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants." In this departure, the applicant commits to follow the guidance of RG 1.143, Revision 2.

COL License Information Item

- COL License Information Item 3.19 — Seismic Design Parameters

The applicant states that Section 2.5S.2 of the COL FSAR provides a site-specific assessment against the Tier 1 site requirements. The site-specific SSI analysis for the reactor building (RB) and the control building (CB) is in Appendix 3A of the FSAR.

Supplemental Information

The applicant adds a new section (Section 3H.6), "Site-Specific Seismic Category I Structure," which provides supplemental information on the design of the site-specific seismic Category I structures.

Design Response Spectra

The applicant developed site-specific horizontal and vertical SSE GMRS for the site, as discussed in FSAR Section 2.5S.2. A comparison of the GMRS with the ABWR DCD SSE response spectra is shown in FSAR Figure 3A-231 for the horizontal direction and in Figure 3A-232 for the vertical direction. In addition to the GMRS, the applicant developed the foundation input response spectra (FIRS) at the free-field foundation elevation of the seismic Category I structures using the same probabilistic models and analyses, which were used to develop the GMRS. Detailed descriptions of the procedures used for calculating the GMRS and the FIRS are in FSAR Subsections 2.5S.2.5 and 2.5S.2.6, respectively. The FIRS for Category I structures are included in FSAR Appendices 3A and 3H.

Site-specific SSE design response spectra for damping values of 2, 3, 4, 5, and 7 percent used for the site-specific SSI analysis of the RB and CB, as well as for the site-specific seismic analysis and for the design of site-specific structures (e.g., the ultimate heat sink/reactor service water (UHS/RSW) pump house, RSW piping tunnel, and other seismic Category I structures) are shown in Figures 3.7-1a, 3.7-1b, 3.7-2a, and 3.7-2b in FSAR Section 3.7. The development of the site-specific SSE spectra shown in FSAR Figures 3.7-1a and 3.7-2a is discussed in Subsection 3H.6.5.1.1 of Appendix 3H. These SSE spectra are compared with the ABWR DCD SSE design response spectra in FSAR Figures 3A-231a and 3A-232a for the horizontal and vertical directions, respectively.

Design Time Histories

The applicant developed synthetic acceleration time histories that are consistent with the input spectra defined and discussed in FSAR Section 3A.16.1, which uses the 1952 Taft earthquake time history as the seed motion. The synthetic time histories are used as input to the SSI analysis. A single set of time histories (two horizontal and one vertical) was developed to satisfy the enveloping requirements of Option 1, Approach 2 in Revision 3 of SRP Section 3.7.1, SRP Acceptance Criterion 1.B. The acceleration, velocity, and displacement time histories of the synthetic ground motions described above in the two horizontal and vertical directions are shown in FSAR Figures 3A-251 through 3A-259. Comparisons of the response spectra of the synthetic time histories, the input spectrum, and 1.3 times the input spectrum in the two horizontal and vertical directions for the 5 percent damping are shown in FSAR Figures 3H.6-12 through 3H.6-14.

The operating basis earthquake (OBE) is not a design-basis earthquake for the ABWR plant. The plant shutdown criteria described in ABWR DCD Subsection 3.7.4.4 will be based on the site-specific SSE response spectra.

Percentage of Critical Damping Values

The applicant states that the percentages of critical damping values considered in the seismic reconciliation analysis of the RB and CB structures are in accordance with the guidance in RG 1.61, Revision 1, “Damping Values for Seismic Design of Nuclear Power Plants,” as discussed in FSAR Section 3A.16.3. The strain-compatible soil damping values considered in the analysis are shown in FSAR Table 3H.6-1 for various soil layers.

The percentages of critical damping values used for the seismic analysis and design of the site-specific seismic Category I structures are discussed in FSAR Subsection 3H.6.5.1.2. The strain-compatible, soil-damping values considered in the above analyses are discussed in Subsection 3H.6.5.2.4.

Supporting Media for Seismic Category I Structures

Soil conditions at the STP site for Units 3 and 4 are described in FSAR Section 2.5S.4. The soil at the site extends down several thousand feet and consists of alternating layers of clay, silt, and sand. Soil layering characteristics, shear wave velocities, unit weight, and Poisson’s ratio are in FSAR Table 2.5S.4-27. The ground water elevation is approximately 2.4 m (8 ft) below grade.

The overall building dimensions and embedment depth for the RB and CB structures are described in Section 3A.17. The UHS basin, UHS cooling towers, RSW pump house, and RSW piping tunnel are described in Subsection 3H.6.5.1.3. FSAR Section 3H.6.7 describes the diesel generator fuel oil storage vaults (DGFOSVs), and FSAR Section 3H.7.3 describes the diesel generator fuel oil tunnels (DGFOTs).

Other Analyses

The liquefaction evaluation is described in FSAR Section 2.5S.4.8. This evaluation uses the measured shear wave velocities at the site and concludes that the site is acceptable from a liquefaction potential point of view.

FSAR Section 2.5S.4 evaluates the bearing capacity and concludes that it meets the Tier 1 requirement in Table 5.0-1 for the allowable bearing capacity of 718 kPa (15 ksf).

3.7.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503 "Final Safety Evaluation Report Related to the Certification of the Advanced Boiling-Water Reactor Design." In addition, the relevant requirements of the Commission regulations for the seismic design parameters, and the associated acceptance criteria, are in Section 3.7.1 of NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, (LWR Edition)," the Standard Review Plan (SRP).

In accordance with Section VIII, "Processes and Changes and Departures," of, "Appendix A to Part 52 -- Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies Tier 1 departures. Tier 1 departures require prior NRC approval and are subject to the requirements in 10 CFR Part 52, Appendix A, Section VIII.A.4.

The regulatory guidance for the seismic design of safety-related SSCs and equipment includes the following:

- RG 1.60, Revision 1, "Design Response Spectra for Seismic Design of Nuclear Power Plants"
- RG 1.61, Revision 1, "Damping Values for Seismic Design of Nuclear Power Plants"
- RG 1.92, Revision 2, "Combining Modal Responses and Spatial Components in Seismic Response Analysis"
- RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion"

3.7.1.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.7.1 of the certified ABWR DCD. The staff reviewed Section 3.7.1 of the STP Units 3 and 4 combined license (COL) FSAR, and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

Tier 1 Departures

- STP DEP T1 5.0-1 Site Parameters (shear wave velocity departure)

The staff reviewed the information on Tier 1 Departure STP DEP T1 5.0-1 involving the site's shear wave velocity parameter in Part 7 of the COL application. This departure from the ABWR DCD requirement affects the seismic analysis.

ABWR DCD Tier 1, Table 5.0 specifies that the minimum shear wave velocity is 305 m/s (1,000 ft/s). As documented in STP Units 3 and 4 COL FSAR Subsections 2.5S.4.4 and 2.5S.4.7, the shear wave velocity at the STP Units 3 and 4 site is lower than the 305 m/s (1,000 ft/s) that is the minimum shear wave velocity specified in the ABWR DCD. Because the site profile is outside the range of soil profiles considered in the ABWR DCD, the applicant performed the site-specific SSI analyses consistent with the guidance in SRP Sections 3.7.1 and 3.7.2 to verify that the results of the site-specific SSI analysis are bounded by the standard plant results included in the ABWR DCD. The applicant also updated the FSAR to incorporate the results of the site-specific SSI analysis.

The computer program SASSI2000 (see FSAR Appendix 3C) was used for the seismic SSI analysis of the RB and CB. The methodology used in SSI analysis is referred to as the flexible volume method and involves partitioning the total SSI system into two substructures, namely the "Structure" and "Foundation." The "Structure" consists of the structure minus the excavated soil, and the "Foundation" consists of the original site (i.e., a layered soil system). Following the general sub-structuring approach, the foundation is analyzed first to establish the foundation impedance and scattering properties. These properties are then used as boundary conditions to analyze the structure. SASSI2000 uses the finite element technique to model the structure and the continuum solution to solve the foundation impedance and site-response problem. The dynamic equation of motion is solved using the complex frequency response method. The Fast Fourier Transform algorithm is used to calculate the time history of responses from the computed transfer functions convolved with the time history of the input motion. The applicant used the direct method (DM) to calculate the impedance matrix for the flexible volume method in SASSI2000.

The RB and CB models are three-dimensional lumped mass-beam models that consider shear, bending, and axial deformations. This lumped mass beam model is coupled with the layered soil model. The model details are described in FSAR Section 3A.19. Structural responses in terms of the accelerations, forces, and moments are computed. Floor response spectra (FRS) are obtained from the calculated response acceleration time histories. The SSI analyses for the three directional earthquake components are performed separately. The maximum co-directional responses for each of the three earthquake components are combined as described in Section 3A.16.2.

The applicant compared the results of the site-specific SSI analyses of the RB and CB with the ABWR DCD envelope results in terms of seismic forces, maximum accelerations and broadened acceleration response spectra. To ensure that the results from both horizontal components are covered, the response spectra in two horizontal directions were enveloped and compared with the ABWR DCD design horizontal response spectra.

FSAR Tables 3A-29 and 3A-30 compare the RB seismic forces and maximum accelerations, respectively, which are enveloped by the ABWR DCD results. FSAR Figures 3A-267 through 3A-292 compare the broadened acceleration response spectra at typical locations in the RB structure. As these figures show, the ABWR DCD results envelop the site-specific RB results for all frequencies above 0.2 Hz.

For the CB, FSAR Tables 3A-31 and 3A-32 compare the seismic forces and maximum accelerations, respectively, which are enveloped by the ABWR DCD results. FSAR Figures 3A-293 through 3A-298 compare the broadened acceleration response spectra at typical locations in the CB structure. As these figures show, the ABWR DCD results envelop the site-specific CB results for all frequencies above 0.2 Hz.

The above comparisons of the applicant's SSI analyses and the results show that the ABWR DCD seismic forces, accelerations, and response spectra bound the results of the site-specific RB and CB SSI analyses.

To evaluate the adequacy of the SSI analysis the staff issued **Requests for Additional Information (RAIs) 03.07.01-4**, **03.07.01-17, 03.07.01-25,** and **03.07.01-23** requesting the applicant to describe the SSI model for the RB and CB. The RAIs asked the applicant to include the model of the subgrade and to explain how the model considered the effects of embedment, the ground water table, backfill properties, and the structural concrete fill below the basemat in the SSI analysis. The staff also asked the applicant to provide for each applicable seismic Category I structure covered by the ABWR DCD a comparison of the results (such as seismic displacements, accelerations, FRS at the mat foundation, the top of the building including some peripheral locations of the building, major equipment locations, and polar crane support) of site-specific SSI analysis using the site-specific design for the SSE ground motion as input with the results documented in the ABWR DCD.

In responses to the above RAIs, the applicant provided additional details in Appendices 3A and 3H of the COL FSAR to facilitate the review of the SSI analyses and the results. The staff reviewed this information and evaluated several issues, as discussed below:

Backfill Properties

The staff reviewed the specification of the strain-compatible backfill properties (shear modulus and damping) used in the SSI analysis. The development of strain-compatible backfill properties conformed to the ABWR DCD procedure. The applicant also provided site-specific inspections, tests, analyses, and acceptance criteria (ITAAC) on the verification of backfill properties used in the site-specific analysis for Category I structures in Table 3.0-11 of Part 9 of the COL application. On the basis of the above information, the staff found the backfill properties used in the site-specific analysis acceptable.

In the RAI responses, the applicant shows that the range of strain-compatible backfill shear wave velocities characterized by the lower-bound (LB), mean, and upper-bound (UB) values is enveloped by either the in situ soils or the standard soil profiles used in the ABWR DCD, as shown in FSAR Figure 3A-230a. In addition, the strain-compatible damping values for the LB, mean, and UB backfill materials are higher than those corresponding to the in situ materials, as shown in FSAR Figure 3A-230b. On the basis of the above information in Item 1 of the response to RAI 03.07.01-17 dated February 4, 2010 (ML100480204), the applicant concludes that the strain-compatible shear modulus and damping properties of the backfill material are bounded by either the LB and UB in situ soil properties or the ABWR DCD soil properties in the case of an UB shear modulus. Therefore, the applicant finds that no additional SSI analysis using backfill material properties is necessary. The staff agreed with this conclusion except that a review of the results in Figure 3A-230b shows the LB damping profile for the backfill to be significantly higher than that of the in situ soils. Because the SSE design motion is specified at the free-field ground surface, the staff was concerned that a higher damping value in the backfill material may result in greater motion at the foundation level compared with that obtained from the in situ soil column, with less damping to compensate for the higher attenuation of the motion in the backfill soils. The staff therefore issued RAI 03.07.01-25 requesting the applicant to provide further evidence that the higher damping value in the backfill material for the LB case will not result in foundation motions that exceed those of the ABWR DCD.

In response to RAI 03.07.01-25, the applicant performed additional site response analyses using backfill LB soil with site-specific SSE input motions and compared the results with those obtained using the ABWR DCD UB strain-compatible soil profile with 0.3g RG 1.60[1] spectra. The figures provided in the response to RAI 03.07.01-25 dated September 15, 2010 (ML102630145), show that the ABWR DCD foundation spectra for the UB soil envelop the corresponding spectra calculated using the backfill LB soil with the increased damping and site-specific SSE motion at all frequencies. On the basis of the above results, the staff concluded that the use of higher damping in the backfill material does not result in foundation motions that exceed those of the ABWR DCD.

Finite Element Mesh Refinement

In the response to RAI 03.07.01-4 dated September 3, 2009 (ML092510038), the applicant provides in the FSAR the SSI models used for the SSI analysis of the RB and CB. The SSI finite element models of the RB and CB conform to those of the ABWR DCD and include the embedment effects. In reviewing these models, the staff found that the horizontal mesh refinement in the foundation soil model is not capable of transmitting frequencies up to 33 Hz (passing frequency), which is considered to be the cut-off frequency for dynamic analysis of seismic loads. In the Revision 1 and Revision 1 of Supplement 1 responses to RAI 03.07.01-25 dated March 22, 2011 (ML110730069), the applicant performs a sensitivity study using the CB model and refines the finite element mesh size, so that the model is capable of passing frequencies up to 33 Hz in both the vertical and horizontal directions. On the basis of this sensitivity study, the staff concluded that the increases in the responses in the sensitivity study are not significant to the conclusion that the ABWR DCD responses envelop the site-specific responses for the RB and CB. The details of this sensitivity study are discussed in “Poisson’s Ratio Effects” below in this safety evaluation report (SER).

Poisson’s Ratio Effects

One of the issues in the SSI analysis was how the applicant treated the effects of a high ground water table (eight feet below the ground surface) at the site in the SSI analysis. In the response to this question in RAI 03.07.01-4 (ML092510038), RAI 03.07.01-17 (ML100480204), and RAI 03.07.01-25 (ML110730069), the applicant states that the Poisson’s ratio was capped at 0.48 for saturated soils in calculating the compression wave velocity. The use of this

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[1] This convention (i.e., 0.3g RG 1.60 spectrum) is used in this section to refer to the RG 1.60 spectrum, anchored to the peak ground acceleration of 0.3g.

Poisson ratio results in calculated compression wave velocities lower than 1,524 m/s (5,000 ft/s) in saturated soils when the shear wave velocities drop below approximately 299 m/s (980 ft/s). For example, as shown in FSAR Tables 3H.6-1a through 3H.6-1c, approximately 23, 17, and 73 m (75, 57, and 240 ft) of the respective soil column of the in situ mean, UB, and LB soil cases have calculated P-wave velocities lower than 1,524 m/s (5000 ft/s.) The use of compression wave velocities lower than 1,524 m/s (5,000 ft/s) in saturated soils will not allow the higher frequency components of the vertical motion to be transmitted into the structure and may result in a less conservative response. As such, the applicant was requested to assess the impact of using P-wave velocities lower than 1,524 m/s (5000 ft/s), in saturated soils on the response of the RB and CB, including in-structure response spectra (ISRS) by performing a sensitivity study and comparing the results for two cases: Case 1 would cap Poisson's ratio at 0.48 for saturated soils and let the P-wave velocity drop below 1,524 m/s (5,000 ft/s) (similar to the applicant's stated procedure); and Case 2 would set P-wave velocity to 1,524 m/s (5,000 ft/s) in saturated soils and allow Poisson's ratio to rise above 0.48 depending on the strain-compatible shear wave velocities.

The applicant submitted the results of a Poisson's ratio sensitivity study to assess the impact of a high ground water table at the site on the SSI analysis in the revised response to RAI 03.07.01-25 (ML110730069). For this sensitivity study, the applicant used the CB model described in FSAR Section 3A.19 and depicted in Figures 3A-265 and 3A-266, with the exception that the 0.61-m (2-ft) thick concrete mudmat under the base slab was also added to the SSI model and the SSI model mesh size was further refined, such that the model is capable of passing frequencies up to 33 Hz in the vertical and horizontal directions. The applicant performed a sensitivity analysis for the LB and UB soil cases. The analytical results using Poisson's ratio limit of 0.495 for saturated soil layers with refined mesh (modified model) are compared with those corresponding to the original SSI analyses that capped Poisson's ratio at 0.48 with coarse mesh (original model). The applicant compared the results from the new and the original SSI analyses including transfer functions, seismic forces and moments, maximum nodal accelerations, and ISRS at several key locations in the structure. The staff's reviews of these results are described below.

The applicant's Revision 1 response to RAI 03.07.01-25 shows some of the transfer function comparisons for the LB and UB soil cases. The results show an increase in the number and amplitude of the peaks in the transfer function in both the horizontal and vertical directions, at frequencies above 10 Hz when Poisson's ratio limit is increased from 0.48 to 0.495. To assess the impact of the above transfer function exceedance on the final response quantities, the results of the modified model were compared with those corresponding to the original model and with the ABWR DCD design envelope.

In a comparison with the results of the original model, the staff found that in terms of the maximum vertical accelerations, the results of the modified model exceeded those of the original model by up to 53.7 percent for the LB soil case and by up to 14.4 percent for the UB soil case. In terms of the maximum horizontal accelerations, the difference between the two models was insignificant (up to about 1.7 percent for the LB and up to about 1.1 percent for the UB soil case). Similarly, in terms of the ISRS, the results show a significant increase in the vertical spectra for the LB soil case when the Poisson's ratio limit is increased to 0.495; the increase for the UB soil case is insignificant. Because of the above increase in the response, the staff concluded that the effect of a higher Poisson's ratio cut-off is significant and should be properly considered in the SSI analysis. As a result, the applicant used a Poisson's ratio cut-off of 0.495 for all new SSI analyses of the site-specific structures. This effect of a high Poisson's ratio cut-off for site-specific seismic Category I structures such as the UHS/RSW pump house is further evaluated by the staff in "Ground Water Effects" in Subsection 3.7.2.4.3 "Procedures Used for Analytical Modeling," in this SER.

The Revision 1 and Revision 1 of Supplement 1 responses to RAI 03.07.01-25 (ML110730067 and ML110730069) compare the broadened enveloped horizontal and vertical spectra from the modified model (i.e., with Poisson's ratio capped at 0.495 and with the refined SSI mesh) and the corresponding ABWR DCD design spectra at typical locations in the CB structure. These responses show that the ABWR DCD design spectra significantly envelop those of the modified model. With this result, the staff concluded that the increases in the responses shown in the sensitivity study and discussed above are not significant to the conclusion that the ABWR DCD responses envelop the site-specific responses for the RB and CB by a significant margin.

To gain additional confidence that the results of the SSI analysis are not affected by numerical instability as the Poisson's ratio approached 0.5, the staff performed a confirmatory analysis using the UHS/RSW pump house model provided by the applicant. The staff was concerned that the use of the higher Poisson's ratio limit of 0.495 for the site-specific seismic Category I structures could result in structural responses that may exceed those of the design. On the basis of the analysis using the UHS/RSW pump house model, the staff concluded that the use of a high Poisson's ratio in the STP SSI analysis does not adversely impact the results. The results of the staff's confirmatory analysis on the Poisson's ratio are discussed below in Subsection 3.7.2.4.18, "Poisson's Ratio Confirmatory Analysis," in this SER.

Concrete Fill

With respect to the effect of the concrete fill layer below the basemat in the response to RAI 03.07.01-17 (ML100480204), the applicant states that the SSI analysis of the RB and CB does not include the concrete fill (i.e., the basemat was modeled as directly supported by the in situ soils). The applicant adds that because of the large margin between the site-specific ISRS and forces obtained from the analysis, and ABWR DCD spectra and forces as presented in Appendix 3A, modeling the in situ soil versus the concrete fill will not have a significant effect on the conclusion of the analysis. That is to say, the site-specific responses are bounded by the ABWR DCD responses. Because of the large margin (see FSAR Tables 3A-29 through 3A-32) between the DCD response and the response from the site-specific analysis (e.g., the ratio of the DCD maximum shear/STP Units 3 and 4 maximum shear at the RB wall = 3.59), the staff found this explanation for not including a 3-m (10-ft) thick foundation soil replacement by concrete in the SSI model acceptable.

Comparison of SSI Results

The staff reviewed the results of the RB and CB SSI analysis in the applicant's response to **RAI 03.07.01-2** (ML092660655). The results in terms of seismic forces and accelerations as well as the broadened acceleration response spectra at typical locations in the RB and CB are in the FSAR Sections 3A.20.1 and 3A.20.2, and were compared with those of the ABWR DCD. On the basis of these comparisons, the staff found that the ABWR DCD results envelop those of the site-specific SSI analyses of the RB and CB by a significant margin.

Further evaluations of the potential impact of the shear wave velocity departure at the STP Units 3 and 4 site due to structure-soil-structure interaction (SSSI) effects are discussed below.

SSSI Effect due to RB+CB+TB

To assess the impact of the shear wave velocity departure at the STP Units 3 and 4 site on seismic soil pressures on the RB and CB, the staff issued **RAIs 03.07.01-5**, **03.07.01-18,** and **03.07.01-26** requesting the applicant to provide the soil pressure profile in between the RB and CB and to discuss how the certified design addresses and bounds the potential effects of an increase in the seismic soil pressures in between the RB and CB due to SSSI effects.

In the responses to **RAIs 03.07.01-5 (ML092370556)**, **03.07.01-18 (ML100480204),** and **03.07.01-26 (ML102630145)**, the applicant performs two-dimensional SSSI analyses of the RB, CB, and turbine building (TB) using the SASSI2000 software to address the impact of the shear wave velocity departure on the seismic soil pressures between the RB and CB. Because the RB, CB, and non-seismic Category I TB are closely spaced in the north-south (N-S) direction, the applicant performed the SSSI analysis in the N-S direction. The SSSI model used above is similar to the model described in ABWR DCD Tier 2, Section 3A.9.7, for considering the SSSI effects on the RB and CB and the soil pressures on the walls of the buildings. To account for variations in soil properties, the applicant performed the SSSI analysis on all three site-specific in situ soil profiles (the LB, mean, and UB) with strain-compatible properties and enveloped the results. The SSI models of RB+CB and RB+CB+TB are shown in FSAR Figures 3A-299 and 3A-300, respectively. Details of the analysis are in FSAR Section 3A.21.

FSAR Figures 3A-301 and 3A-302 show the calculated dynamic soil pressure profiles in between the RB and CB for the RB and CB wall, respectively, together with the corresponding seismic design pressures from the ABWR DCD. After reviewing the soil pressure profiles in Figures 3A-301 and 3A-302, the staff concluded that the increase in the dynamic soil pressures in between the RB and CB due to SSSI effects on the RB, CB, and TB is bounded by the ABWR DCD design pressures and is therefore acceptable.

SSSI Effects due to Crane Wall

FSAR Subsection 2.5S.4.5.2.4 states that a reinforced concrete retaining wall will be constructed on the east side of the RB, CB, and TB to facilitate the excavation activities and to accommodate the reach of the heavy lift crane. The walls will vary in their exposed height to a maximum of 27.4 m (90 ft). The area on the west side of the retaining walls will be backfilled as construction progresses, and the walls will be abandoned in place. Because of the close proximity (about 3 m [10 ft]) of the crane wall to the RB and CB and the significant depth of the relatively stiff crane wall, the staff was concerned that the wall may act as a barrier to reflect the seismic waves due to a kinematic interaction with the surrounding soil. The result could be an increase in the seismic demand on the RB and CB. Therefore, **RAIs 03.07.01-14** and **03.07.01-24** asked the applicant to describe in the FSAR how the effect of a retaining wall is considered in the SSI modeling and analysis of the RB and CB.

In the responses to **RAIs 03.07.01-14** and **03.07.01-24 (ML092430131** and **ML100550613)**, the applicant acknowledges that the seismic soil pressures on the exterior walls of the RB and CB could be affected by the presence of the crane wall. However, because the site-specific SSE input spectra are only about 43 percent of the ABWR DCD SSE spectra (i.e., 0.13g modified RG 1.60 spectra versus 0.3g RG 1.60 spectra) and the retaining wall is a relatively light structure, the change in the seismic soil pressure due to the presence of the crane wall will be more than offset by the reduction from a lower input motion. To confirm the above conclusions, the applicant performed the SSI analysis of the RB and CB by explicitly incorporating the crane wall into the SSI model.

The analyses were performed using 2-D models of the RB and CB with and without the crane wall for the site-specific conditions, including the site-specific SSE and soil properties. The applicant used SASSI2000 software for the SSI analysis. The results of the SSI analysis are in the Revision 1, Supplement 1 response to **RAI 03.07.01-24 (ML102070067)**. The results include a comparison of the maximum forces and moments, the response spectra, and the RB and CB seismic pressures with and without the crane walls. The staff reviewed the above results and concluded that the crane wall has a negligible effect on the resulting forces and moments in the buildings as well as on the in-structure response. The staff also concluded that an increase in dynamic soil pressures on the RB and CB walls due to the crane wall effect is bounded by the corresponding ABWR DCD seismic design pressures (per ABWR DCD Figures 3H.2-11 and 3H.2-14, respectively).

Evaluation of the Effects on RB and CB Mat Foundations

The supplementary information in FSAR Section 3.7.1 under the heading of "Other Analysis" states that:

> In the development of settlement estimates, the representative shear wave velocity value for intervals within a soil column is only one input used in the derivation of the elastic modulus for layers within that column. Since this derived elastic modulus value is first adjusted for strain and then weighted with estimated values derived from either SPT tests (for granular material) or undrained shear strength tests (for cohesive soils) the effect of variability of shear wave velocity upon settlement calculations is significantly attenuated.

On the basis of the above discussion, the applicant concludes that the shear wave velocities at the plant site do not impact the plant design.

RAI 03.07.01-7 asked the applicant to provide the following information:

- A comparison of the estimated spring constant values (and its potential degree of variability) under the mat foundations for the RB and CB for the site-specific conditions and the corresponding ABWR DCD parameters used in the certified design.

- Justification for any differences between the calculated site-specific spring constant values and the ABWR DCD parameters as to their effect on mat design forces.

The applicant's response to RAI 03.07.01-7 (ML092610377) estimates the spring constant values for the site-specific condition and compares the results with those of the ABWR DCD values. The applicant's estimate assumes an undrained Poisson's ratio for the foundation material and concludes that the calculated spring values are higher than those of the ABWR DCD and the mat design is thus acceptable for the RB and CB.

In evaluating the applicant's response, the staff was concerned that the use of the undrained Poisson's ratio may be unconservative for the mat settlement and design forces. The use of the undrained Poisson's ratio (i.e., 0.47–0.48) assumes that the incompressible pore water resists the vertical stresses transmitted to the saturated foundation soils. Nonetheless, depending on the permeability of the foundation soil, the excess pore water pressures can dissipate quickly and transfer the stresses to the soil grains. The staff issued RAI 03.07.01-20 requesting the applicant to compare the site-specific soil spring constant values calculated using the drained

Poisson's ratio of foundation soils with those of the ABWR DCD, and to justify any differences as to their effect on the mat design forces.

In the response to RAI 03.07.01-20 (ML100550613), the applicant compares the calculated spring constants for both the undrained and drained Poisson's ratios for the RB and CB mats. The calculated soil spring constants are higher than those of the ABWR DCD design values except for the CB mat for the LB soil case. To evaluate the impact of the calculated CB lower spring constants on the mat design, the applicant performed a sensitivity analysis using the finite element method to compare the stresses in the CB base mat obtained from the site-specific LB spring values versus those obtained from the ABWR DCD-derived soil spring constants. This analysis was performed for the total dead load of the structure with seismic forces. The applicant describes the model and plots the results of the analyses in the response to RAI 03.07.01-28 (ML102630145). The staff reviewed the above results and found no significant difference in the CB mat stresses calculated using site-specific and ABWR DCD spring values. On the basis of the estimated soil spring constants using site-specific conditions and the results of stresses in the CB mat, the staff concluded that the shear wave velocity departure at the STP Units 3 and 4 site does not adversely impact the RB and CB mat designs. FSAR Subsection 3H.1.5.2 describes the impact of the shear wave velocity on the foundation spring constants and mat design.

In conclusion, the staff reviewed the applicant's information regarding the potential impact of the Tier 1 departure involving the shear wave velocity parameter on the seismic analysis of DCD structures. The staff found that the ABWR DCD results envelop those of the site-specific seismic analysis of the RB and CB.

- STD DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category 1 to Non-Seismic

The staff's review of this departure is documented in Subsection 3.7.2.4 of this SER.

<u>*COL License Information Item*</u>

- COL License Information Item 3.19 — Seismic Design Parameters

This COL license information item requires the COL applicant to demonstrate that the standard design is applicable to the site according to the procedure specified in ABWR DCD Subsection 2.3.1.2. The applicant indicates that Section 2.5S.2 of the COL FSAR provides a site-specific assessment against the Tier 1 site requirements. The site-specific SSI analysis for the RB and the CB is in Appendix 3A of the COL FSAR. NRC staff reviewed the applicant's site-specific analysis, as discussed in the staff's evaluation of Tier 1 Departure STP DEP T1 5.0-1 in this SER. The staff found that the applicant has adequately addressed COL License Information Item 3.19.

3.7.1.4.1 Design Response Spectra

Section 2.5S.2 of the COL FSAR, "Vibratory Ground Motion," describes the development of the site-specific GMRS for the STP Units 3 and 4 site following the guidance in RG 1.208. RG 1.208 incorporates site-specific seismic hazard assessments that satisfy the requirements of 10 CFR 100.23 and lead to the establishment of the SSE. The horizontal GMRS were evaluated by calculating the uniform hazard response spectra (UHRS) from the probabilistic seismic hazard assessment and then propagating this bedrock motion to the ground surface by performing a site response analysis based on seismic wave transmission characteristics of the site and performance-based procedures. The vertical GMRS were developed by scaling the horizontal GMRS using a frequency-dependent, vertical-to-horizontal factor (FSAR Subsection 2.5S.2.6). The GMRS were characterized by horizontal and vertical response spectra for a 5 percent damping determined as free-field motions on the ground surface.

FSAR Subsection 2.5S.2.5.4, "Site Response Analysis," describes the methodology used for the site response analysis to develop the GMRS and FIRS from the UHRS at the STP Units 3 and 4 site. The probabilistic computer program PSHAKE, which is based on the Random Vibration Theory, was used for site response analysis. This program is similar to the deterministic computer program SHAKE except that (a) the input motion is provided in terms of the acceleration response spectrum and associated spectral damping instead of the spectrum-compatible acceleration time histories; and (b) the output is obtained in terms of the power spectral density (PSD) instead of the acceleration time history responses, which is then converted to the acceleration response spectrum at each soil layer interface. Section 2.5S of this SER documents the acceptance of the GMRS.

FSAR Section 3A.16.1 and Subsection 3H.6.5.1.1.1, "Design Response Spectra," describe the development of the site-specific horizontal and vertical SSE design spectra for the STP Units 3 and 4 site. The SSE design spectra are defined at the free-field ground surface and are set equal to the 0.13g RG 1.60 response spectra in the horizontal and vertical directions, except that the spectral accelerations below 2.5 Hz for the horizontal direction and 3.5 Hz for the vertical direction were increased to envelop the corresponding GMRS. To assess the acceptability of the site-specific SSE design spectra for the seismic reconciliation analysis of RB and CB as well as the seismic evaluation and design of the site-specific seismic Category I structures, the staff issued **RAI 03.07.01-2** followed by **RAI 03.07.01-15** requesting the applicant to compare the site-specific SSE design response spectra with the ABWR DCD design spectra (certified seismic design response spectra) and the GMRS. In addition, the staff requested the applicant to provide both the SSE and OBE spectra for the horizontal and vertical directions at the finished grade of the free field, for all applicable damping values used in the analysis.

In the responses to the above RAIs (**ML092370556** and **ML100480204**), the applicant provides comparisons of the ABWR DCD design spectrum with the GMRS and the site-specific SSE spectra for the horizontal and vertical directions, including proposed changes in the FSAR. The staff reviewed the above results and found that the ABWR DCD design spectra envelop the GMRS and site-specific design spectra by a significant margin. The applicant also compares the site-specific SSE design spectra and the GMRS in FSAR Figure 3H.6-1 for the horizontal direction and in Figure 3H.6-2 for the vertical direction. The staff reviewed these comparisons and found that the site-specific SSE design spectra for both the horizontal and vertical directions envelop the GMRS at all frequencies for these structures.

FSAR Figure 3.7-1b shows the SSE design spectra for the 2, 3, 4, and 7 percent damping for the horizontal component and Figure 3.7-2b shows the same data for the vertical direction. The staff will use these spectra as target spectra to assess the acceptance of the SSE design synthetic time histories matched to 5 percent-damped SSE design spectra, as discussed later. In regards to the OBE spectra, the OBE is not a design-basis earthquake for the ABWR DCD. The plant shutdown criteria described in ABWR DCD Subsection 3.7.4.4 are based on the site-specific SSE response spectra shown in Figures 3.7-1a and 3.7-2a. The staff found the STP Units 3 and 4 plant shutdown criteria acceptable because it meets the ABWR DCD Subsection 3.7.4.4 requirement.

To further assess the acceptance of the input spectra at the free-field ground surface **RAIs 03.07.01-2**, **03.07.01-3**, **03.07.01-10, 03.07.01-13**, **03.07.01-15**, **03.07.01-16,** and **03.07.01-23** asked the applicant to describe how the FIRS at the foundation of each seismic Category I structure were determined (including the soil model, soil properties, computer programs, and analysis assumptions). In addition, the RAIs asked the applicant to compare the FIRS and the envelope of the three response spectra obtained (through the deconvolution analysis with three SSI soil profiles) using the SHAKE program, with the input design time history as applied to the free-field ground surface. The RAIs also asked the applicant to show that the envelope of the three response spectra obtained above will envelope the FIRS and meet the requirement of 10 CFR Part 50, Appendix S (i.e., the horizontal component of the SSE ground motion in the free field at the foundation levels of structures must be an appropriate response spectrum with a peak ground acceleration of at least 0.1g).

In the responses to **RAI 03.07.01-2** and its supplements **(ML092660655, ML101340651** and **ML111050565)**, the applicant confirms that the FIRS were calculated using the same UHRS and site response analysis procedure that were used to develop the GMRS as described in FSAR Section 2.5S.2. The FSAR was revised to include this clarification in Subsection 2.5S.2.6. The procedure used for developing the foundation motion at the free-field SHAKE outcrop of each seismic Category I structure from the input spectra applied to the ground surface is consistent with the guidance in DC/COL-ISG-017, "Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses." The applicant compares the SSE-based foundation response spectra for two horizontal and vertical directions for three soil profiles (LB, mean, and UB) with the FIRS and 0.1g RG 1.60 in Figures 3A-233 through 3A-241 for the RB; Figures 3A-242 through 3A-250 for the CB; Figures 3H.6-3a through 3H.6-5c for the UHS basin; Figures 3H.6-6a through 3H.6-8c for the RSW piping tunnel; Figures 3H.6-9a through 3H.6-11c for the RSW pump house; Figures 3H.6-11d through 3H.6-11 for the DGFOSV; and Figures 3H.7-22 through 3H.7-30 for the DGFOT. These figures indicate that the SSE-based response spectra calculated at the foundation levels of the seismic Category I structures referred to above envelope the corresponding FIRS and an appropriate broad band spectra characterized by the 0.1g RG 1.60 spectra.

The staff reviewed the results presented above and concluded that the site-specific SSE design spectra meet the SRP Section 3.7.1 acceptance criteria and are therefore acceptable. The staff performed the technical review of the design response spectra developments, as discussed above, in accordance with the guidance in SRP Section 3.7.1, "Seismic Design Parameters", and DC/COL-ISG-17.

3.7.1.4.2 Design Time Histories

Development of Site-Specific SSE Design Time Histories

To evaluate the acceptability of the applicant's site-specific design time histories in accordance with SRP Section 3.7.1, SRP Acceptance Criteria 1.A and 1 B, the staff issued **RAIs 03.07.01-2**, **03.07.01-11**, **03.07.01-15**, and **03.07.01-21** requesting the following information that was not included in the original FSAR:

1. Design time histories for three orthogonal directions including the time step and duration of input motions

2. A demonstration that each pair of time histories is statistically independent.

3. The range of frequencies at which spectral accelerations were calculated and confirmation that the 5 percent damped response spectrum of the synthetic time histories does not fall more than 10 percent below the target response spectrum at any one frequency.

4. Comparisons of the response spectra of the design time histories and target design spectra for damping values of 2, 3, 4 and 7 percent.

5. A demonstration that the design time histories have adequate strong motion duration and appropriate velocity/acceleration (V/A) and AD/V^2 ratios (ratio of acceleration times distance over square of the velocity) consistent with characteristic values for the magnitude and distance of the controlling seismic events defined by the uniform hazard.

The staff reviewed the applicant's responses that provide the basis for accepting the design time histories.

1. In the response to RAI 03.07.01-2 and its supplements, the applicant provides plots of the acceleration, velocity, and displacement time histories in the x-, y-, and z-directions. The staff reviewed these time history plots and found that the selected duration of the synthetic time histories (using the Taft earthquake as seed motion) did not provide sufficient time for these time histories to attenuate to low residual ground motion values at the end of earthquake duration. Therefore, RAI 03.07.01-11 asked the applicant to justify the selected duration of the synthetic time histories (per the guidance of SRP Section 3.7.1, SRP Acceptance Criterion 1.B) that do not reflect the real earthquake characteristics.

   In the responses to RAI 03.07.01-11 (ML092510038) and RAI 03.07.01-21 (ML100480204), the applicant states that the duration of time histories for the Arias intensity to rise from 5 percent to 75 percent is 11.2 seconds for the two horizontal design time histories and 12.2 seconds for the vertical design time history. For the characteristic earthquake time history, this duration was calculated to be 20 to 45 seconds. The applicant justifies the shorter duration of the design time history based on the following:

   a) In accordance with SRP Subsection 3.7.1.II, "SRP Acceptance Criteria," synthetic time histories should be based on seed motions from recorded earthquakes. Strong motion recorded earthquakes with a 20-45 seconds duration of the time histories for Arias intensity to rise from 5 percent to 75 percent are not readily available to be used for the seed time histories to generate the synthetic time histories.

   b) The time histories are used for the linear analysis. For the linear analysis, the duration of time histories is not critical provided that the duration is comparable to recorded strong motion earthquakes and the spectra of time history closely match the target response spectra. For the design time histories, the duration is consistent with the Taft earthquake and the time history closely matches the target response spectra.

   c) In the actual analysis, trailing zeros (quiet time) are added to the synthetic time history, thus making the total duration of the time history 40.96 seconds.

d) The ground motions for the Taft earthquake (in California) after 22 seconds are small and the peak responses occur at a much earlier time than the end of the time history record. As such, the linear analysis results are not expected to be affected by the time history not attenuating to residual values.

Based on a review of the time histories and the fact that the strong motion duration of the synthetic time history meets the minimum duration of 6 seconds as described in SRP Section 3.7.1 acceptance criteria, the staff concluded that this characteristic of the synthetic time histories is acceptable.

2. In the response to **RAI 03.07.01-11 (ML092510038)**, the applicant states that for the time history analysis, the two horizontal and one vertical time histories are applied separately and the maximum responses are combined using the square root of the sum of the squares (SRSS) or the 100-40-40 percent spatial combination rule. Therefore, because the applicant followed RG 1.92, Revision 2, without exception, the staff concluded that the statistical independence of the three time history components developed above is not required.

3. In the response to **RAI 03.07.01-15 (ML100480204)**, the applicant provides comparisons of the 5 percent damped response spectrum of the synthetic time history; the SSE target spectra; 1.3 times the SSE target spectrum; and the GMRS in FSAR Figure 3H.6-12 for the E-W direction; Figure 3H.6-13 for the N-S direction; and Figure 3H.6-14 for the vertical direction. The applicant also provides the response spectra of the synthetic time histories for comparison with the target spectra at 275 calculated frequency points in Tables 3H.6-2d through 3H.6-2f. Tables 3H.6-2d through 3H.6-2f show the extent (in percentages) of the 5 percent damped response spectra of the synthetic time histories falling below the target response spectrum at any frequency. In addition, per paragraph ii.(d) of SRP Acceptance Criterion 1.B, Option 1, Approach 2 of SRP Section 3.7.1, the computed 5 percent damped response spectrum of the artificial time history does not exceed the target response spectrum at any frequency by more than 30 percent. The staff reviewed the above results and concluded that the spectra are acceptable because the number of calculated frequency points is large enough to eliminate uncertainties associated with spectra calculations. Additionally, the calculated spectra of the synthetic motions satisfy the acceptance criteria of SRP Section 3.7.1 in that the 5 percent damped response spectrum of the synthetic time histories does not rise more than 30 percent above and fall more than 10 percent below the target response spectrum at any one frequency.

4. In the response to **RAI 03.07.01-15 (ML100480204)**, the applicant compares the response spectra of site-specific design time histories and target spectra for 2, 3, 4, and 7 percent damping for the E-W direction in Figures 3H.6-12a through 3H.6-12d; for the N-S direction in Figures 3H.6-13a through 3H.6-13d; and for the vertical direction in Figures 3H.6-14a through 3H.6-14d. These comparisons were requested so that the appropriateness of the synthetic design motions can be evaluated for different damping values that are significant to the design of site-specific structures. The staff reviewed the above results and found that (with the exception of a few frequencies) the 2, 3, 4, and 7 percent damped response spectra of the synthetic design motions envelop the corresponding target spectra at all frequencies. The few frequencies where the response spectra of the synthetic design motions fall slightly below the target spectra are insignificant and well covered by the margin in the design spectra compared to the GMRS and 0.1g RG 1.60 spectra.

5. In the response to **RAI 03.07.01-11 (ML092510038)**, the applicant states that the calculated characteristic earthquake V/A was 52 to 115 cm/s/g (20.47 and 45.28 in./s/g) and the AD/V^2 was 2.03 to 5.28. For the design time histories, the V/A was 230, 288, and 167 cm/s/g (90.55, 113.39, and 65.75 in./s/g) for the two horizontal and one vertical components, respectively; and the AD/V^2 values were 2.08, 1.89, and 3.02, respectively. This variation between the design time histories and the characteristic earthquake is due to the conservative design response spectra. The design response spectrum is a 0.13g RG 1.60 spectrum with enhanced low-frequency content to account for the very deep soil site. The comparison of the V/A and AD/V^2 value of the characteristic earthquake and the conservative design response spectra shows that the design response spectra have a higher energy content (a greater maximum velocity) and are therefore conservative. The staff found the design time histories acceptable because the design time histories have higher energy contents than the GMRS and the 0.1g RG 1.60 spectra.

Development of Standard Plant SSE Time Histories

Seismic analyses of the DGFOSV and DGFOT use the SSE ground motion spectra in Table 5.0 of ABWR DCD Tier 1, in addition to the site-specific SSE ground motions described in FSAR Sections 3H.6.7 and 3H.7. Since the ABWR DCD does not include digitized information for the SSE time histories, RAI 03.07.01-2 Supplement 2 asked the applicant to demonstrate that the new standard plant time histories are consistent with the RG 1.60 response spectra anchored to a peak ground acceleration of 0.3g and satisfy SRP Section 3.7.1 acceptance criteria. In the response to **RAI 03.07.01-2** Supplement 2 dated April 11, 2011 **(ML111050565)**, the applicant states that the acceleration time history records obtained from the 1994 Northridge Earthquake were used as seed time histories for generating new synthetic time histories. The applicant adds that these new time histories were developed in accordance with SRP Section 3.7.1 acceptance criteria using computer programs SYNQKE-R, HIST, and QUAKE, as described in FSAR Appendix 3C.

Plots of the acceleration, velocity, and displacement time histories of one vertical component and two horizontal components are shown in FSAR Figures 3H.8-1 through 3H.8-3. Plots of the response spectra for 2, 3, 4, 5, and 7 percent damping that compare the target response spectra (RG 1.60 spectra) with the synthetic time histories spectra are shown in FSAR Figures 3H.8-4 through 3H.8-18. Plots of the PSD functions that compare the target PSD corresponding to the RG 1.60 spectra with the PSD of the synthetic time histories are shown in FSAR Figures 3H.8-19 through 3H.8-21. The staff reviewed the above results and found the new standard plant SSE time histories acceptable, in accordance with SRP Section 3.7.1 acceptance criteria.

3.7.1.4.3 Percentage of Critical Damping Values

In **RAI 03.07.01-12**, the staff asked the applicant to provide the percentages of critical damping values used in the seismic analysis of site-specific Category I SSCs. The staff also asked the applicant to justify any exceptions that were taken from RG 1.61; to provide a plant-specific technical basis (in reference to Regulatory Position C1.2 of RG 1.61) for using damping values higher than the OBE damping values specified in Table 2 of RG 1.61, but not greater than the SSE damping values specified in Table 1 of RG 1.61; and to provide the soil material damping values used in the analysis.

In the response to **RAI 03.07.01-12 (ML092370556)**, the applicant states that the damping values used for SSCs are the same as those listed in ABWR DCD Table 3.7.1. Those damping values are the same as in RG 1.61 and RG1.84, except for the cable trays and conduits, as stated in ABWR DCD Subsection 3.7.1.3. The applicant also states that the SSE damping values are used to generate the ISRS because the UHS/RSW pump house is highly stressed under an SSE event. The response also updates FSAR Table 3H.6-1 with the strain-compatible soil damping values.

The staff reviewed the response to **RAI 03.07.01-12** and issued **RAI 03.07.01-22 (ML110730069)**. This RAI requested additional information on stress levels, including comparisons with code-level allowable stresses that justify the use of the SSE level damping values for each of the site-specific seismic Category I structures. In the response to **RAI 03.07.01-22 (ML100480204)**, the applicant states that the SSE damping is no longer used in the latest SSI analysis of the UHS/RSW pump house to generate the ISRS. Instead, an OBE damping value of 4 percent is used in accordance with Table 2 of RG 1.61, Revision 1. The applicant also clarifies the damping values used to generate the ISRS and updates FSAR Subsection 3H.6.5.1.2 to state that:

> *The OBE damping values were used for the generation of in-structure response spectra (ISRS) for all site-specific Seismic Category I structures. The only exception is the cracked case SSI analysis for the Reactor Service Water (RSW) Piping Tunnels where SSE damping (i.e. 7%) was used because of high stress levels. All other SSI analysis cases of RSW Piping Tunnels used OBE damping (i.e. 4%) damping.*

During the NRC audit conducted March 14 through 18, 2011, the staff confirmed that the applicant had used SSE damping to generate the ISRS for the cracked case RSW piping tunnel. During this audit, the staff reviewed the pertinent stress calculations and found them acceptable. The calculation for the cracked case RSW piping tunnel shows a significant amount of stress based on the calculated ratio of required capacity to provided capacity, thus justifying the use of 7 percent damping. For all other site-specific seismic Category I structures, the applicant uses the OBE damping of 4 percent to generate the ISRS in accordance with RG 1.61, Revision 1; and is therefore acceptable.

3.7.1.4.4 Supporting Media for Seismic Category I Structures

RAIs 03.07.01-6, **03.07.01-13**, **03.07.01-19,** and **03.07.01-27** asked the applicant to describe the supporting media for each Category I structure, the dimensions of the structural foundation, and the actual structural height. Because the minimum shear wave velocities may be less than 305 m/s (1,000 ft/s), the RAIs asked the applicant to provide quantitative results of additional studies performed to consider the potential impact of the actual site-specific shear wave velocity (including its degree of variability) on soil-structure interaction, settlement calculations, and the design of foundation elements.

In the responses to **RAIs 03.07.01-6**, **03.07.01-13**, **03.07.01-19,** and **03.07.01-27 S2 R1 (ML092610377, ML092360772, ML101620284,** and **ML110730064)**, the applicant provides the overall building dimensions and embedment depth for the RB and CB structures in Section 3A.17; for the UHS basin, UHS cooling towers, RSW pump house, and RSW piping tunnel in Subsection 3H.6.5.1.3; for the DGFOSV in Section 3H.6.7; and for the DGFOT in Section 3H.7.3. The applicant includes adequate information about the supporting media for seismic Category I structures per the acceptance criteria of SRP Subsection 3.7.1.II.3. The staff's evaluation of the site-specific analysis for the RB and CB is discussed earlier in this SER. Staff's evaluation of the analysis for other structures and the results are documented in Subsection 3.7.2.4 of this SER.

Other Analyses

Section 2.5 of this SER reviews the liquefaction, bearing capacity, and lateral earth pressure evaluations. The effect of the shear wave velocity departure on the RB and CB mat foundations is reviewed earlier in this section, and in Subsection 3.8.4.4.1, Part C, of this SER.

3.7.1.5 *Post Combined License Activities*

The applicant identifies the following site-specific ITAAC to verify soil properties:

- Table 3.0-11, Backfill under Category I Structures

3.7.1.6 *Conclusion*

The NRC staff's finding related to information incorporated by reference is in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the seismic design parameters that were incorporated by reference have been resolved.

The staff's review confirmed that the applicant has adequately addressed the Tier 1 departures relevant to this section in accordance with Section 3.7.1 of NUREG–0800.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations, the guidance in Section 3.7.1 of NUREG–0800, and other NRC RGs. The staff's review concluded that the applicant has adequately addressed COL License Information Item 3.19 and has provided sufficient supplemental information to satisfy the NRC requirements and guidance in Section 3.7.1 of NUREG–0800.

3.7.2 Seismic System Analysis

3.7.2.1 *Introduction*

This FSAR section addresses the seismic analysis methods and acceptance criteria used for the ABWR seismic Category I structures.

As documented in Subsections 2.5S.4.4 and 2.5S.4.7 of the STP Units 3 and 4 COL FSAR, the shear wave velocity at the STP Units 3 and 4 site varies horizontally within a soil stratum; varies vertically with depth; and shows values lower than the minimum 305 m/s (1,000 ft/s) specified in the ABWR DCD. Because the site profile is outside the range of soil profiles considered in the ABWR DCD, in accordance with the guidance of Sections 3.7.1 and 3.7.2 of NUREG–0800, site-specific SSI analyses are needed to verify that the seismic design documented in the ABWR DCD is applicable to the STP Units 3 and 4 site.

The SSI analytical approach is based on the finite-element method using the substructuring technique. The computer program SASSI2000 was used for the analysis. This computer program uses finite elements with complex moduli for modeling structural and foundation properties, and it is based on the flexible volume method of substructuring and the frequency domain complex response method of analysis. The SSI methodology is the same as the methodology described in Sections 3A.5.2 and 3A.5.3 of the ABWR DCD. The methodology for the free-field site response will be the same as the methodology described in Section 3A.6 of the ABWR DCD.

The scope of the staff's review of the seismic analysis of safety-related SSCs and equipment includes the following:

- Seismic analysis methods
- Natural frequencies and responses
- Procedures used for analytical modeling
- Soil-structure interaction
- Development of in-structure response spectra
- Three components of earthquake motion
- Combination of modal responses
- Interaction of non-Category I structures with Category I SSCs
- Effects of parameter variations on FRS
- Use of equivalent vertical static factors
- Method used to account for torsional effects
- Comparison of responses
- Analysis procedure for damping
- Determination of seismic overturning moments and sliding forces for seismic Category I structures

3.7.2.2 *Summary of Application*

Section 3.7.2 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.7.2 of the certified ABWR DCD, Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in FSAR Section 3.7.2, the applicant provides the following:

Tier 1 Departures

- STP DEP T1 5.0-1 — Site Parameters (shear wave velocity departure)

This departure states that the shear wave velocity at the STP Units 3 and 4 site varies both horizontally within a soil stratum and vertically with depth and does not meet the minimum shear wave velocity requirements of ABWR DCD Table 5.0. The applicant indicates that a site-specific SSI analysis, as discussed in Appendix 3A of the COL FSAR, was performed using site-specific soil properties and site-specific SSE ground motions to confirm that the standard plant results in the ABWR DCD envelop the results of the site-specific SSI analysis.

- STP DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category I to Non-Seismic

The referenced ABWR DCD Tier 1, Section 2.15.13 states that the exterior walls of the RWB below grade and the basemat are classified as seismic Category I structures. This departure changes the seismic category of the RWB substructure from seismic Category I to non-seismic. The applicant refers to NUREG–1503, Section 3.8.4, which states that the RWB is not a seismic Category I structure. The RWB does not house any safety-related systems or components. RG 1.143 provides detailed guidance for the design of the radwaste processing SSCs. This departure commits to follow the guidance of RG 1.143, Revision 2.

COL License Information Item

- COL License Information Item 3.22 — Assessment of Interaction Due to Seismic Effects

The applicant commits (COM 3.7-2) to developing a procedure "to confirm that all nonsafety-related SSCs located in the same room as safety-related SSCs are evaluated and correctly dispositioned for inspection of the as-built plant for II/I interactions."

Supplemental Information

The applicant adds the following to the COL FSAR:

1. Section 3H.6, "Site-Specific Seismic Category 1 Structure," which provides supplemental information on the design of the UHS, RSW piping tunnel, and DGFOSV.

2. Section 3H.7, "Diesel Generator Fuel Oil Tunnel," which provides supplemental information on the design and analysis of the DGFOT.

3. Section 3H.8, "Development of Standard Plant SSE Time Histories," which provides SSE time histories consistent with RG 1.60 spectra anchored at 0.3g.

4. Section 3H.9, "Extreme Environmental Design Parameters for Seismic Analysis, Design, Stability Evaluation and seismic Category II/I Design," which shows the extreme environmental design parameters used for seismic analysis, structural design, stability evaluation, and seismic Category II/I design of the UHS/RSW pump house, RSW piping tunnel, RWB, control building annex (CBA), and service building (SB).

5. Section 3H.10, “STP 3 & 4 Resolution of Issues with Subtraction Method of Analysis Identified by DNFSB,” which addresses the Defense Nuclear Facilities Safety Board’s (DNFSBs) issues with the SASSI subtraction method (SM) identified in its letter from Peter S. Winokur to Daniel B. Poneman of the U.S. Department of Energy (DOE), dated April 8, 2011.

The applicant performed site-specific SSI analyses of the site-specific structures including the RSW piping tunnel and the UHS/RSW pump house. The applicant also updated the COL FSAR with the results of the final seismic analysis.

3.7.2.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for seismic system analysis and the associated acceptance criteria are in Section 3.7.2 of NUREG–0800.

In accordance with Section VIII, “Processes and Changes and Departures,” of, “Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor,” the applicant identifies Tier 1 departures. Tier 1 departures require prior NRC approval and are subject to the requirements specified in 10 CFR Part 52, Appendix A, Section VIII.A.4.

The regulatory guidance for the seismic analysis of safety-related SSCs and equipment includes the following:

- RG 1.60, Revision 1, “Design Response Spectra for the Seismic Design of Nuclear Power Plants”
- RG 1.61, Revision 1, “Damping Values for the Seismic Design of Nuclear Power Plants”
- RG 1.92, Revision 2, “Combining Modal Responses and Spatial Components in Seismic Response Analysis”
- RG 1.122, Revision 1, "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components"
- DC/COL-ISG-017, “Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses”

3.7.2.4 *Technical Evaluation*

As documented in NUREG–1503, the NRC staff reviewed and approved Section 3.7.2 of the certified ABWR DCD. The staff reviewed Section 3.7.2 of the COL FSAR Revision 7 and checked the referenced ABWR DCD to ensure that the combination of information in the COL FSAR and in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff’s review confirmed that the information in the application and the information incorporated by reference both address the required information relating to this section.

[1] See *“Finality of Referenced NRC Approvals”* in SER Section 1.1.3, for a discussion on the staff’s review related to verification of the scope of information to be included in a COL application that references a design certification.

The staff reviewed the information in the COL FSAR:

Tier 1 Departures

- STP DEP T1 5.0-1 Site Parameters

The staff's review of this departure is documented in Subsection 3.7.1.4 of this SER.

- STD DEP T1 2.15-1 Reclassification of Radwaste Building from Seismic Category 1 to Non-Seismic

ABWR DCD Tier 2 shows the RWB located on the west side of the CB, with the south edge of the RWB aligned with the north edge of the RB (ABWR DCD Figure 1.2-1). However, this configuration changed in the STP Units 3 and 4 site where the RWB was relocated immediately west of the RB and RSW piping tunnel. Due to the close proximity of the RWB to the RB and RSW piping tunnel, and the fact that the RWB is classified as a non-seismic Category I structure with the potential to interact with seismic Category I structures, **RAI 03.07.02-11** and **RAI 03.07.02-19** asked the applicant to provide the seismic input motion incorporating the effects of the SSSI for a Category II/I interaction evaluation of the RWB. The staff also asked the applicant to include the design spectra input for the RWB in the COL FSAR. In addition, the RAIs asked the applicant to address any potential increases in dynamic soil pressures on the RWB exterior walls due to SSSI effects.

In the September 9, 2009, response to **RAI 03.07.02-11 (ML092530685)** and the April 29, 2010, response to **RAI 03.07.02-19 (ML101250162)**, the applicant states that the earthquake input used at the foundation level was enveloped with 0.3g RG 1.60 response spectra and the induced acceleration response spectra due to the site-specific SSE, which was determined from an SSI analysis accounting for the impact of the nearby RB. In this SSI analysis, five interaction nodes corresponding to the four corners and the center of the RB were added to the 3-D SSI model of the RB at depths corresponding to the bottom elevation of the RWB foundation. The average response of these five interaction nodes was enveloped with 0.3g RG 1.60 spectra to determine the SSE input motion at the foundation level. In addition, the applicant provides the SSE input motion at the foundation level of the RWB in terms of 7 percent damped horizontal and vertical smooth acceleration response spectra in COL FSAR Figures 3.7-41 and 3.7-42, respectively.

The staff reviewed the design spectra at the foundation level of the RWB, as well as the procedure used to estimate these spectra. The staff concluded that the design spectra input for the non-seismic Category I RWB are acceptable because:

- The RWB is a much lighter structure than the RB. Therefore, the soil motions at the foundation level of the RWB, calculated from the SSI analysis of the RB using site-specific SSE, should adequately capture the RB's inertial effects.

- The foundation motion calculated from the SSI analysis of the RB was further enveloped by 0.3g RG 1.60 spectra, which is significantly higher than the site-specific SSE.

In order to assess the effects of the SSSI on the lateral pressures on the RWB walls, the applicant performed the SSSI analysis of the E-W 2-D section of the RWB with and without the RB and RSW piping tunnel using SASSI2000. The 2-D SSSI model of the RWB, the RSW piping tunnel, and the RB is shown in COL FSAR Figure 3H.6-210. The details of the analysis are described in COL FSAR Subsection 3H.6.5.3 and in the Supplement 1 response to **RAI 03.07.02-24 (ML103360074)**. The SSSI analysis was performed for in situ and backfill UB soil with a Poisson's ratio limit of 0.495 for saturated layers. The site-specific SSE motions were used as inputs to the SSSI analysis. The calculated seismic soil pressures on the RWB east and west walls are shown in COL FSAR Figure 3H.3-50 and Figure 3H.3-51, respectively. These figures show a significant increase in seismic soil pressures on the RWB east and west walls due to SSSI effects. The staff reviewed the analysis procedure and results and concluded that the large increase in lateral seismic soil pressures on the RWB is justified due to the very close proximity of the three structures.

Because the applicant considered the inertial effect of the RB through the SSI analysis, along with the SSSI effect in establishing the seismic input design spectra and seismic soil pressure demand, the staff considered the seismic input and the soil pressure used for the design of the RWB acceptable.

This departure and the design of the RWB walls and the stability evaluation of the RWB for II/I evaluation are further reviewed in Subsection 3.8.4.4 of this SER.

COL License Information Item

- COL License Information Item 3.22 Assessment of Interaction Due to Seismic Effects

Subsection 3.7.3.4 of this SER documents the staff's review of this COL license information item.

Supplemental Information

3.7.2.4.1 Seismic Analysis Methods

RAI 03.07.02-2 asked the applicant to describe the seismic analytical methods used for site-specific Category I structures (including the UHS/RSW pump house and the RSW piping tunnel) in sufficient detail comparable to ABWR DCD Subsection 3.7.2.1.

In the September 3, 2009, response to **RAI 03.07.02-2 (ML092510038)**, the applicant states that the seismic analysis of the UHS/RSW pump house was performed with SASSI2000 using a frequency-domain time history analysis, as described in Subsection 3H.6.5.2.1 of the COL FSAR. The applicant states that the SSI analyses were performed for three orthogonal (two horizontal and one vertical) directions accounting for the translational, rocking, and torsional responses of the structures and foundations. The model consisted primarily of plate, beam, and solid elements. The density of the elements, which were modified to account for applicable live loads, represented structural mass. Lumped masses were used to represent the weight of the equipment in the RSW pump house, and the impulsive water mass was calculated using the procedure described in Commentary Section C3.5.4 of American Society of Civil Engineer (ASCE) 4–98, "Seismic Analysis of Safety-Related Nuclear Structures," January 1, 2000. The UHS/RSW pump house model is in FSAR Figure 3H.6-40.

In the same response, the applicant states that an equivalent static analytical method was used for the seismic analysis of the RSW piping tunnel, as described in Subsection 3H.6.6.2.2 of the COL FSAR. Dynamic soil pressures on the tunnel walls were computed using the method described in ASCE 4–98, Subsection 3.5.3.2. Strains created in the tunnel walls due to the passage of seismic waves through the soil during the SSI were computed using the method described in ASCE 4–98, Subsection 3.5.2.1. In addition, the applicant states that the concrete elements of the buried RSW piping tunnel were sized so that the structure was rigid, with a minimum frequency greater than 33 Hz and without in-structure amplification. Therefore, the input spectra were used as the ISRS.

The staff reviewed Subsection 3H.6.5.2, "Seismic System Analysis," and the corresponding Appendix 3H subsections and found the information in those subsections to be incomplete. Therefore, follow-up **RAI 03.07.02-14** specifically asked the applicant to provide the following additional information regarding "Seismic Analysis Methods" and to include this information in the COL FSAR:

1. The finite element model

2. The method used to model backfill material in the SSI analysis

3. The method used to incorporate ground water effects in the SSI analysis

4. The analysis method used to obtain seismic forces and moments for design evaluations

5. The analysis method used to model concrete cracking

6. The analysis method used to assess the effects of soil separation from the walls

In the February 10, 2010, response to **RAI 03.07.02-14 (ML100550613)**, the applicant provides the finite element model in Figure 3H.6-40 requested in Item 1 above.

In the response to Item 2, the applicant states that the effect of backfill material in the SSI analysis is included by performing separate SSI analyses of the UHS/RSW pump house incorporating backfill horizons behind the walls. The results of these analyses were enveloped with the results obtained from using an in situ soil model behind the walls. The applicant adds that the calculation of strain-compatible properties for the backfill material is described in the response to Item 3 of **RAI 03.07.02-17**.

A response to Item 3 concerning the modeling of ground water effects in the SSI analysis is in the response to Item 2 of **RAI 03.07.01-17**. A response to Item 5 concerning the modeling of concrete cracking is in the response to **RAI 03.07.02-8**. A response to Item 4 concerning the method used to obtain the seismic forces is in the response to Item 11 of **RAI 03.07.02-15**.

With respect to analytical modeling, the staff's review of Items 2, 3, and 5 concerning the modeling of strain-compatible backfill properties; ground water effects; and concrete cracking in the SSI analysis is described below in Subsection 3.7.2.4.3, "Procedures Used for Analytical Modeling." Also for the SSI analysis, the staff's review of Item 4 concerning the analysis performed to obtain seismic forces is described under the topic "UHS/RSW Pump House" in Subsection 3.7.2.4.4, "Soil-Structure Interaction," of this SER.

Effects of Soil Separation from the Walls

In the response to Item 6, the applicant states that the method recommended in ASCE 4–98, Subsection 3.3.1.9 was used to evaluate the effects of soil separation from the walls. The ASCE 4–98 criteria are general, and the NRC has not endorsed them for estimating the depth of soil separation for seismic Category I structures such as the UHS/RSW pump house.

Therefore, follow-up **RAI 03.07.02-23** asked the applicant to provide an additional basis to justify the conservatism of ASCE guidelines in estimating the depth of soil separation. In the September 15, 2010, response to **RAI 03.07.02-23 (ML102630145)**, the applicant calculates the dynamic soil pressures along the height of each soil-bearing wall from the SSI analysis of the UHS/RSW pump house and compares the results with the static soil pressures acting on the walls. From this comparison, the applicant calculates the net negative soil pressure exerted on each wall for two conditions: one position assumes that the ground water table is located below the pump house basemat and the other assumes that the ground water table is located 1.8 m (6 ft) below grade level. The results for the first case are in the response to **RAI 03.07.02-23**. On the basis of the results of these analyses, the applicant confirmed that the 6.1-m (20-ft) soil separation depth used in the SSI analysis case based on ASCE 4–98 is justified. The staff reviewed these analytical assumptions and results and found that the ASCE 4–98 method for establishing the 6.1-m (20-ft) soil separation depth for the site-specific SSI analysis to evaluate SSE input is acceptable.

In conclusion, the seismic SSI analyses of site-specific seismic Category I structures including the UHS/RSW pump house, RSW piping tunnel, DGFOSV, and DGFOT were performed using a frequency-domain time history analytical method described in Appendix A of the ABWR DCD using SASSI2000. The ABWR DCD used the DM analysis in SASSI. The STP Units 3 and 4 originally used the SM analysis for all seismic SSSI and some SSI analyses. In a letter from Peter S. Winokur of the DNFSB to Daniel B. Poneman of the DOE, dated April 8, 2011, DNFSB identified a technical issue (see Subsection 3.7.2.2 of this SER) in the SASSI when SM is used to analyze embedded structures—the results may be nonconservative. As discussed in Section 3H.10 of the COL FSAR, STP Units 3 and 4 have used either DM or a modified subtraction method (MSM) to re-analyze all site-specific seismic Category I structures that were originally analyzed using the SM in SASSI2000. The NRC staff reviewed the applicability of the MSM in SASSI2000 for the STP Units 3 and 4 applications. On the basis of this review and as described in Subsection 3.7.2.4.20, “DNFSB SASSI Subtraction Method Issues,” of this SER, the staff found the seismic SSI analytical methods acceptable.

3.7.2.4.2 Natural Frequencies and Responses

In the response to Item 1 of **RAI 03.07.02-15 (ML100550613)**, the applicant provides the fixed-base natural frequencies and mass participation factors for the UHS/RSW pump house for up to 33 Hz in Table 3H.6-3, as part of the Supplement 1 response to **RAI 03.07.01-13.** In the response to Item 2 of **RAI 03.07.02-15**, the applicant provides the maximum accelerations and displacements in Table 3H.6-4 as part of the Supplement 1 response to **RAI 03.07.01-13.** The applicant also provides the broadened ISRS that were calculated at several key locations in the structure in Figures 3H.6-16 through 3H.6-39 in the Supplement 1 response to **RAI 03.07.01-13 (ML093270047)**.

A review of the fixed-base natural frequencies and response spectra indicate that the dominant modes of the UHS/RSW pump house are below 23 Hz in all three (x-, y- and z-) directions. The maximum accelerations reported in Table 3H.6-4 indicates significant amplification of the pump house operating floor and roof slab in the vertical direction. Similarly large amplifications were noted for the bottom, mid-level, and top of the cooling tower wall accelerations in the x- and y-directions. The reported maximum displacements for the cooling tower walls in Table 3H.6-4 are also amplified significantly with respect to the UHS basemat. These local amplifications are attributed to the out-of-plane response of the floor, roof, and wall slabs.

The applicant addressed the staff's concerns regarding the effects of structural and soil model refinement, ground water table, and cracked concrete cases in the analysis.
Subsection 3.7.2.4.3, "Procedures Used for Analytical Modeling," of this SER, documents the staff's assessment of the applicant's sensitivity studies addressing the impact of the effects of structural and soil model refinement, ground water table, and cracked concrete cases on the calculated seismic responses. On the basis of the above evaluations and the applicant's information, the staff, per SRP Section 3.7.2, SRP Acceptance Criterion 2, found the information on natural frequencies and the responses in the FSAR acceptable.

3.7.2.4.3 Procedures Used for Analytical Modeling

RAI 03.07.02-4 asked the applicant to provide the procedures used for analytical modeling per SRP Section 3.7.2, SRP Acceptance Criterion 3 guidance corresponding to the seismic analysis performed for site-specific Category I structures (including the UHS/RSW pump house and RSW piping tunnel). Specifically, the staff asked the applicant to provide the following information:

1. The criteria and procedures used to model the seismic system analyses [including structural material properties, modeling of member stiffness, modeling of mass (structural mass, live loads, floor loads, and equipment loads), modeling of damping, modeling of hydrodynamic effects, etc.]

2. The type of finite element model used; the effects of element mesh size, shape, and aspect ratio on solution accuracy; and the time steps used in the time history analysis, if applicable

3. The criteria and bases for determining whether a structure is analyzed as part of a structural system analysis or independently as a subsystem, decoupling the criteria for subsystems

4. The method used to address floor and wall flexibility in the structural modeling

5. The analytical models used for the dynamic analysis of the UHS/RSW pump house and the RSW piping tunnel

6. Special considerations such as wave passage effects, lateral earth pressures, and ground water effects for the RSW piping tunnel analysis

The staff's review of the response to **RAI 03.07.02-4 (ML092610377)** is discussed below:

1. In the response to Item 1 of **RAI 03.07.02-4**, the applicant provides additional details regarding the seismic analysis of the UHS/RSW pump house and includes this information in COL FSAR Subsection 3H.6.5.2.3. Material properties for the model's concrete elements are presented in COL FSAR Subsection 3H.6.4.4.1. The applicant also analyzes an additional case using cracked concrete properties, as discussed in the response to **RAI 03.07.02-8 (ML092370556)**. Other modeling details are described in the COL FSAR: mass of the structures and equipment is described in Subsection 3H.6.4.3.1.1, structural damping in Subsection 3H.6.5.1.2, and hydrodynamic water mass in Subsection 3H.6.5.2.3. The staff found this information to be acceptable because the analytical model incorporates the stiffness, mass, and

damping characteristics of the structural systems and includes an additional case using cracked concrete properties.

2. In the response to Item 2 of **RAI 03.07.02-4**, the applicant states that the finite element model of the UHS/RSW pump house is shown in Figure 3H.6-40. The response to Item 2 of **RAI 03.07.02-4** also states that *"The model mesh size is detailed enough to model the principal features of the structure and transmit a frequency of at least 33 Hz."* The staff found that the applicant's response to RAI 03.07.02-4 meets the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 4 and is therefore acceptable.
   RAI 03.07.02-16 asked the applicant to (a) provide the criteria and quantitative basis to show that the element sizes are sufficiently small to transmit frequencies of up to 33 Hz for the three soil cases; and (b) justify that the aspect ratio of the elements is such that it does not affect the accuracy of the solution. The applicant performed an adequate mesh sensitivity study. Details of the staff's review of the applicant's response to **RAI 03.07.02-16 (ML100550613)** regarding the adequacy of the structural and foundation mesh for seismic response analysis of the UHS/RSW pump house is discussed later in this section under the topic "Model Refinement and Passing Frequency."

3. In the response to Item 3 of **RAI 03.07.02-4**, the applicant states that the structural elements with significant stiffness or mass were included in the finite element model. Piping, electrical cable trays and conduits, grating, and HVAC duct work mass were included in the dead load, as discussed in COL FSAR Subsection 3H.6.4.3.1.1. Large equipment (e.g., pumps and fans) was included in the model as lumped masses. These items were analyzed as subsystems, as described in COL FSAR Sections 3.9 and 3.10. The staff found the information acceptable per SRP Section 3.7.2, SRP Acceptance Criterion 3.

4. In the response to Item 4 of **RAI 03.07.02-4**, the applicant states that the floor and wall flexibility was modeled. The floor slabs and walls were modeled using the finite element model. The adequacy of the refinement of the finite element mesh to capture the out-of-plane response of the floor slabs and walls is further discussed in this section under the topic "Model Refinement and Passing Frequency" in this SER.

5. In the response to Item 5 of **RAI 03.07.02-4**, the applicant states that the equivalent static analysis method was used to analyze the seismic response of the RSW piping tunnel, as described in COL FSAR Subsection 3H.6.5.3. The staff's review and evaluation of the seismic analysis of the RSW tunnel along with the analytical model is discussed under the topic "RSW Piping Tunnel" in Subsection 3.7.2.4.4, "Soil-Structure Interaction," of this SER. As explained later in Subsection 3.7.2.4.4, the staff's review concluded that the seismic analysis method along with the analytical model for determining the seismic demand for the RSW piping tunnel is acceptable.

6. In the response to Item 6 of **RAI 03.07.02-4**, the applicant states that dynamic soil pressures on the RSW piping tunnel walls were computed using the method described in ASCE 4–98, Subsection 3.5.3.2.2, as described in COL FSAR Subsection 3H.6.6.2.2. The staff's review and evaluation of seismic soil pressures on the RSW piping tunnel (including SSSI effects due to the proximity of heavy RB and RSW structures) is discussed under the topic "RSW Piping Tunnel" in Subsection 3.7.2.4.4, "Soil-Structure Interaction," of this SER. The applicant further states that strains created in the tunnel walls from the passage of seismic waves through the soil during a SSE were computed

using the method described in ASCE 4–98, Subsection 3.5.2.1. These tunnel strains from wave passage effects are reviewed in Subsection 3.8.4.4.2, Subpart B.1.4 (Item c) of this SER.

Concerning the procedures for analytically modeling the UHS/RSW pump house, the staff reviewed the following issues:

Model Refinement and Passing Frequency

The adequacy of the model mesh for the seismic response analysis of the UHS/RSW pump house was reviewed for structural mesh refinement and foundation soil mesh refinement, as discussed below:

a) *Structural Mesh Refinement*

In the response to **RAI 03.07.02-16 (ML100550613)**, the applicant uses two structural finite element models with different mesh sizes for the SSI and structural analyses. The SSI structural mesh is relatively coarse and the model was used to obtain maximum acceleration responses and the ISRS using SASSI2000. The structural mesh was refined and the model was used to perform a pseudo-static analysis with SAP2000 to calculate forces and moments in the structural members for design. Because the two structural meshes were used for different purposes (i.e., the dynamic model for the SSI analysis and the static model for the structural analysis), their adequacies were evaluated separately.

In the response to Item 1(a) of **RAI 03.07.02-16**, the applicant provides the results of a sensitivity study examining the adequacy of the structural mesh for the structural analysis with SAP2000. The sensitivity study was performed for two representative walls of the UHS/RSW pump house. They were analyzed as stand-alone panels with fixed boundary conditions subjected to uniform static in-plane and out-of-plane loads. Comparisons of the results in terms of the in-plane and out-of-plane forces and moments for two cases corresponding to the original mesh and new mesh—in which each element in the wall panel was divided into four elements—are also included in the response to Item 1(a). On the basis of comparable results from the two models, the applicant states that the mesh used for the design of the UHS/RSW pump house is acceptable. The staff reviewed the results of this sensitivity study and found no significant difference in the resulting membrane and out-of-plane forces and moments for two cases corresponding to the original mesh (used for the design) and the more refined mesh used in the sensitivity analysis. On this basis, the staff found the applicant’s conclusion to be acceptable.

With respect to the adequacy of the structural mesh size used in the SSI model as part of the response to Item 1(b) of **RAI 03.07.02-16,** the applicant performs another sensitivity study to compare the fixed-base modal frequencies and mass participation factors of the structural model used in the SSI analysis with those of a new structural model obtained by dividing each element in the SSI structural model into four elements. In the response to Item 1(b), the applicant presents the results of this sensitivity study in terms of modal frequencies, mass participation factors, and associated mode shapes for the major modes in the E-W and N-S directions. On the basis of comparable results from the two models, the applicant concludes that the structural model used in the SSI analysis is adequate. The staff reviewed the results of this sensitivity study and found that the applicant had presented comparisons for only the first two modes of the fixed-base structure in the E-W direction and the N-S direction. Although the comparisons of the modes from the two models are good, they only represent a mass participation factor of less than 20 percent for the E-W and N-S directions. In addition, the local modes are not reflected in these comparisons. Therefore, in **RAI 03.07.02-25** the staff requested a comparison of the higher modes (including the out-of-plane modes of the slabs and walls) to ensure the adequacy of the SSI mesh for transmitting frequencies of at least 33 Hz. The RAI also asked the applicant to include comparisons of the ISRS for the roofs, slabs, and wall panels of the fixed-base structures calculated using coarse SSI and fine design meshes subject to representative horizontal and vertical foundation motions. The staff needed this information to ensure that the coarse mesh size (for modeling the structure) used in the SSI analysis was adequate for evaluating the SSI effects.

To further assess the adequacy of the structural mesh size used in the UHS/RSW pump house SSI model, in the March 15, 2011, response to **RAI 03.07.02-25 (ML110770440),** the applicant compares the results of the fixed-base time history response analysis for the UHS/RSW pump house with three different mesh sizes:

1. Fixed-base time history analysis of the structural model used in the SSI model (called the original SSI model).

2. Fixed-base time history analysis of the structural model used in the subsequent refined SSI model (called the refined SSI model). The refined SSI analysis is described in the Supplement 2 response to **RAI 03.07.02-24 (ML103550646)**.

3. Fixed-base time history analysis of a more refined structural model (called the refined structural model). In this model, each element of the original SSI model was divided into four elements.

The staff reviewed the original SSI, refined SSI, and refined structural models and the details of their analyses. The staff noted that the mesh sizes for the three models were significantly different from one another, with the mesh refinement of the refined SSI model falling between the original SSI model and the refined structural model. The applicant analyzed all three models using the time history modal superposition method and included all modes up to a frequency of about 51 Hz.

The response to **RAI 03.07.02-25** compares the un-widened ISRS (for a 5 percent damping) from analyses of the three structural models described above, including a comparison of maximum accelerations from the original SSI model and the refined structural model analyses. On the basis of these comparisons, the staff found good agreement between the spectra and maximum accelerations generated from the original SSI model and the refined structural model in the x-, y-, and z-directions, with the exception of the following cases:

- Vertical excitation at the center of the pump house roof
- Vertical excitation at the center of the pump house operating floor
- Vertical excitation of the cooling tower walls
- Out-of-plane excitation of the UHS basin walls

The above results indicate that a more refined mesh for the pump house roof and operating floor, UHS basin walls, and cooling tower walls results in higher spectra and/or maximum accelerations. The increase in accelerations at the pump house roof and operating floor slabs, cooling tower walls, and UHS basin walls are discussed under the topic “Resolution of Items Observed in Various Sensitivity Studies” later in this section of the SER.

The comparison of the response spectra in the response to this RAI shows that the structural mesh of the refined SSI model used for the subsequent refined SSI analysis described in the December 14, 2010, Supplement 2 response to **RAI 03.07.02-24 (ML103550646)** converged because the changes between the results from the refined SSI model and the refined structural model are, in general, insignificant. In a few cases where the differences are more pronounced, results from the refined structural model were lower than the results from the refined SSI model, which signifies that further refinement is not necessary.

The staff found the applicant's structural mesh refinement study discussed above to be acceptable for demonstrating the adequacy of the analytical model in transmitting the frequency content of interest per SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines.

b) Foundation Soil Mesh Refinement

RAI 03.07.02-5 requested the applicant to provide the SSI analysis performed for the site-specific structures (including the UHS basin, UHS cooling tower enclosures, RSW pump house, and RSW piping tunnel) in accordance with SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines. More specifically, Item 1 of this RAI asked the applicant to provide a model of the structure and supporting soil (including backfill material) in sufficient detail for the staff to review. In the September 15, 2009, response to Item 1 of this RAI **(ML092610377)**, the applicant states that SASSI2000 was used to perform the SSI analysis of the UHS/RSW pump house and that this SASSI2000 analysis addresses the embedment of the structure, ground water effects, layering of the soil, and variations of strain-dependent soil properties. The applicant also states that the soil layer thicknesses used in the SSI model were small enough to transmit frequencies of up to 33 Hz for mean in situ soil properties. As described in COL FSAR Subsection 3H.6.5.2.4, an additional set of SSI analyses was performed to account for backfill placed adjacent to the walls. These analyses were performed by modeling the backfill as the soil horizon above the foundation level in the SASSI2000 model. The soil layer thicknesses used for the backfill were small enough to transmit frequencies of up to 33 Hz for mean backfill soil properties.

10 CFR Part 50, Appendix S states that the design of seismic Category I structures must take into account the SSI effects. To properly account for the SSI effects, the SSI model should be adequate for transmitting the maximum frequency content of interest, which for this site is the peak ground acceleration frequency (33 Hz) of the site-specific GMRS. Therefore, Item 1 of **RAI 03.07.02-17** requested the applicant to provide the criteria for soil and backfill layer thicknesses, as shown in FSAR Tables 3H.6-1 and 3H.6-2, that is small enough to transmit frequencies of up to 33 Hz for the SSI analysis using SASSI2000. The RAI also requested the applicant to justify not using LB soil/backfill properties to determine the soil layer thicknesses capable of transmitting frequencies of up to 33 Hz. In the February 4, 2010, response to **RAI 03.07.02-17 (ML100480204)**, the applicant states that the layer thicknesses used in the SSI model for both in situ and backfill soils were modified to be sufficiently small to conservatively transmit frequencies of up to 33 Hz for the corresponding mean soil properties. In the response to Item 1 of **RAI 03.07.02-17**, the applicant provides new Tables 3H.6-1a, b, and c for the in situ soils as well as new Tables 3H.6-2a, b, and c for the backfill soils showing the actual layer thicknesses used in the SSI model, along with strain-compatible soil properties and passing frequency values for all three soil cases (i.e., mean, UB, and LB, respectively).

The applicant's response to Item 1 of **RAI 03.07.02-17** also provides the following shear wave length criteria for determining the maximum soil layer thicknesses in the SASSI2000 SSI model that are capable of adequately transmitting the highest frequency of interest for the mean soil case.

$$H = V_s / (5 \times F_{t\text{-}s})$$

Where: V_s is the shear wave velocity and $F_{t\text{-}s}$ is the transmittal frequency

The staff found the selection of maximum layer thicknesses based on shear wave length criteria to be acceptable for ensuring that a correct variation of ground motion with depth is calculated for the site response solution in the SASSI2000 finite element model. However, for the LB soil case, the highest transmitted frequency for the calculated in situ and backfill materials using these criteria is about 26 Hz. The applicant further stated that this lower cutoff frequency (26 Hz) is justified in light of the recommendation of ASCE 4–98, Subsection 3.3.3.5. However, the NRC has not endorsed the ASCE-98 criteria for selecting the cutoff frequency for the SSI analysis. For the impedance solution aspect of the SASSI2000 SSI model, the maximum horizontal dimension of the excavated soil elements should also satisfy the above shear wave length criteria, where H is the maximum horizontal dimension of the soil elements. Therefore, **RAI 03.07.02-26** requested the applicant to provide a quantitative assessment for the LB soil case demonstrating that the results using the existing soil mesh size will be conservative when compared to the results using a more refined soil mesh size (thus meeting the criteria stated above for both element thickness and horizontal element dimension) capable of transmitting a frequency of 33 Hz. The staff needed this information to ensure that the use of the existing soil mesh size in the SSI model adequately accounts for SSE frequencies of interest in the evaluation of SSI effects.

The applicant performed two additional UHS/RSW pump house SSI analyses with the UB soil case, considering both full and empty UHS basins and with a refined model to address the soil mesh refinement issues. Details of the sensitivity analyses are described in the Supplement 2 response to **RAI 03.07.02-24** dated December 14, 2010 (ML103550646). This response also includes the original SSI model and the refined SSI model. A cutoff frequency of 33 Hz was used in these analyses for the transfer function calculation. For soil layers below ground water level, Poisson’s ratio was capped at 0.495 for determining the compression wave velocity. In the original SSI model, Poisson’s ratio was capped at 0.48.

The soil mesh refinement needed to meet the passing frequency requirement caused the model to exceed the capability of SASSI2000 Version 3, which has been used for all of the SSI analyses. To analyze this larger model, the applicant modified SASSI2000 to allow the handling of larger file sizes and to reduce the runtime by using a more efficient solver. The staff reviewed the validation of the modified SASSI2000 software. See Subsection 3.7.2.4.16, “Computer Programs Verification and Validation Issues,” of this SER for details of the staff’s review. The applicant used a passing frequency of about 24 Hz for the SSI analysis, because energy contents of site-specific GMRS motions above 24 Hz are insignificant.

To further assess the adequacy of the passing frequency, the refined SSI model for the full basin case was also analyzed with a cutoff frequency of 23.5 Hz instead of 33 Hz. The un-widened 5 percent damped ISRS from this analysis were compared with those from the refined SSI analysis, which used a 33 Hz cutoff frequency. A comparison of the y- and z-directions was included in the December 14, 2010, response to RAI 03.07.02-24. The comparisons also show the corresponding spectra from the original SSI analysis, which have a passing frequency of 15.6 Hz and a cutoff frequency of 25.4 Hz. On the basis of the above results, the applicant states the following:

- Spectra from the original SSI model analysis with a 15.6 Hz passing frequency were compared with spectra from the refined SSI model analysis with a 23.5 Hz passing frequency, and the comparison showed that the responses in the original model were adequately captured for frequencies significantly beyond 15.6 Hz.

- Note that the increase in the spectra from the refined SSI analysis at the four locations is mainly due to structural mesh refinement rather than to the SSI model refinement for the passing frequency. These explanations indicate that a passing frequency of 23.5 Hz is adequate.

The staff found the soil mesh refinement sensitivity study as discussed above to be acceptable and in accordance with SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines. The increase in seismic demand as a result of the sensitivity study is addressed under the topic "Resolution of Items Observed in Various Sensitivity Studies" later in this section of the SER.

c) *Impact of the Empty Basin*

The original SSI analysis of the UHS/RSW pump house considered a full UHS basin. The applicant expanded the SSI analysis of the UHS/RSW pump house to also consider an empty UHS basin, as described below.

Following a loss of coolant accident, the applicant indicates that the UHS/RSW pump house will need to perform its safety-related cooling function for a period of 30 days without any makeup water. During this period, the UHS basin water inventory will decrease and the water height within the basin may get as low as 0.9 m (3 ft) above the top of the UHS basin foundation, which is nearly an empty UHS basin.

In order to assess the impact of the reduced UHS basin water inventory on the SSI analysis, the applicant modified the original SSI model by removing the hydrodynamic mass. The applicant then analyzed this SSI model for six cases (i.e., in situ LB, mean, and UB in situ soil cases; UB backfill soil case; mean in situ soil case with separation in the top 6.1 m (20 ft); and mean in situ soil case with cracked concrete). The Supplement 2 response to **RAI 03.07.02-24 (ML103550646)** compares the resulting response spectra for full and empty UHS basins for selected nodes of the UHS/RSW pump house. These results show a significant impact on the resulting ISRS and seismic forces. Based on the results of the empty UHS basin, the applicant stated that modifications in the existing ISRS of the UHS/RSW pump house were warranted. As discussed below under the topic "Resolution of Items Observed in Various Sensitivity Studies" in this section of the SER, the applicant used conservative scaling factors to adjust the results of the coarse SSI mesh upward to account for the effects of both the structural and foundation mesh discretization, on the basis of sensitivity analyses that included both empty and full basin cases.

d) *Resolution of Items Observed in Various Sensitivity Studies*

In the Supplement 2 response to **RAI 03.07.02-24**, the applicant modifies the existing ISRS to address (a) local increases in the response spectra as a result of structural mesh refinement and SSI model refinement; and (b) the impact of the empty basin. The applicant states that the following procedure was used to conservatively account for these increases in the design:

Step 1: Comparisons of the envelopes of the ISRS ratios from the structural mesh sensitivity study (described in the response to **RAI 03.07.02-25 [ML110770440]**) and comparisons of the spectra from the SSI model refinement (described in the Supplement 2 response to **RAI 03.07.02-24**) for both empty and full basins were used to determine the enveloped modification factors for the following ISRS:

- Vertical response spectra at the center of the pump house roof
- Vertical response spectra at the center of the pump house operating floor
- Vertical response spectra for the cooling tower walls
- Out-of-plane response spectra for the basin walls

The resulting modification factors for the above response spectra are shown in the Supplement 2 response to **RAI 03.07.02-24**.

Step 2: Adjust the applicable spectra in the four locations listed above by multiplying them by the modification factors determined in Step 1. This step is performed for the results from each of the eight analysis cases for the full basin and each of the six analysis cases for the empty basin using the original SSI mesh.

Step 3: For all locations and all x-, y-, and z-directions, determine the enveloped un-widened response spectra from all eight cases of the full basin SSI analysis using the original SSI mesh and the UB refined SSI mesh full basin analyses.

Step 4: For all locations and all x-, y-, and z-directions, determine the enveloped un-widened response spectra from all six cases of the empty basin SSI analysis using the original SSI mesh and the UB refined SSI mesh empty basin analyses.

Step 5: For all locations and all x-, y-, and z-directions, determine the envelope of the spectra from Steps 3 and 4.

Step 6: Increase the results of Step 5 to include an interpolation between the results of Step 3 and Step 4 to account for a potential frequency shift between the empty and full basin cases. The result is a single data set that envelops the empty and full basin analytical sets and includes the interpolated values between the empty and full basin envelopes. The results of this step are visible in the Supplement 2 response to **RAI 03.07.02-24**.

Step 7: Widen the spectra produced in Step 6 by ±15 percent on the frequency scale. The results are visible in the Supplement 2 response to **RAI 03.07.02-24**.

Step 8: To produce the final ISRS for the UHS/RSW pump house, fill in all valleys of the spectra obtained in Step 7 to remove local minima. The results are visible in the Supplement 2 response to **RAI 03.07.02-24**.

The revised ISRS per the above criteria are shown in COL FSAR Figures 3H.6-16 through 3H.6-39. The staff reviewed the above procedure and found it to be conservative based on the following:

- The applicant used conservative scaling factors to adjust the results of the coarse SSI mesh upward to account for the effects of both the structural and foundation mesh discretization, on the basis of sensitivity analyses that included both empty and full basin cases.

- The resulting spectra were widened by ±15 percent on the frequency scale to cover any frequency shifts.

- All the valleys in the spectra were filled to remove local minima.

To further confirm the results of these sensitivity studies, the staff performed a confirmatory SSI analysis of the UHS/RSW pump house using the refined SSI model, as discussed in Subsection 3.7.2.4.17, “Confirmatory SSI Analysis,” of this SER. On the basis of the results of the confirmatory SSI analysis and the conservative procedure the applicant used to account for the model discretization, the staff found the results of spectra generation for the UHS/RSW pump house acceptable.

In the Supplement 2 response to **RAI 03.07.02-24 (ML103550646)**, the applicant states that the design of the UHS/RSW pump house envelops the maximum accelerations from the original and refined SSI analyses for both full and empty UHS basin cases. The Supplement 1 response to **RAI 03.08.04-30 (ML110770440)** provides the criteria for addressing any increases in the maximum accelerations resulting from (a) the structural mesh refinement and SSI mesh refinement; and (b) the impact of an empty basin on the structural design of the UHS/RSW pump house. This information is reviewed in Subsection 3.8.4.4.2, Subpart B.1.4 of this SER. A discussion of the resolution of the impact from the use of the SM on the results of the SSI analysis is in Subsection 3.7.2.4.20, which is located later in this SER.

Hydrodynamic Effects of Water in the UHS Basin

Item 13 of Part 1 of **RAI 03.07.02-15** requested the applicant to provide the modeling details on how the hydrodynamic effects of water in the UHS basin were calculated and applied in the seismic analysis of the UHS basin.

In the response to Item 13 of Part 1 of **RAI 03.07.02-15** dated February 10, 2010 **(ML100550613)**, the applicant states that hydrodynamic effects on the UHS basin walls were determined in accordance with ASCE 4–98 Subsection 3.1.6.3, “Building Model Hydrodynamic Mass Effects.” According to ASCE 4–98 Section 3.1.6.3, the fluids contained in the basin of a structure shall be modeled to represent both impulsive and convective (sloshing) effects. ASCE 4–98 Section 3.1.6.3 also states that for water depths greater than 15.2 m (50 ft), hydrodynamic effects resulting from the compressibility of water shall be considered in the dynamic analysis.

To account for the impulsive effect of water in the UHS basin from the horizontal input, the applicant calculated the corresponding impulsive mass and distributed it as lumped added mass on the basin walls and buttresses in the SSI model. Because the impulsive force of water acts on both sides of the buttress, the amount of pressure assigned was twice the portion of the basin walls shielded by the buttresses. For the vertical input, the impulsive mass was lumped to the basin basemat in the SSI model. According to Subsection 3.1.6.3(d) of ASCE 4–98, for water depths less than 15.2 m (50 ft) the entire water mass may be lumped at the foundation mat of the basin. For water depths greater than 15.2 m (50 ft), the effects from the compressibility of water shall be determined on the basis of engineering mechanics principles. The water depth (h) in the UHS basin is 21.6 m (71 ft). Based on a compression velocity of water (V_c) equal to 1,463 m/s (4,800 ft/s), the applicant calculated the vertical frequency of the water column to be $f_v = V_c / 4h = 17$ Hz. The applicant stated that because the predominant SSI frequencies were below 17 Hz, the vertical water mass was lumped at the foundation basemat.

At the NRC audit of STP Units 3 and 4 during the week of October 18 through October 22, 2010, the staff reviewed the hydrodynamic mass calculations for the UHS basin and observed that the amplification of the hydrodynamic pressures on the UHS basin basemat and walls resulting from the frequency of the water column had not been considered for the vertical input motion. As a result, during the audit the staff requested the applicant to address this issue in **RAI 03.07.02-28.**

In the March 15, 2011, response to **RAI 03.07.02-28 (ML110770440)** to account for the hydrodynamic pressure on the UHS basin walls due to vertical excitation, the applicant indicates that the wall pressures were scaled upward by a factor of (1+ α), where α is equal to the maximum spectral value obtained from the 5 percent damped acceleration response spectra of the base motion. The staff compared this value against a more conservative spectral value obtained from the 3 percent damped spectra at the fundamental frequency of the water column in the basin and found the two values to be very close. Based on this assessment, the staff found the hydrodynamic pressures on the UHS basin walls due to vertical excitation to be acceptable. Subsequently, at the NRC audit of STP Units 3 and 4 during the week of February 27 through March 3, 2012, the staff requested the applicant to address any potential impact of the DNFSB issue on UHS basin hydrodynamic wall pressure from the vertical excitation effect. In the Supplement 3 response to RAI 03.07.01-29 **(ML12103A369)**, the applicant compares the acceleration response spectra that were computed using both the SM and MSM at the UHS basin basemat in Figure 03.07.01-29 S3.1 of the response. This figure reveals that the impact of the MSM on the vertical acceleration response spectra of the UHS basin basemat is negligible. Therefore, this issue is considered resolved. A discussion of the resolution to the impact from the use of the SM on the results of the SSI analysis is in Subsection 3.7.2.4.20, which is located later in this SER.

In performing the confirmatory SSI analysis of the UHS/RSW pump house, as described in Subsection 3.7.2.4.17, “Confirmatory SSI Analysis,” in this SER, the staff noted that the SSI model did not assign hydrodynamic masses to the submerged columns inside the UHS basin. Because of the relatively large surface area of these columns, their responses and design could be affected by the omission of hydrodynamic effects. As such, Item 1 of **RAI 03.07.02-28** asked the applicant to justify not assigning hydrodynamic masses to the submerged columns inside the UHS basin, which could impact the calculated member forces and stresses in the columns.

In the response to Item 1 of **RAI 03.07.02-28**, the applicant states that the 3-D finite element analytical model used for the UHS/RSW pump house design was revised to include seismic loads resulting from an effective hydrodynamic mass on the UHS basin columns for the calculation of member forces. The response includes details of this model.

The applicant also investigated the hydrodynamic mass effect by performing a local analysis of the columns with added mass incorporating various boundary conditions at the top and bottom of the columns, as discussed in the Supplement 4 response to **RAI 03.08.04-30 (ML11181A002)**. Column acceleration scale factors reported in this response indicate an increase in the column stresses. In the Supplement 6 response to **RAI 03.08.04-30 (ML11259A056)**, the applicant indicates that the results of the hydrodynamic mass effect on the UHS basin columns are incorporated into the design. Details of the above analyses and how the effects were considered in the design are reviewed in the response to **RAI 03.08.04-30** in Subsection 3.8.4.4.2, Subpart B.1.4 of this SER.

The applicant also calculated the convective added mass in the global x- and y-directions representing fluid oscillation in the basin (sloshing effect), in accordance with ASCE 4–98, Subsection 3.1.6.3(c). The applicant incorporated the mass into the SSI model. This mass was distributed from the top of the water surface to the center of the equivalent oscillating mass. The staff reviewed the modeling procedure the applicant had used to distribute the convective mass in the SSI model. Based on this review, the staff found the above procedure conservative and in accordance with standard engineering practices. The procedure is thus acceptable.

In conclusion, the applicant has adequately evaluated the hydrodynamic effects of water in establishing the seismic demand of the UHS basin.

Strain-Compatible Soil and Backfill Properties

Item 3 of **RAI 03.07.02-5** asked the applicant to describe the procedure for addressing strain-dependent soil and backfill properties in the SSI analysis of the site-specific structures. In the response to Item 3 of **RAI 03.07.02-5 (ML092610377)**, the applicant states that the strain-compatible soil shear wave velocity and damping for the SSI model were obtained from the same ground response analysis that was used to develop the GMRS, as described in COL FSAR Section 2.5S.2. Three sets of in situ soil properties were used—LB, mean, and UB properties—as COL FSAR Table 3H.6-1 shows. Similarly, three sets of backfill soil properties were used—LB, mean, and UB properties—as COL FSAR Table 3H.6-2 shows. In reviewing this information, the staff did not find sufficient details on how the strain-compatible backfill properties were obtained. Therefore, to further clarify this information, in Item 3 of **RAI 03.07.02-17** the staff requested a detailed description of the development of the LB, mean, and UB strain-compatible properties for the backfill soil from the results of the probabilistic site response analysis, as COL FSAR Section 2.5S.2 describes.

In the response to Item 3 of **RAI 03.07.02-17** dated February 4, 2010 **(ML100480204),** the applicant states that a two-step procedure was used to estimate the average strain-compatible dynamic properties for the backfill. The SSE-compatible backfill shear strains were assumed to be comparable to those of the surrounding soils. These were then averaged over the entire depth of the backfill and were used to estimate the backfill shear modulus and damping based on the standard G/G_{max} and damping versus the effective shear strain for granular soils having an 85 percent relative density, which is published in the literature. To calculate the average UB value,the resulting mean value was multiplied by 1.5. To calculate the average LB value, the resulting mean value was divided by 1.5. This is consistent with the guidelines for SSI analysis described in the Acceptance Criteria of SRP Section 3.7.2. This response clarified that the strain-compatible properties for the backfill soil were not developed from the probabilistic site response analysis. Because the extent of the backfill compared to the surrounding soil is generally small and can be considered part of the SSI rather than the site response model, the staff found the applicant's approximate procedure described above acceptable for estimating the LB, mean, and UB strain-compatible backfill properties for the SSI analysis of the site-specific structures.

Ground Water Effects

Because of the high ground water table at the STP 3 and 4 site, Item 3 of **RAI 03.07.01-4** and Item 2 of follow-up **RAI 03.07.01-17** asked the applicant to describe how the ground water effects were modeled in the SSI analysis. In the December 17, 2009, response to Item 2 of **RAI 03.07.01-17 (ML100480204)**, the applicant states that ground water effects were included in the SSI analysis by setting the compression wave velocity, V_p, of saturated soils to 1,524 m/s (5,000 ft/s) except when Poisson's ratio, v, calculated from Equation (1) below, is higher than 0.48. In those cases, Poisson's ratio is set to the maximum value of 0.48 and the compression wave velocity is recalculated using Equation (2). In these equations, V_s is the shear wave velocity of the soil layers:

$$v = 0.5\ [(5{,}000\ /\ V_s)^2 - 2]\ /\ [(5{,}000\ /\ V_s)^2 - 1] \qquad (1)$$
$$V_p = V_s\ [(2 - 2v)\ /\ (1 - 2v)]^{0.5} \qquad (2)$$

Arbitrarily capping the Poisson's ratio at 0.48 for saturated soils may result in a calculated compression wave velocity lower than 1,524 m/s (5,000 ft/s), when the shear wave velocity drops below approximately 298 m/s (980 ft/s). For example, in the response to **RAI 03.07.02-17** Tables 3H.6-1a through 3H.6-2c show that approximately 22, 17, and 73 m (75, 57, and 240 ft) of the respective soil columns of the in situ mean, UB, and LB soil cases have calculated-compression wave velocities of less than 1,524 m/s (5,000 ft/s). The use of compression wave velocities in saturated soils of less than 1,524 m/s (5,000 ft/s) will not allow the higher frequency components of vertical motion to be transmitted into the structure and may result in a less conservative response. As such, Item 3 of **RAI 03.07.01-25** asked the applicant to assess the impact from using compression wave velocities lower than 1,524 m/s (5,000 ft/s) in saturated soils on the response of structures (including the ISRS) by performing a sensitivity study and comparing the results for two cases:

- Case 1, with Poisson's ratio capped at 0.48 for saturated soils and the compression wave velocity allowed to drop below 1,524 m/s (5,000 ft/s) (similar to the procedure used by the applicant).

- Case 2, with the compression wave velocity set to 1,524 m/s (5,000 ft/s) in saturated soils and the Poisson's ratio allowed to rise above 0.48, depending on the in situ shear wave velocities.

To address this issue, the applicant performed a sensitivity study of the CB as part of an evaluation of the soil parameter's departure from the ABWR DCD using a higher Poisson's ratio of 0.495 and both LB and UB soil profiles. In Item 3 of the Supplement 1 response to **RAI 03.07.01-25** dated November 29, 2010 **(ML103360074)**, the applicant compares the seismic responses from two analyses with Poisson's ratio capped at 0.48 and 0.495. Based on these comparisons, the applicant states that the results obtained from the analysis with Poisson's ratio capped at 0.495 were, in general, close to the corresponding enveloped responses obtained from the analysis with Poisson's ratio capped at 0.48, with the exception of some responses in the vertical direction, especially vertical responses of the floor slabs. The applicant further states that for these exceedances, the following considerations apply:

- For the CB and RB, where the original site-specific SSI analyses used 0.48 as the Poisson's ratio cutoff (as described in COL FSAR Appendix 3A), the ABWR DCD responses were higher than the site-specific responses. In terms of the ISRS, even the modified responses with 0.495 as the Poisson's ratio cutoff showed similar margins when compared to the ABWR DCD responses. Therefore, the increases in vertical responses shown in this sensitivity study (discussed above) were not significant to the conclusion that the ABWR DCD responses significantly envelop the site-specific responses for the CB and RB.

- A Poisson's ratio of 0.495 was used for the new SSI analyses of the site-specific structures. Therefore, the conclusions derived from the new SSI analyses include the effects of a higher Poisson's ratio cutoff.

The staff reviewed the results of the Poisson's ratio sensitivity study provided in the RAI response and found that for the LB soil profile, there were significant differences in the transfer functions and vertical responses obtained from the two analyses. The results also indicated that for the case with higher Poisson's ratio, the transfer functions showed a number of narrow peaks (spikes) that did not exist in the original analysis. The applicant did not provide any explanation for the differences in these transfer functions. However, the staff believes that these spikes are likely caused by the use of the SM. A detailed evaluation and discussion of this issue is in Subsection 3.7.2.4.20, "DNFSB SASSI Subtraction Method Issues".

In view of the results of this sensitivity study, the applicant decided to reanalyze the UHS/RSW pump house and analyze all the remaining site-specific Category I structures using a higher Poisson's ratio value of 0.495. However, for the UHS/RSW pump house, the applicant performed the reanalysis with the higher Poisson's ratio for only the UB soil case. The applicant's verification and validation (V&V) benchmark problems for SASSI2000 also did not establish a cutoff limit for the Poisson's ratio. In the absence of a more extensive sensitivity study of the limiting value of Poisson's ratio for the STP site and an analysis of the UHS/RSW pump house for the LB soil case with a higher Poisson ratio, the staff did not have enough confidence in the conservatism of the final seismic design basis results; not without further evaluation. To gain greater confidence and assurance in the results of the SSI analysis for the site-specific Category I structures, the staff performed a confirmatory SSI analysis to verify the acceptability of the results of the STP SSI analysis for the UHS/RSW pump house. A discussion of the results of this confirmatory analysis is in Subsection 3.7.2.4.18, "Poisson's Ratio Confirmatory Analysis," which is located later in this SER.

Based on the above review and the staff's "Poisson's Ratio Confirmatory Analysis," the staff found the use of a Poisson's ratio limiting value of 0.495 acceptable in the STP Units 3 and 4 applications for modeling the ground water effects.

Ground Water Elevation

Based on Section 2.4S.12 and Table 2.0-2, the ground water elevation at the STP Units 3 and 4 site is reported at 8.5 m (28.0 ft) MSL, while the analyses have assumed a ground water elevation of 7.7 m (25.5 ft) MSL. During the staff's audit of STP Units 3 and 4 conducted May 23 through May 27, 2011, the staff reviewed relevant calculation documents to assess the effect of a change in the ground water elevation from 7.7 to 8.5 m (25.5 to 28 ft) on the GMRS and FIRS. The staff's review concluded that a 0.7 m (2.5 ft) rise in the ground water level results in a decrease in the confining pressure of about 7.47 kPa (156 pounds per square foot [psf]), which is small compared to the range of soil pressures (172–689 kPa [3,600–14,400 psf]) used to develop the strain-dependent soil shear modulus and damping ratio relationships used in the ground response analysis. In addition, the effect on the low-strain shear wave velocity is also small, which renders the effect on the GMRS and FIRS calculation negligible.

To evaluate the impact of a 0.7-m (2.5-ft) increase in the ground water level on the results of the SSI analysis, the applicant performed a sensitivity analysis of the DGFOSV in which the Poisson's ratio of soil layers was adjusted to reflect the rise in ground water. The analysis was performed for bounding in situ LB and backfill UB soil cases. The results showed a small increase in the calculated ISRS at frequencies above 8 Hz resulting from the effect of the 0.7 m (2.5-ft) rise in the ground water level. The staff asked the applicant to further assess this issue by repeating the above analysis using the MSM with the ground water level set at 8.5 m (28 ft) (see the Supplement 1 response to RAI 03.07.01-29 **ML113250374**). As discussed in COL FSAR Subsection 3H.6.5.2.4.3, the SSI analysis was repeated using the MSM to address the

DNFSB SASSI issue. Also, in these analyses the ground water table was changed to 8.5 m (28 ft) MSL. Based on a comparison of the resulting response spectra from these analyses with those from the SM analysis, additional factors were determined for increasing the ISRS obtained from the original SM analysis. The staff found the applicant's resolution of inconsistencies in the ground water level assumed in the original analysis acceptable, because the revised analysis used the correct ground water table elevation. The results of the MSM analyses are further reviewed as part of the SASSI SM issues in Subsection 3.7.2.4.20, which is located later in this SER.

Modeling of UHS Basin Column Connections to the Basemat

A review of the applicant's SSI model of the UHS/RSW pump house in the response to RAI 03.07.02-21 (ML100890620) for the staff's confirmatory analysis, as well as the description of the SSI model in the response to **RAI 03.07.02-15 (ML100550613)** revealed that the columns inside the UHS basin are rigidly connected to the UHS basin basemat. To provide the moment transfer at the column/basemat connections, all of the columns were extended into the solid elements with rigid massless beams, with the exception of two columns located at the UD/U4 and UD/U8 line intersections. These two columns have pin connections at the basemat, causing higher accelerations at the top of the columns compared to the rest of the columns. As such, Item 2 of **RAI 03.07.02-28** asked the applicant to clarify whether these pinned connections at the base of the columns at UD/U4 and UD/U8 are consistent with the intent of the UHS basin design.

In the response to Item 2 of **RAI 03.07.02-28** dated March 15, 2011 **(ML110770440)**, the applicant states that all UHS basin columns are intended to have fixed connections at the basemat. The applicant adds that pinned connections at the base of columns UD/U4 and UD/U8 are only present in the original SSI model, while in the 3-D finite element analysis design model and the refined SSI model these connections at the basemat are fixed. Furthermore, the presence of pinned connections for the two columns in the original SSI model should not significantly affect the results of the SSI analysis because the UHS basin columns stiffness contribution to the total stiffness of the structure is small. In addition, as described in the December 14, 2010, response to the Supplement 2 response to **RAI 03.07.02-24 (ML103550646)**, the design of the UHS/RSW pump house and the ISRS are based on the enveloped results from SSI analyses with both the original SSI model and the refined SSI model. In the refined SSI model, fixed connections at the basemat are used for these two columns.

Based on the results of the staff's confirmatory SSI analysis of the UHS/RSW pump house as discussed in Subsection 3.7.2.4.17, "Confirmatory SSI Analysis," and the fact that the applicant enveloped the results of the original and refined SSI models as described above, the staff found the applicant's conclusion acceptable that the presence of pinned connections for the two columns in the original SSI model should not significantly affect the results of the SSI analysis.

In conclusion, based on the above evaluation and the staff's review (as part of staff's confirmatory SSI analysis) of the STP Units 3 and 4 SSI model for the UHS/RSW pump house, the staff concluded that the procedures used for analytical modeling are acceptable.

3.7.2.4.4 Soil-Structure Interaction

Amplified Ground Motion Input to the SSI Analysis

The input motion for the SSI model for the UHS/RSW pump house was applied to the free-field ground surface. The input motion consisted of the site-specific SSE design time histories for three orthogonal directions. The staff found this approach for specifying the input motion to the UHS/RSW pump house SSI model consistent with ABWR DCD Section 3A.6. Because the UHS/RSW pump house is a heavy structure, the staff found this approach acceptable because the input motion is not expected to be significantly affected by the nearby lighter structures. The staff's review of the site-specific SSE design time histories is documented in Subsection 3.7.1.4.2 of this SER.

However, for lighter structures such as the RSW piping tunnel, the DGFOSV, and the DGFOT, the seismic input could be amplified from the presence of nearby heavy structures such as the RB and the UHS/RSW pump house. In the March14 through 18 (2011) audit, the staff identified insufficient details on how the input motion used in the 3-D SSI model of the RSW piping tunnel, DGFOSV, and DGFOT considered the effects of the SSSI from the nearby heavy structures. The staff asked the applicant to include this information in the COL FSAR. The applicant addressed this concern in the Supplement 3 response to **RAI 03.07.01-27 (ML11143A054)**. The staff's review of this response is as follows:

a) *Reactor Service Water (RSW) Piping Tunnel*

To account for the amplification of input motion from the nearby heavy RB and UHS/RSW pump house structures, the applicant states that the following procedure was used:

- In the 3-D SSI analysis of the RB for a site-specific SSE, one interaction node at the ground surface and one interaction node at the depth corresponding to the bottom elevation of the RSW piping tunnel were placed at six locations along the centerline of the RSW piping tunnel.

- In the 3-D SSI analysis of the UHS/RSW pump house for a site-specific SSE, one interaction node at the ground surface and one interaction node at the depth corresponding to the bottom elevation of the RSW piping tunnel were placed at one location on the centerline of the RSW piping tunnel.

- From the above SSI analyses of the RB and the UHS/RSW pump house, the resulting amplified response spectra at the interaction nodes representing the response of the RSW piping tunnel were obtained. In order to find a reasonable envelope for these response spectra to use in the SSI analysis of the RSW piping tunnel, the spectra were compared to 1.15 x site-specific SSE to identify those that exceed 1.15 x site-specific SSE. New COL FSAR Figures 3H.6-209a through 3H.6-209d include the response spectra that exceed 1.15 x site-specific SSE.

- Based on the comparisons of response spectra shown in Figures 3H.6-209a through 3H.6-209d, six motions were selected as enveloping amplified motions for the SSI analysis. These six motions correspond to 1.15 x site-specific SSE and amplified motion time histories for Nodes 29378, 29379, 29390, 29392, and 15129.

- Using the 1.15 x site-specific SSE input and acceleration time histories for the five nodes (noted above) obtained from the RB and UHS/RSW pump house SSI analyses for each soil case, a SSI analysis was performed for the RSW piping tunnel for the corresponding soil cases. The response spectra and maximum accelerations from these SSI analyses were enveloped to produce final response spectra and maximum accelerations for the design.

The applicant revised COL FSAR Subsection 3H.6.5.3 and provided Figures 3H.6-209a through 3H.6-209d to show the input motions used for the SSI analysis of the RSW piping tunnel. The staff reviewed the information and results and found the above method acceptable, because it incorporates the effects of the SSSI due to nearby heavy structures into the input for 3-D SSI analyses of the RSW piping tunnel.

b) *Diesel Generator Fuel Oil Storage Vaults*

The applicant states that five interaction nodes at the ground surface and five interaction nodes at the depth corresponding to the bottom elevation of the DGFOSV foundations were added to the 3-D SSI model of the RB to obtain free-field responses for three DGFOSVs. These five nodes correspond to the four corners and the center of the DGFOSV. The RB SSI model was analyzed for the STP Units 3 and 4 site-specific SSE. For each of the three DGFOSVs, the spectra at the surface-level nodes and the spectra at the five foundation-level nodes were each calculated and averaged. Then the envelope of the two averages was calculated. In the SSI analysis of the UHS/RSW pump house, interaction nodes were added to the model and amplified motion was obtained for the DGFOSV close to the UHS/RSW pump house. The applicant further states that, because the diesel oil tank is a standard plant design equipment, the input motion for the SSI analysis also considered the 0.3g RG 1.60 response spectra. Therefore, the envelope of the 0.3g RG 1.60 response spectra and the enveloped average spectra for the three DGFOSVs were used as the input response spectra for the SSI analysis of the DGFOSV. As a result of this response, the applicant revised COL FSAR Section 3H.6.7 and provided comparisons of the spectra in COL FSAR Figures 3H.6-222a through 3H.6-222c, which show that the 0.3g RG 1.60 response spectra envelop all other amplified ground motion spectra. The staff reviewed the above information and results and considered the method used to incorporate the SSSI effects from the nearby heavy structures into the input for the 3-D SSI analyses of the DGFOSVs. The staff found the method conservative and therefore acceptable.

c) *Diesel Generator Fuel Oil Tunnels*

The applicant states that in the 3-D SSI analysis of the RB for the site-specific SSE, one interaction node at the ground surface and one interaction node at the depth corresponding to the bottom elevation of the DGFOT were placed at several locations along each of the three DGFOTs. The envelope of amplified motions at these interaction nodes was further enveloped with the 0.3g RG 1.60 response spectra and the results were used for the SSI analysis of the DGFOT. As shown in COL FSAR Figures 3H.7-30a through 3H.7-30c, the 0.3g RG 1.60 response spectra were found to be the bounding spectra. As a result of this response, the applicant revised COL FSAR Subsection 3H.7.5.2.1 and provided Figures 3H.7-30a through 3H.7-30c to show the input motions used for the SSI analysis of the DGFOT. The staff reviewed the above information and results and considered the method used to incorporate SSSI effects from the nearby heavy structures into the input for the 3-D SSI analyses of the DGFOTs to be conservative and therefore acceptable.

Seismic Analysis of UHS/RSW Pump House and RSW Piping Tunnel

RAI 03.07.02-5 asked the applicant to provide in COL FSAR Subsection 3H6.5.2.4 the SSI analysis performed for the site-specific structures (including the UHS/RSW pump house and the RSW piping tunnel) in accordance with SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines, in sufficient detail for the staff to review. More specifically, the RAI requested the following information:

1. Model of the structure and supporting soil, including backfill material
2. Model boundaries and the location of input ground motion
3. Procedure for addressing strain-dependent soil and backfill properties in the SSI analysis
4. Method of accounting for the effects of potential variability in the soil and backfill properties at the site
5. Potential effect of side soil-wall separation during a seismic event
6. Methods of the SSI analysis (e.g., time domain and/or frequency domain analysis, consideration of soil and structural damping, etc.) and results in the form of enveloped seismic responses (including the ISRS) at key locations in the site-specific structures

In the response to Item 1 of **RAI 03.07.02-5** dated September 15, 2009 **(ML092610377)**, the applicant states that for the UHS/RSW pump house, SSI effects were accounted for by using SASSI2000 in conjunction with time histories described in COL FSAR Subsection 3H.6.5.1.1.2. The structural model is shown in Figure 3H.6-15 and described in COL FSAR Subsection 3H.6.5.2.3. The applicant adds that the soil layer thicknesses used in the SSI model were sufficiently small enough to transmit frequencies of up to 33 Hz for the mean in situ soil case. To account for the backfill placed adjacent to the walls, an additional set of SSI analyses was performed by modeling the backfill as the soil horizon above the foundation level in the SSI model, as described in COL FSAR Subsection 3H.6.5.2.4. In addition, the soil layer thicknesses used for the backfill were sufficiently small to transmit frequencies of up to 33 Hz for the mean backfill soil case. The responses obtained from this set of SSI analyses and the analyses using in situ soil as the horizon were enveloped. The staff reviewed the above information per the SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines, and found the modeling method to be acceptable. The adequacy of the finite element SSI model to transmit frequencies of up to 33 Hz in the SSI analysis was reviewed earlier under the topic “Model Refinement and Passing Frequency” in Subsection 3.7.2.4.3, “Procedures Used for Analytical Modeling,” of this SER.

In the response to Item 2 of **RAI 03.07.02-5**, the applicant states that for the UHS/RSW pump house SSI analysis, the input motion described in COL FSAR Subsection 3H.6.5.1.1.2 was applied to the finished grade in the free field, and SASSI2000 “transmitting boundaries” were used at the side boundaries of the model. For the bottom boundary, elastic halfspace was used. The staff found that the SSI analysis is based on a substructure method that does not use lateral transmitting boundaries on the side or halfspace boundaries at the bottom of the SSI model, such as those used by the total SSI models in which structure and soil domains are analyzed together in one step. Therefore, Item 2 of **RAI 03.07.02-17** asked the applicant to clarify and correct the above statement. The staff reviewed the applicant’s clarifying statement in the response to **RAI 03.07.02-17 (ML100480204)** and found the method of the SSI analysis acceptable.

The applicant's response to Item 3 of **RAI 03.07.02-5** in conjunction with the response to a follow-up question in RAI 03.07.02-17 regarding soil and backfill properties used in the SSI model was reviewed earlier under the topic "Strain-Compatible Soil and Backfill Properties" in Section 3.7.2.4.3 of this SER. The staff found the response acceptable, because the model uses the strain-dependent soil properties per SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines.

In the response to Item 4 of **RAI 03.07.02-5**, the applicant states that the variability in soil and backfill properties was accounted for by performing a total of six SSI analyses (three with LB, mean, and UB in situ soil cases and three with LB, mean, and UB backfill soil cases) and enveloping the results for all cases. The staff found the response acceptable, because it meets SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines regarding the modeling of supporting soil.

In the response to Item 5 of **RAI 03.07.02-5**, the applicant states that the effects of side soil-wall separation for the UHS/RSW pump house during a seismic event will be analyzed using the method described in ASCE 4–98 Subsection 3.3.1.9. A review of this issue is discussed earlier under the topic "Effects of Soil Separation from the Walls" in Subsection 3.7.2.4.1, "Seismic Analysis Methods," of this SER. The review is based on response to **RAI 03.07.02-23** and the response is found acceptable.

In the response to Item 6 of **RAI 03.07.02-5**, the applicant states that the results of the SSI analyses for the UHS/RSW pump house and RSW piping tunnel were not available. Therefore, Item 4 of **RAI 03.07.02-17** and **RAI 03.07.02-3** requested the applicant to provide this information per SRP Section 3.7.2, SRP Acceptance Criterion 2 guidance corresponding to the seismic analysis performed for site-specific Category I structures (including the UHS/RSW pump house and the RSW piping tunnel), in sufficient detail comparable to ABWR DCD Subsection 3.7.2.2. A discussion of the staff's review follows below. The staff reviewed the information in the applicant's response to **RAI 03.07.02-3**, including proposed changes to COL FSAR Subsection 3H.5.2.2 regarding seismic analyses of the UHS/RSW pump house and the RSW piping tunnel, as well as any additional follow-up information. As discussed below, the applicant's SSI analysis of UHS/RSW pump house and the RSW piping tunnel is considered acceptable.

UHS/RSW Pump House

In the response to **RAI 03.07.02-3** dated September 3, 2009 **(ML092510038)**, the applicant states that the seismic analysis of the UHS/RSW pumphouse was performed using a frequency-domain time history analysis with SASSI2000. In addition, the applicant calculated the fixed-base dominant frequencies and mass participation factors for the UHS/RSW pump house and provides the results in Table 3H.6-3 (up to 33 Hz) in the response to **RAI 03.07.01-13**. A single set of three-component acceleration time histories was used in the analysis. The applicant states that the seismic accelerations and displacements for the UHS/RSW pump house are in Table 3H.6-4 and the ISRS are in Figures 3H.6-16 through 3H.6-39 in the response to **RAI 03.07.01-13**.

The staff did not find the referenced tables and figures in the response to **RAI 03.07.01-13**. In addition, the response to **RAI 03.07.02-3** did not provide details on the seismic analysis of the

UHS/RSW pump house. Because the staff found the response to **RAI 03.07.02-3** incomplete, the staff issued **RAI 03.07.02-15** requesting the following information regarding the UHS/RSW pump house:

1. Fixed-base dominant frequencies and mass participation factors referenced in Table 3H.6-3

2. Seismic accelerations and displacements referenced in Table 3H.6-4

3. A sufficiently detailed description of the model and method used to calculate fixed-base frequencies and participation factors

4. A description of how the three orthogonal components of the input motion were applied and how the results were combined

5. A description of how the input motion was specified in the SSI analyses

6. A description of the frequencies the model was analyzed for in SASSI2000 and the frequency cutoff that was used

7. A figure showing the finite element model of the structure in relation to the layered soil system

8. A description of how ground water effects were treated in the SASSI2000 model

9. A description of the time step, number of acceleration points, and duration of motion (including duration of quiet zone) used in the input motion for the SASSI analysis

10. A description of how seismic forces and moments were calculated for design (including plots of total shear and moment diagram profiles)

11. If a separate static analysis was performed to obtain seismic forces and moments, a sufficiently detailed description of how this model was applied (i.e., model, boundary conditions, loads, soil spring values, etc.)

12. Calculated maximum values of the soil-retaining wall displacements relative to the free field

13. Further details on how hydrodynamic forces were calculated and applied to the equivalent static model

The staff's review of the applicant's response to **RAI 03.07.02-15** dated February 10, 2010 **(ML100550613)**, is discussed below:

1. A review of Item 1 of **RAI 03.07.02-15** is provided earlier in Subsection 3.7.2.4.2, "Natural Frequencies and Responses," of this SER. The response is acceptable.

2. A review of Item 2 of **RAI 03.07.02-15** is provided earlier in Subsection 3.7.2.4.2 of this SER. The response is acceptable.

3. In the response to Item 3 of **RAI 03.07.02-15**, the applicant describes the finite element model development and details for the UHS basin with the enclosed cooling tower and the RSW pump house. The revised FSAR Figures 3H.6-15 and 3H.6-15a in this RAI response show a 3-D finite element model of the UHS/RSW pump house used for the SSI analysis with SASSI2000. The applicant performed all SSI analyses using uncracked concrete properties and OBE damping. One additional case corresponding to the cracked concrete properties was also analyzed. Structural mass included the dead load of structures and major equipment plus a 2.4-kPa (50-psf) load corresponding to attachment loads such as piping, grating, electrical cable trays and conduits, and HVAC, plus 25 percent of the floor live load. The applicant calculated the impulsive water mass using the procedure described in Commentary Section C3.5.4 of ASCE 4–98 and includes the calculated mass in the model. The staff found that the applicant has addressed the staff's concerns regarding the effects of structural and soil model refinement, the ground water table, and cracked concrete cases in the analysis. The staff's assessment of the applicant's sensitivity studies addressing the impact of the effects of structural and soil model refinement, the ground water table, and cracked concrete cases on the calculated seismic responses is provided earlier in Subsection 3.7.2.4.3, "Procedures used for Analytical Modeling," of this SER.

4. In the response to Item 4 of **RAI 03.07.02-15**, the applicant states that the seismic wave field in the SSI analysis consisted of vertically propagating plane shear and compression waves with control motion specified at the free-field ground surface. For the shear wave field, the control motion consisted of two horizontal components of input motion in the x- and y-directions. For the compression wave field, the control motion consisted of one vertical component of input motion in the z-direction. The applicant performed three separate SSI analyses with the input motion specified in the x-, y-, and z-directions. The co-directional responses from the three analyses were combined using the SRSS method. The final responses were then calculated by enveloping the responses in each direction for all cases in the analysis. The staff reviewed this information according to guidance in the SRP Section 3.7.2 Acceptance Criteria and RG 1.92 and found it acceptable.

5. The applicant provided the requested clarifications regarding the application of input motion to the SSI model in the response to Item 5 of **RAI 03.07.02-15**. The applicant applied the input motions at the free-field ground surface. The staff found this approach consistent with ABWR DCD Section 3A.6 and therefore acceptable.

6. In the response to Item 6 of **RAI 03.07.02-15**, the applicant provides a table summarizing the calculated frequencies as well as the cutoff frequencies used in the SSI analysis for various cases (including LB, best estimate [BE], and UB in situ soil cases; LB, BE, and UB backfill soil cases; and cracked concrete and de-bonded soil cases). The selected cutoff frequency for the different cases in the analysis varies from a low of about 16 Hz to a high of 25 Hz. The applicant states that the lowest cutoff frequency (16 Hz) meets the recommended values in ASCE 4–98, Subsection C3.3.3.4.

   With respect to the frequency cutoff, the staff has not endorsed ASCE 4–98, Subsection C3.3.3.4 as acceptable criteria for selecting the cutoff frequency in the SSI analysis, particularly for generating the ISRS. Therefore, the staff issued **RAI 03.07.02-24** requesting a comparison of the transfer functions by increasing the frequency cutoff to a minimum of 33 Hz for all cases under consideration in the analysis, thus demonstrating that the cutoff frequencies used in the SSI analysis are acceptable.

The staff needed this information to ensure that the selected cutoff frequencies of less than 33 Hz for the SSI analysis will accurately or conservatively account for the effects of the SSI. The applicant's response regarding the impact of the frequency cutoff on the SSI results for the UHS/RSW pump house was reviewed earlier under the topic "Model Refinement and Passing Frequency" in Subsection 3.7.2.4.3 of this SER. The staff found this response acceptable.

7. In the response to Item 7 of **RAI 03.07.02-15**, the applicant refers to FSAR Figure 3H.6-15. This figure shows a N-S cross section of the 3-D finite element model of the structure with soil layers. The staff's review of this information was discussed earlier in Item 3 of the response. The staff found the information acceptable.

8. The applicant's response to Item 8 of **RAI 03.07.02-15** regarding the treatment of the ground water table in the SSI analysis was reviewed earlier under the topic "Ground Water Effects" in Subsection 3.7.2.4.3 of this SER. The staff found this information acceptable.

9. The applicant's response to Item 9 of **RAI 03.07.02-15** regarding the time step, the number of acceleration points, and the duration of motion (including duration of the quiet zone) used in the input motion for the SASSI analysis was reviewed earlier in Subsection 3.7.1.4.2, "Design Time Histories," of this SER. The staff found this information acceptable.

10. The applicant's response to Item 10 of **RAI 03.07.02-15** regarding the calculation of seismic forces and moments for design was reviewed as part of the applicant's response to Item 11 of the response to **RAI 03.07.02-15.** This review is discussed below in Item 11.

11. In the response to Item 11 of **RAI 03.07.02-15**, the applicant describes in detail the equivalent static analysis used to calculate seismic forces and moments for the seismic design of the UHS/RSW pump house. The equivalent static analysis was performed using SAP2000. The applicant provides the finite element model used in the SAP2000 analysis, as well as two tables comparing two section-cut seismic forces and moments calculated from the dynamic SSI analysis using SASSI2000 and an equivalent static analysis using SAP2000. The comparison was made to further support the conservatism inherent in the equivalent static procedure used for the design of the UHS/RSW pump house. The pseudo-static analysis procedure used by the applicant to develop seismic forces and moments for the design of the UHS/RSW pump house is acceptable because the staff found that the method is in accordance with industry practice and is conservative.

    In addition, the applicant provides the soil spring constants in the x-, y-, and z-directions for the UHS/RSW pump house foundations used in the equivalent static analysis. The soil springs are reviewed in the response to **RAI 03.08.04-23** in Subsection 3.8.4.4.2, Subpart A.1 of this SER.

    Furthermore, in the response to Item 11 of the RAI, the applicant states that dynamic soil loads were calculated in accordance with ASCE 4–98, Subsection 3.5.3.2.2 and were compared to the enveloped soil pressures from the SSI analysis. In addition, the maximum pressures were used in the SAP2000 model. However, no results were provided in the response.

RAI 03.07.02-12 asked the applicant to provide the dynamic lateral soil pressures used on the exterior walls below grade calculated from the SSI analysis, for a comparison with those from the ASCE 4–98 recommendations, thus ensuring that the structure's inertial interaction effect on the exterior walls is adequately accounted for in the wall design. The staff's review of the information is as follows:

In the response to RAI 03.07.02-12 **(ML092530685)**, the applicant provides the requested calculated pressures, which include the effects of the SSI in the Supplement 2 response to RAI 03.07.01-13 **(ML100050225)**. The envelope of soil pressures on the UHS/RSW pump house walls was calculated from the SSI analysis (and is included in the Supplement 2 response to RAI 03.07.01-13) and compared to soil pressures calculated using the ASCE 4–98 procedures (shown in Figures 3H.6-41, 3H.6-42, and 3H.6-43 of the same response). The comparison revealed that for the UHS basin east and west walls, the SSI pressures were significantly higher than those of ASCE 4–98 (by approximately a factor of 2 or more). For the UHS basin north and south walls, the SSI pressures were higher at all points but, in particular, significantly higher near grade. The overall difference between the magnitude of the calculated SSI and ASCE 4–98 pressures for the RSW pump house is less than those for the UHS basin, but the distribution of pressures is quite different. Based on the results provided by the applicant, the staff concluded that the use of pressures premised on ASCE 4–98 methodology may under predict the pressure loads on the UHS/RSW pump house walls and could impact the wall design. Therefore, the staff issued RAI 03.07.02-22 requesting a justification for not using the dynamic soil pressures calculated from the SSI analysis for the design of the UHS/RSW pump house walls, which takes into account inertial effects of the structure as well as the SSI effects.

In the response to **RAI 03.07.02-22 (ML110770440)**, the applicant states that the design of the UHS/RSW pump house was revised to consider, in addition to incremental seismic soil pressures per ASCE 4–98, incremental seismic soil pressures from the SSI analysis for full and empty UHS basin cases and incremental seismic soil pressures from the SSSI analysis. Details of the SSI analysis of the UHS/RSW pump house for full and empty UHS basin cases are in COL FSAR Subsection 3H.6.5.2.4. Details of the SSSI analysis of the UHS/RSW pump house with other structures (i.e., the RSW piping tunnel and the DGFOSVs) are in COL FSAR Subsection 3H.6.5.3. Comparisons of the lateral seismic soil pressures from the SSI and SSSI analyses with those from the ASCE 4–98 methodology, together with the envelope of seismic soil pressures used for the design, are shown in Figures 3H.6-218 through 3H.6-220 in the COL FSAR.

Because the applicant considered the SSI and SSSI effect in establishing the seismic soil pressure demand, the staff found the soil pressure used for the design of the UHS/RSW pump house walls acceptable.

12. In the response to Item 12 of **RAI 03.07.02-15**, the applicant summarizes the envelope of calculated maximum displacements relative to the input motion at the free-field grade level obtained from the SSI analyses of several selected locations in the UHS/RSW pump house structure. The maximum calculated horizontal displacements of the soil-retaining walls are less than 0.5 cm (0.2 in.). This calculation indicates that a full passive soil pressure would not be mobilized for the site-specific SSE input.

The staff also reviewed the calculated displacements for the cooling tower walls (top, mid-level, and bottom) provided by the applicant in a summary table in this response. The reported displacements in the E-W direction are significantly higher than those in the N-S direction, which is consistent with the stiffness of the cooling tower structure along the E-W and N-S axes. The reported displacements in the E-W direction for the mid-level of the cooling tower are generally higher than those for the top and bottom. The applicant attributes this displacement to the out-of-plane behavior of the flexible walls, and the staff agreed. Based on the above information, the staff found no discrepancies in the pattern of reported displacements in the structure. The staff's assessment of the applicant's sensitivity studies to address the impact of SSI mesh refinement issues on the calculated seismic responses of the UHS/RSW pump house was documented earlier under the topic "Model Refinement and Passing Frequency" in Subsection 3.7.2.4.3 of this SER. The staff found the applicant's sensitivity studies acceptable.

13. The applicant's response to Item 13 of **RAI 03.07.02-15** regarding how hydrodynamic effects were considered in the seismic analysis of the UHS basin was previously reviewed and found to be acceptable, as discussed under the topic "Hydrodynamic Effects of Water in the UHS Basin" in Subsection 3.7.2.4.3 of this SER.

In conclusion, the staff found the site-specific SSI analyses performed to determine the seismic demand for the UHS/RSW pump house acceptable and in accordance with the SRP Section 3.7.2, SRP Acceptance Criterion 4 guidelines. A resolution of the impact of the use of the SM on the results of the SSI analysis is discussed in Subsection 3.7.2.4.20 of this SER.

RSW Piping Tunnel

In the response to **RAI 03.07.02-3 (ML092510038)**, the applicant states that the concrete elements of the RSW piping tunnel were sized so that the structure is rigid with a minimum frequency greater than 33 Hz (i.e., there will be no structural amplifications). Therefore, the horizontal and vertical input spectra defined in COL FSAR Subsection 3H.6.5.1.1.1 were established as the ISRS for the RSW piping tunnel. The applicant also states that an equivalent static analysis method was used to analyze the RSW piping tunnel, as described in COL FSAR Subsection 3H.6.6.2.2. The applicant used the method described in ASCE 4–98, Subsection 3.5.3.2 to calculate dynamic soil pressures on the RSW piping tunnel walls. The strains created in the tunnel walls from the passage of seismic waves through the soil during the SSE were computed using the method described in ASCE 4–98, Subsection 3.5.2.1.

The staff reviewed this response and found no details on the seismic modeling and analysis of the RSW piping tunnel. For example, because of a lack of modeling of a portion of the RSW piping tunnel runs between the RB and RWB, the response is expected to be significantly affected by the SSSI effects as a result of RWS piping tunnel's close proximity to the heavy RB and RWB structures. The methodology described by the applicant for the seismic response analysis of the RSW piping tunnel also did not address the SSI effects. Therefore, the staff issued **RAI 03.07.02-15** requesting the following information regarding the RSW piping tunnel seismic analysis:

1. A description of the equivalent static analysis method used for the RSW piping tunnel

2. A description of how seismic and static loads were calculated and applied to the model; show the model and boundary conditions (including the soil springs) used in the analysis

3. A description of the type of strains (tensile or compression) calculated in the RSW piping tunnel

4. A description of how both axial strains and transverse shear demands were considered in the analysis of the RSW piping tunnel

5. A description of how concrete elements of the RSW piping tunnel were determined to be rigid so that there were no in-structure amplifications

6. A description of the SSI analysis from which accelerations were obtained to establish SSI forces for the analysis of the RSW piping tunnel

The staff's review of the applicant's response to **RAI 03.07.02-15 (ML100550613)** is provided below:

1. In the response to Items 1, 5, and 6 of **RAI 03.07.02-15**, the applicant states that the 2-D SSI analysis of the RSW piping tunnel was performed to quantify the in-structure amplification. The applicant includes in the response the results of this new analysis in terms of the broadened horizontal and vertical ISRS for different damping values in Figures 3H.6-138 and 3H.6-139, respectively. These figures show that the spectra's peak is lower than the 0.21g zero period acceleration (ZPA) used for the design of the tunnel walls and slabs. However, the staff did not find any details on the SSI analysis of the tunnel in the applicant's response, as requested in the RAI. As a result, the staff issued **RAI 03.07.02-24** requesting a detailed description of how the SSI analysis of the RSW piping tunnel was performed (including the SSI methodology, figures showing the SSI model and boundary conditions, a summary of the soil and structure properties, the input motion, etc.). The RAI asked the applicant to include this information in the COL FSAR.

   In the Supplement 1 response to **RAI 03.07.02-24 (ML103360074)** and in a subsequent Revision 1 to the Supplement 1 response to **RAI 03.07.02-24** dated March 7, 2011 **(ML110730066)**, the applicant states that three sections of the RSW piping tunnel were used in the SSI analysis with SASSI2000:

   <u>Section 1</u>: An E-W typical 2-D section between the UHS/RSW pump house and the RB for the SSI analysis of the RSW piping tunnel to determine the seismic demand

   <u>Section 2</u>: An E-W 2-D section of the tunnel between the RWB and the RB for the SSSI analysis to determine the SSSI effect on seismic soil pressures

   <u>Section 3</u>: A N-S 2-D section of the tunnel between the DGFOSV and UHS/RSW pump house for the SSSI analysis to determine the SSSI effect on seismic soil pressures

   The SSI model for Section 1 and the SSSI models for Sections 2 and 3 are shown in FSAR Figures 3H.6-209, 3H.6-210, and 3H.6-211, respectively.

   The applicant used the SSI model for Section 1 to analyze the dynamic response of the tunnel. From this analysis, the applicant developed the seismic demand in terms of maximum accelerations, ISRS, and dynamic soil pressures for the design. Because this model does not explicitly include SSSI effects, the input motion consisted of an amplified site-specific SSE motion considering the effects from the nearby heavy RB and UHS/RSW pump house structures, as discussed earlier under the topic "Amplified Ground Motion Input to SSI Analysis" in Subsection 3.7.2.4.4, "Soil-Structure Interaction," of this SER. The SSI analyses were performed for eight cases (three LB, mean, and UB in situ soil cases; three LB, mean, and UB backfill soil cases; one mean in situ soil case with cracked concrete; and another with soil separation from the sidewalls in the upper 6 m [20 ft]). The concrete was assigned a 4 percent damping in all cases except the cracked concrete case, which assumed a 7 percent damping. The ground water table was modeled at 2.4 m (8 ft) below surface level. Ground water effects were modeled using a minimum P-wave velocity of 1,524 m/s (5,000 ft/s), except where the use of this minimum P-wave velocity results in a Poisson's ratio that exceeds 0.495. The cutoff frequency was set at 33 Hz. Other details of the modeling and the analysis are in COL FSAR Subsection 3H.6.5.3. The widened envelope of the resulting response spectra for the base slab, intermediate floors, and roof slab are shown in FSAR Figures 3H.6-138 and 3H.6-139 and were used as the design ISRS for the RSW piping tunnel.

The staff reviewed this information and found it acceptable, because the analysis includes a consideration of both SSI and SSSI effects (as discussed in the following paragraphs) with one exception: the applicant was asked to justify the use of 7 percent damping for the cracked concrete case generating the ISRS. The response to **RAI 03.07.01-22 (ML110730069)** states that the interaction coefficient (i.e., required capacity/provided capacity) for walls, interior slabs, the roof slab, and the basemat of the RSW piping tunnels are 0.68, 0.75, 0.44, and 0.78 respectively. Considering these interaction coefficients, the amount of cracking will be significant, thus justifying the use of a 7 percent damping. The staff found the applicant's justification in the response to **RAI 03.07.02-24** acceptable and supported by the response to **RAI 03.07.01-22**, because the applicant is using this damping value only for the cracked concrete case.

The applicant used the SSSI model for Section 2 to determine any increases in lateral seismic soil pressures on the tunnel walls resulting from the tunnel's proximity to heavy RB and RWB structures. Because this model explicitly includes SSSI effects, the input motion consisted of site-specific SSE motion specified at grade elevation. The analysis was performed for a single case of UB in situ soil. Other modeling details are similar to those of Section 1 and are described in COL FSAR Subsection 3H.6.5.3. The calculated seismic soil pressures are shown in FSAR Figures 3H.6-212 and 3H.6-213. The results indicate significant increases in lateral seismic soil pressures in some portions of the wall on both the tunnel east and west walls from SSSI effects. The seismic soil pressures used for the design of the walls envelop seismic soil pressures calculated from all soil cases in both the SSI and SSSI analyses, with the exception of a 0.3 m (2 ft) wall height at a depth of 6.7 to 7.3 m (22 to 24 ft) below the ground surface from the soil separation case. Based on a review of the above pressure distributions, the staff concluded that although there was some local exceedance, the seismic soil pressure profile used for the design envelopes the total seismic soil pressure demand from the SSSI analysis. Therefore, the design pressures are acceptable. The design of the RSW tunnel is reviewed in Subsection 3.8.4.4.2, Subpart B.1 of this SER.

The applicant used the SSSI model for Section 3 to determine any increases in the lateral seismic soil pressures on the tunnel walls resulting from the tunnel's proximity to the heavy RB and UHS/RSW pump house structures as well as to two adjacent

DGFOSVs. Again, because this model explicitly includes SSSI effects, the input motion consisted of site-specific SSE motion specified at grade elevation. The analysis was performed for a single case of UB in situ soil. Other modeling details are similar to those of Sections 1 and 2 and are described in COL FSAR Subsection 3H.6.5.3. The calculated seismic soil pressures are shown in FSAR Figures 3H.6-214 and 3H.6-215. The calculated seismic soil pressures on the east, north, and south walls of the RSW piping tunnel are enveloped by the seismic design pressures and are therefore acceptable.

RAI 03.07.01-29 requested the applicant to further justify the use of the UB in situ soil case instead of the UB backfill soil case for calculating the seismic soil pressures on the RSW piping tunnel walls, because the latter represents a stiffer material in-between the walls. In the Supplement 1 response to **RAI 03.07.01-29 (ML113360516)**, the applicant presents the results of the SSSI analysis considering LB in situ, UB in situ, and UB backfill soil cases showing the governing soil case for soil pressures for the RSW piping tunnel east and west walls. The staff reviewed these results and found that the bounding soil cases are the LB in situ and the UB backfill soil cases.

The applicant also performed a sensitivity study to evaluate the effect of including vertical motion in the calculated lateral seismic soil pressures on the tunnel walls. To verify this sensitivity, the SSSI model for Section 2 (i.e., the E-W 2-D section of the RSW piping tunnel between the RWB and RB) was analyzed for both E-W and vertical input motions. The resulting soil pressures were based on the SRSS of the results for two horizontal motions and one vertical motion and were compared with those using the horizontal motion only. The comparisons are shown in FSAR Figures 3H.6-216 and 3H.6-217. The results show that the effect of vertical input motion on the resulting soil pressure is negligible.

2. In the responses to Items 2 and 6 of **RAI 03.07.02-15**, the applicant states that simple manual calculations were used for the analysis and for the design of individual components of the RSW piping tunnel. In this analysis, the tunnel walls, slabs, and basemat were considered to be rigid elements, and seismic loads were calculated based on a ZPA of 0.21g. The applicant further states that the analysis does not include any model or soil springs; and the seismic loads are applied in terms of dynamic soil pressures on the exterior walls, calculated according to ASCE 4–98 recommendations. The staff has not endorsed the dynamic soil pressures recommended in ASCE 4–98 for the design of tunnel walls. Therefore, Item 2 of Part 2 of **RAI 03.07.02-24** asked the applicant to compare the dynamic soil pressures on the tunnel walls that were calculated using the 2-D SSI model with those of ASCE 4–98. The staff requested these comparisons to ensure that the design pressures still bound when considering the effects of a kinematic interaction between tunnel structures and surrounding soils, as well as the effects of the SSSI from nearby heavy structures.

   In Revision 1 to the Supplement 1 response to **RAI 03.07.02-24 (ML110730066)**, the applicant provides the requested comparisons between the seismic soil pressures from the SSSI analysis and the calculated seismic soil pressures using the method in ASCE 4–98 in FSAR Figures 3H.6-212 through 3H.6-215. The comparisons show that the seismic soil pressures accounting for the SSSI effects significantly exceed those recommended by ASCE 4–98. As a result, the applicant reevaluated the existing design to address any increases in seismic soil pressures resulting from the SSSI effects. Based on this reevaluation, the applicant states that the existing design is adequate for the SSSI soil pressures. However, the applicant revised a portion of the design for the access region near the UHS/RSW pump house because of design change. The applicant revised FSAR Table 3H.6-6 to reflect this design change. The staff's review of the RSW tunnel design is in Subsection 3.8.4.4.2, Subpart B.1 of this SER.

In the response to Part 2 of Revision 1 to the Supplement 1 response to **RAI 03.07.02-24**, the applicant states that a finite element analysis using a 2-D SAP2000 model with soil springs representing the foundation was also performed to confirm the adequacy of the design using manual calculations, as described in the response to **RAI 03.07.02-15**. Furthermore, the RSW piping tunnel design accounts for axial tensile strains and induced forces at the tunnel bends from SSE wave propagation. COL FSAR Subsection 3H.6.6.2.2 describes how axial tensile strains are accounted for. Induced forces at the tunnel bends are determined in accordance with ASCE 4–98, Subsection 3.5.2.2 by considering the structure as a beam on the elastic foundation. The analytical methods to determine the axial strain on the RSW piping tunnel due to SSE wave propagation is considered acceptable, because the procedure is in accordance with the guidance of SRP Section 3.7.2, SRP Acceptance Criterion 12. The tunnel design for the axial tensile strains and the forces induced at the tunnel bends from the SSE wave propagation are reviewed in Subsection 3.8.4.4.2, Subpart B.1.4 (Item c) of this SER.

3. In the responses to Items 3 and 4 in **RAI 03.07.02-15 (ML100550613),** the applicant states that the tensile axial strain on the RSW piping tunnel from the SSE wave propagation is determined using the equations and commentary outlined in Subsection 3.5.2.1 of ASCE 4–98. Equation 3.5-1 of ASCE 4–98 is used to compute the axial strain. The tunnel design for the axial tensile strains and forces induced at the tunnel bends from the SSE wave propagation is reviewed in Subsection 3.8.4.4.2, Subpart B.1.4 (Item c) of this SER.

In conclusion, the staff found the site-specific SSI analyses acceptable for determining the seismic demand for the RSW piping tunnel and in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 4. A discussion of the resolution to the impact from using the SM on the results of the SSI analysis is in Subsection 3.7.2.4.20, which is located later in this SER.

Seismic Analysis of Diesel Generator Fuel Oil Storage Vaults (DGFOSVs)

Because the shear wave velocity parameter of the subgrade material (soil or backfill) supporting these structures may be less than 305 m/s (1,000 ft/s), the staff issued **RAI 03.07.01-19** requesting information about the supporting media for DGFOSVs. The RAI also requested the quantitative results for the reconciliatory site-specific seismic analysis with appropriate consideration for the dynamic soil or backfill properties and the potential impact on the FIRS, SSI, settlement calculations, and the structural design of the DGFOSVs.

In the Revision 2 response to Item 1 of **RAI 03.07.01-19** dated June 7, 2010 **(ML101620284)**, the applicant presents the approach used to develop the input motions for the SSI analysis and the design of the DGFOSVs, while taking into account the impact of the nearby heavy RB and UHS/RSW pump house structures. The applicant states that:

> *Conservatively, a 3-dimensional SAP2000 response spectrum analysis was used to obtain the safe-shutdown earthquake (SSE) design forces due to structure*

inertia. The seismic induced dynamic soil pressures on DGFOSV walls were computed using the method of ASCE 4-98, Subsection 3.5.3.2.

The response, however, did not provide details as to how the SSI analysis of the DGFOSVs was performed and how the input motions developed were subsequently specified in the SSI analysis of the DGFOSVs to develop the structural responses and the ISRS for equipment and subsystems within the DGFOSVs. In the response, it appeared that the applicant had not explicitly included the DGFOSV structural model in the SASSI2000 model of the RB and UHS/RSW pump house SSI models to properly evaluate the SSSI effects on the DGFOSVs. In order for the staff to determine whether the evaluation of the DGFOSVs for the SSE appropriately accounted for the SSI effects, Part 1 of **RAI 03.07.01-27** asked the applicant to provide the following information:

a) A detailed description of the method used for the SSI analysis of the DGFOSV, including the procedures for the treatment of the strain-dependent backfill material properties in the model; the input motion used and how it is specified in the analysis, the variation in soil properties, and the computer programs used for the SSI analysis.

b) A detailed description of how the SAP2000 analysis of the DGFOSV was performed, including how foundation soil/backfill materials were represented, the number of modes extracted, the modal damping values used, how the input motion is specified, and the type of boundary conditions used.

c) A detailed description demonstrating that the DGFOSV foundation response spectra and dynamic soil pressure (on the DGFOSV basement walls using the ASCE 4–98 criteria) used in the design of the DGFOSV will envelop the results of the SSSI analysis, which explicitly models the DGFOSV structure in the SSI model of the RB and RSW pump house structures.

d) A detailed description of whether there are any Category I tunnel structures for transporting diesel fuel oil between the DGFOSV and the diesel generator located in other buildings. The description should include the layout, configuration, seismic analysis, and design method.

In the Supplement 1 response to Items (a) through (c) of **RAI 03.07.01-27 (ML110730064)**, the applicant provides the requested information. The response to Item (d) of **RAI 03.07.01-27** is reviewed later in this SER under the topic “Seismic Analysis of Diesel Generator Fuel Oil Tunnels (DGFOTs).”

In the Supplement 1 response to Item (a) of **RAI 03.07.01-27 (ML103620340)** and the revised Supplement 1 response dated March 7, 2011 **(ML110730064**), the applicant describes the DGFOSVs as reinforced concrete structures located below grade, with an access room above grade. The DGFOSVs house fuel oil tanks and transfer pumps. COL FSAR Figure 3H.6-221 shows the locations of the DGFOSVs and nearby structures. The applicant also states that two types of SSI analyses were performed for the DGFOSVs:

(a) 3-D SSI analyses of only the DGFOSVs for calculating the ISRS and design accelerations/forces of the structure. These analyses considered both full and empty fuel oil tanks.

(b) 2-D SSSI analysis of the DGFOSVs with adjacent structures to obtain seismic soil pressures.

(a) 3-D SSI Analysis of the DGFOSVs

As discussed in COL FSAR Section 3H.6.7, the applicant modeled the fuel tank with the fuel and the tank mass, lumped at the tank's center of gravity, with the fuel tank lumped mass rigidly connected to the basemat at the tank saddle locations. To address the staff's concern regarding this modeling approach, the applicant stated that the fuel tank procurement specification would require the fuel tank with fuel in it to have predominant frequencies greater than 33 Hz in both horizontal and vertical directions. The fuel tank portion of the model was assigned a 0.5 percent damping. For other parts of the structure, 7 percent and 4 percent damping values were used. The results from the 7 percent structural damping were then used to design the DGFOSV, and the results from the 4 percent structural damping were used to generate the ISRS. The analyses considered both full and empty fuel oil tank conditions. COL FSAR Table 3H.6-1 shows the strain-compatible soil properties of the in situ soils, and Table 3H.6-2 shows the strain-compatible soil properties of the backfill soils. However, for soil layers below the water table, the Poisson's ratio was capped at 0.495. The full fuel tank case was analyzed for both 4 percent and 7 percent structural damping in 8 cases (LB, mean, and UB in situ soil; LB, mean, and UB backfill over LB, mean, and UB in situ soil, respectively; UB in situ soil with soil separation; and UB in situ soil with cracked concrete). The empty fuel oil tank case was analyzed for the UB in situ soil case only. Input consisted of 0.3g RG 1.60 spectra, which enveloped all other spectra derived from the SSI analyses to take into account the impact from nearby large structures, as discussed in COL FSAR Section 3H.6.7. This COL FSAR section also describes the modeling and other details of the analyses, including the passing frequency requirement of 33 Hz and the development of design spectra to evaluate equipment inside the DGFOSVs. All 3-D SSI analyses of the DGFOSV were initially performed with SASSI2000 using the SM.

The staff raised verification and validation (V&V) issues regarding the accuracy of the SM used in the SASSI2000 to calculate the seismic response of embedded structures, as well as a similar concern the DNFSB raised in a recent letter to the U.S. DOE. To address the V&V issue, the applicant reanalyzed the seismic SSI response of the DGFOSV using the MSM. The staff reviewed and accepted the revised results of the SSI DGFOSV analysis for the STP Units 3 and 4 application in the context of the SM issues. Subsection 3.7.2.4.20 of this SER discusses the staff's review and the basis for the staff's acceptance of the new results.

The staff reviewed the amplified ground motions used for inputs to the SSI DGFOSV model earlier under the topic "Amplified Ground Motion Input to SSI Analysis" in this subsection of the SER, and found acceptable.

(b) 2-D SSSI Analysis of the DGFOSVs

Two 2-D SSSI models were developed and analyzed to evaluate the effects of nearby structures on the three DGFOSVs and to calculate seismic soil pressures on the structures. The first SSSI analysis models a section cut in the N-S direction consisting of the UHS/RSW pump house, the RSW piping tunnel, DGFOSV No. 1B, DGFOSV No. 1C, and the RB. Details of this SSSI analysis are in Item 1 of the Supplement 1 response to **RAI 03.07.02-24 (ML110730066)** and COL FSAR Subsection 3H.6.5.3. The second SSSI analysis models a section cut in the E-W direction consisting of the DGFOT, DGFOSV 1A, and the crane foundation retaining wall (CFRW). Details of this SSSI analysis are in Item (a) of the revised Supplement 1 response to **RAI 03.07.01-27 (ML110730064)** and in COL FSAR Section 3H.6.7. The soil properties for in situ and backfill soils are similar to those used in the above SSI analyses. To account for the effects of soil variations, the applicant performed five analyses

(UB in situ soil with UB backfill in between the structures; LB in situ soil with LB backfill in between the structures; and mean in situ soil with UB, LB, and mean backfill in between the structures). Input motion to the SSSI models consisted of a site-specific SSE described in COL FSAR Subsection 3H.6.5.1.1.2. The applicant performed all of the above 2-D SSSI analyses with SASSI2000 using the SM.

In the revised Supplement 1 response to Item (b) of **RAI 03.07.01-27**, the applicant states that the SAP2000 response spectrum analysis (RSA), which was previously used for the SSI analysis of the DGFOSVs in the response to **RAI 03.07.01-19**, is no longer used. Item (a) in the RAI response above describes this revised SSI analysis, which is the basis of all ISRS and maximum accelerations for the DGFOSV analyses. The applicant also revised the design of the DGFOSVs and conservatively determined seismic loads using the equivalent static method. Details of the equivalent static method are in Subsection 3.8.4.4.2, Subpart B.2.4 of this SER.

In the revised Supplement 1 response to Item (c) of **RAI 03.07.01-27**, the applicant provides comparisons of the 5 percent damped response spectra at the DGFOSV foundation level obtained from the 3-D SSI analysis (enveloped for all soil cases and used for the DGFOSV design) and 2-D SSSI analysis. These comparisons show that the design spectra obtained from the 3-D SSI analysis envelop the spectra obtained from 2-D SSSI analysis. The applicant also revised the structural analysis and design of the DGFOSV to consider incremental seismic soil pressures from the SSSI analysis described in the above Part 1a response. Figures 3H.6-226 through 3H.6-231 in the revised Supplement 1 response to **RAI 03.07.01-27** show comparisons of incremental seismic soil pressures obtained from the SSSI ASCE 4–98 methodology and incremental seismic soil pressures used in the design of all DGFOSV walls. Because dynamic soil pressures used in this design significantly envelop the seismic soil pressures calculated from SSI and SSSI analyses of all soil cases, as well as those using ASCE 4–98, the staff found the design seismic soil pressures acceptable. Subsection 3.8.4.4.2, Subpart B.2.4 of this SER reviews the DGFOSV design.

Because the SASSI2000 SM was used to calculate the SSSI response of the DGFOSV, the applicant revisited the results of both SSSI DGFOSV analyses in order to address the V&V issues raised by the staff regarding the accuracy of using the SM in SASSI2000 to calculate the seismic response of embedded structures, as well as a similar concern raised by the DNFSB in a recent letter to the DOE. To address this issue, the applicant reanalyzed the seismic SSSI response of both DGFOSV Sections 5 and 7 (see Figure 03.07.01-29 S1.79 in the Supplement 1 response to **RAI 03.07.01-29 [ML113360516]**) using the MSM and performed additional sensitivity studies to assess the impact from using the SM on wall pressures. The staff reviewed and accepted the revised results of the SSSI DGFOSV analysis for the STP Units 3 and 4 application in light of the SM issues. A discussion of the staff's review and the basis for accepting the new results are in Subsection 3.7.2.4.20, which is located later in this SER.

<u>*Seismic Analysis of Diesel Generator Fuel Oil Tunnels*</u>

In order for the staff to determine whether the evaluation of the DGFOTs for the SSE appropriately accounted for SSI effects, Item (d) of **RAI 03.07.01-27** asked the applicant to describe in detail whether there are any Category I tunnel structures for transporting diesel fuel oil between the DGFOSV and the diesel generator located in other buildings. The description should include the layout, configuration, seismic analysis, and design method.

In Item (d) of the Supplement 2 response to **RAI 03.07.01-27 (ML110730064),** the applicant states that there are three reinforced concrete DGFOTs approximately 15.2, 60.9, and 67.1 m (50, 200, and 220 ft) long for each unit. Each DGFOT is connected at one end to the RB and at the other end to a DGFOSV. There is a seismic gap between each of the tunnels and the adjoining RB or DGFOSV. COL FSAR Figure 3H.6-221 shows the layout of the DGFOT. Each DGFOT has two access regions that extend above grade. One access region is located where the tunnel interfaces with the DGFOSV, and another region is located where the tunnel interfaces with the RB. Dimensions and other details of the DGFOTs are in COL FSAR Section 3H.7.3.

Seismic analyses of the DGFOTs include SSI and fixed-base analyses to generate ISRS and SSSI analyses to develop seismic soil pressures on the tunnel walls for design. Reviews of these analyses are below.

<u>Seismic Analysis for Generating ISRS</u>

For the final ISRS for the tunnels and their access regions, the applicant used the widened enveloped spectra of the resulting ISRS from the following two seismic analyses:

(a) 2-D SSI analysis of a typical cross section of the DGFOT

(b) 3-D fixed-base seismic analysis of DGFOT No. 1B (approximately 15.2 m [50 ft] long), including its access regions at the two ends of the tunnel

A review of these two seismic analyses is discussed below.

(a) 2-D SSI Analysis of the DGFOT

A 2-D plane-strain model of the DGFOT was developed and used for this SSI analysis. Figure 3H.7-20 in COL FSAR Subsection 3H.7.5.2.1 depicts this model. There were sixteen case analyses performed with strain-compatible soil properties. Eight site-specific soil and backfill profiles include the LB, mean, and UB in situ soil; LB backfill over LB in situ soil; mean backfill over mean in situ soil; UB backfill over UB in situ soil; UB in situ soil with soil separation; and UB in situ soil with cracked concrete. Eight ABWR DCD generic soil profiles include UB1D, VP3D, VP4D, VP5D, VP7D, R, R with soil separation, and R with cracked concrete. Water table was assumed at 1.8 m (6 ft) below grade for site-specific soil and backfill cases and 0.6 m (2 ft) below grade for the ABWR DCD cases. In the site-specific soil and backfill cases, ground water effects were considered by using a minimum compression wave velocity of 1,463 m/s (4,800 ft/s) with Poisson's ratio capped at 0.495. The structure was assigned a 4 percent damping that included cracked concrete. The SSI model was meshed to allow an analysis of a passing frequency of at least 33 Hz. Acceleration time histories consistent with RG 1.60 response spectra anchored at 0.3g peak ground acceleration were used as input at the grade. This motion envelops both site-specific input motions and amplified site-specific motions considering the impact of nearby heavy RB and UHS/RSW pump house structures. COL FSAR Subsection 3H.7.5.2.1 describes additional details of these analyses. The computed ISRS at the top of the floor slab (middle of the span), at the slab roof (middle of the span), and at the mid-height of two walls of the cross section of the tunnel were enveloped in all cases to conservatively provide the ISRS for the entire 2-D cross section of the tunnel. The applicant performed the SSI analyses of the DGFOT with SASSI2000 using the DM.

During the October 18 through October 22 (2010) audit, the staff reviewed the applicant's analyses. The staff requested the applicant to calculate the FIRS for the DGFOT and to provide comparisons with the outcrop spectra at the same foundation level and with a broadband spectrum anchored at 0.1g. The staff needed this information to ensure that the foundation outcrop spectra used in the SSI model envelop the FIRS for all cases in the analysis and meet the minimum threshold in SRP Section 3.7.1 and 10 CFR Part 50, Appendix S. Figures 3H.7-22 through 3H.7-30 in the Supplement 2 response to **RAI 03.07.01-27**, depict comparisons of the outcrop response spectra and the FIRS in two horizontal directions and one vertical direction for the LB, mean, and UB in situ soil cases. These figures show that the foundation outcrop spectra envelop the FIRS in all cases, and that the response spectra at the SHAKE outcrop of the DGFOT foundation level envelop a minimum broadband spectrum conservatively defined as the RG 1.60 spectrum anchored at 0.1g.

(b) 3-D Fixed-Base Seismic Analysis of the DGFOT

The applicant developed a 3-D fixed-base model (fixed basemat) of DGFOT No. 1B running between the RB and DGFOSV No. 1B. The applicant used this model for the seismic analysis of the DGFOT, as described in COL FSAR Subsection 3H.7.5.2.1. The seismic analysis was performed with SAP2000 using the time history modal superposition method. The structure was assigned a 4 percent damping. Three-component acceleration time histories consistent with the RG 1.60 response spectra anchored at 0.3g peak ground acceleration were used as inputs at the base of the model. Other details of the analyses, including the ISRS at selected nodes, are in COL FSAR Subsection 3H.7.5.2.1.

The corresponding ISRS obtained from the 2-D SSI analysis and the 3-D fixed-base analysis described above were enveloped and used to develop the final widened ISRS for the horizontal and vertical directions of the entire DGFOTs and their access tunnels at the two ends. COL FSAR Figures 3H.7-31 and 3H.7-32, respectively, depict these final spectra.

The staff reviewed the results of the SSI DGFOT analyses and found them to be conservative; they meet the guidance of the SRP Section 3.7.2 acceptance criteria as follows.

- The SSI model input is a conservative envelope of the amplified ground motions that account for the SSSI effects from nearby heavy structures and the 0.3 g RG 1.60 spectra; the FIRS envelops the minimum requirement that FIRS must be greater than RG 1.60 anchored to 0.1g.

- The SSI analysis incorporates both the site-specific soil cases as well as the DCD generic soil cases.

- The ISRS envelops the SSI results as well as those from the fixed-base DGFOT analysis that used 0.3g RG 1.60 as input at the base of the model.

Based on the above evaluation and the fact that the seismic SSI DGFOT analysis was performed using the DM in SASSI2000, the results were therefore unaffected by the numerical issues connected to the use of the SM in the SSI analysis. The staff accepted the results of the DGFOT seismic analysis for the STP Units 3 and 4 application. For further discussions on the SASSI SM issues, see Subsection 3.7.2.4.20 below in this SER.

The amplified ground motions used for inputs to the SSI model of the DGFOT were reviewed earlier under the topic "Amplified Ground Motion Input to SSI Analysis" in this subsection of the SER. The staff found them acceptable.

2-D SSSI Analysis of the DGFOT

The applicant developed and analyzed two 2-D SSSI models to calculate the seismic soil pressures on the DGFOT walls. The models considered the effects from the nearby structures.

The first SSSI model was for a section cut in the E-W direction consisting of DGFOT No. 1C, DGFOSV 1A, and the CFRW. Details of this SSSI analysis are in the Supplement 1 response to **RAI 03.07.01-27** and in COL FSAR Section 3H.6.7. This model is discussed earlier in this subsection of the SER under the topic "2-D SSSI Analysis of DGFOSV."

The second SSSI model was also for a section cut in the E-W direction through the RB, DGFOT 1A, and the CFRW. COL FSAR Figure 3H.7-21 depicts the structural part of this SSSI model. The methodology for the SSSI model (including strain-compatible soil properties, soil case analyses, and the methods of analysis) is the same methodology used for the section cut through DGFOT 1C, DGFOSV 1A, and the CFRW, which is described above under the topic "2-D SSSI Analysis of DGFOSV" in this SER. This SSSI model was also meshed for passing frequencies up to at least 33 Hz. Details of this SSSI analysis are in the Supplement 1 response to **RAI 03.07.01-27** and COL FSAR Subsection 3H.7.5.2.2. COL FSAR Figures 3H.7-5 through 3H.7-8 show comparisons of the incremental SSI, SSSI, and ASCE 4-98 seismic soil pressures together with the enveloping seismic pressures used for the design of the DGFOT walls.

The staff reviewed the analytical procedure used to develop the seismic soil pressures for the design of the DGFOTs. In this review, the staff found the design pressures to be conservative because:

- The seismic soil pressure profiles are based on the maximum absolute values.

- The SSSI seismic soil pressures envelop the results from two different sections accounting for the effects from the adjacent heavy structures.

- The soil pressures in the seismic design envelop the seismic pressures from both the SSI and SSSI analyses together with those obtained using the ASCE 4–98 procedure.

Based on the above evaluations and the fact that the seismic SSI DGFOT analysis used the DM in SASSI2000 and was therefore unaffected by the numerical issues connected to the use of the SM in the SSI analysis, the staff accepted the DGFOT seismic design soil pressures for the STP Units 3 and 4 application. For additional details on the SASSI SM issues, see Subsection 3.7.2.4.20 later in this SER.

The design of the DGFOTs for axial tensile strain and seismic-induced forces at the tunnel bends due to SSE wave propagation is described in the response to Part 10 of **RAI 03.08.04-30** and is reviewed in Subsection 3.8.4.4.2, Subpart B.3.4 (Item a) of this SER.

3.7.2.4.5 Development of In-Structure Response Spectra

RAI 03.07.02-6 asked the applicant to provide the procedure used to develop the ISRS for the site-specific Category I structures in accordance with SRP Section 3.7.2, SRP Acceptance Criterion 5 guidance. The information should include but not be limited to the following:

1. Method for combining the three ISRS in a given direction and developed from separate analyses of the three directions of the input motion

2. Frequency increments for the ISRS calculation

3. Spectrum smoothing and broadening to account for uncertainty in soil and structural parameters

In the response to **RAI 03.07.02-6 (ML092370556)**, the applicant states that the ISRS in a given direction was obtained by combining the three co-directional ISRS developed from separate analyses of the three directions of input motion using the SRSS method. The applicant adds that the frequency increment for calculating the ISRS was either smaller than or the same as the frequency increment specified in Table 1 of RG 1.122; the ISRS were peak broadened by ±15 percent. The applicant provides this information in COL FSAR Subsection 3H.6.5.2.5. The staff reviewed the response per the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 5 and found it acceptable.

3.7.2.4.6 Three Components of Earthquake Motion

RAI 03.07.02-7 asked the applicant to provide the procedure used for combining the responses due to three components of earthquake motion for the site-specific Category I structures, in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 6. The applicant was also asked which of the acceptable methods in RG 1.92 were used to analyze the site-specific Category I structures.

In the response to **RAI 03.07.02-7 (ML092370556)**, the applicant states that the total structural responses (accelerations, displacements, and forces) were calculated by combining the co-directional responses due to the three components of earthquake motion through either the SRSS method or the 100-40-40 percent spatial combination rule. The applicant provides this information in COL FSAR Subsections 3H.6.5.2.6 and 3H.6.5.1.1.2. The staff found the response acceptable per the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 6.

3.7.2.4.7 Combination of Modal Responses

In COL FSAR Subsection 3H.6.5.2.7 the applicant states that because a frequency-domain seismic analysis was performed, there were no modal responses to be combined. The staff agreed with this observation.

3.7.2.4.8 Interaction of Non-Category I Structures with Category I SSCs

ABWR DCD Subsection 3.7.2.8, which addresses the interaction of non-Category I structures with seismic Category I structures, is incorporated by reference into the COL FSAR with supplemental and COL license information included in COL FSAR Subsection 3.7.5.4. ABWR DCD Subsection 3.7.2.8 specifies three criteria and notes that all non-Category I structures must meet one of these three criteria. The staff required additional information to determine the implementation of these criteria for non-Category I structures with the potential to interact with seismic Category I structures. Therefore, the staff issued **RAI 03.07.02-1** requesting the following:

a) A figure or table that includes the identification and location of each Category I and non-seismic Category I structure, including the height of each structure and the separation distance between structures.

b) The specific criteria of ABWR DCD 3.7.2.8 that each non-Category I structure is designed to meet.

c) A description of how non-Category I structures with the potential to interact with seismic Category I structures are evaluated for their sliding and overturning potential (including the coefficient of friction used and its basis) during an SSE. Also, the calculated factors of safety against sliding and overturning for the applicable non-Category I structures.

d) A statement of whether or not any non-Category I structures are designed to meet Criterion (2) of ABWR DCD 3.7.2.8 and, if so, the technical basis for determining that the collapse of a non-Category I structure will not compromise the integrity of a seismic Category I structure.

In the response to Item (a) of **RAI 03.07.02-1(ML092610377)**, the applicant provides Figure 3.7-38 (incorporated later as Figure 3.7-40 in the COL FSAR) showing the locations and separation distances of seismic Category I structures and non-Category I structures with the potential to interact with seismic Category I structures. The applicant states that the design is still in progress and some of the information in this figure may change. Notwithstanding the potential for change, the applicant will ensure that any changes will not create a potential interaction between seismic Category I and non-Category I structures using the criteria described in ABWR DCD Subsection 3.7.2.8. The staff found the response acceptable.

In the response to Item (b) of **RAI 03.07.02-1**, the applicant states that each non-Category I structure was designed to meet either Criterion (1) or (3) of ABWR DCD Subsection 3.7.2.8. The applicant also states that, based on the current progress of the design planning and engineering, five non-Category I structures (per unit)—the TB, RWB, SB, CBA, and plant stack— are anticipated to require enhanced seismic design and analysis to satisfy Criterion (3) of ABWR DCD Subsection 3.7.2.8. Item 1 of **RAI 03.07.02-13** asked the applicant to include in COL FSAR Subsection 3.7.2.8 the five identified non-Category I structures that could potentially interact with Category I structures. In the response to Item 1 of **RAI 03.07.02-13 (ML100550613)**, the applicant revises COL FSAR Subsection 3.7.2.8 to include this information. The staff found the response acceptable.

In the response to Item (c) of **RAI 03.07.02-1**, the applicant states that the sliding and overturning potential as a result of an SSE could not be evaluated, because the design planning and engineering specifically associated with non-Category I structures that could potentially interact with seismic Category I structures has not yet progressed to that point. However, as identified in SRP Section 3.7.2, SRP Acceptance Criterion 8, the staff needed to review the applicant's seismic design for these non-Category I structures. Therefore, Item 2 of **RAI 03.07.02-13** asked the applicant to provide factors of safety against sliding and overturning (including the basis of the coefficient of friction used in the analysis) during an SSE for the TB, RWB, SB, CBA, and plant stack. In the response to Item 2 of **RAI 03.07.02-13**, the applicant states that the plant stack is an integral part of the RB roof. Therefore, the design of the stack on the RB roof is covered under the certified design of the RB, and the calculation of the stability safety factor is not applicable to the plant stack. With respect to other non-seismic Category I structures, the applicant states that for the II/I evaluation of the TB, the seismic input motion is the site-specific SSE. For the RWB, SB, and CBA, the seismic input motion is the amplified site-specific SSE that considers the effects from the nearby heavy RB and CB structures. Because the TB is a relatively heavy structure, the staff did not expect the nearby RB to affect the response of the TB. Therefore, the staff concluded that the use of the site-specific SSE as input for II/I evaluation of the TB is acceptable. The development of the amplified ground motions for the seismic SSI analysis of the TB, RWB, SB, and CBA for II/I evaluation is discussed below. Subsection 3.8.4.4.3, Part C of this SER reviews the design for the required factor of safety against sliding and overturning and the basis for the coefficient of friction for the TB, RWB, SB, and CBA.

In the response to Item (d) of **RAI 03.07.02-1**, the applicant states that no non-Category I structures proposed as part of STP Units 3 and 4 are designed to meet Criterion (2) of ABWR DCD Subsection 3.7.2.8. Rather, as stated above in the response to Item (b), each non-Category I structure is designed to meet either Criterion (1) or (3) of ABWR DCD Subsection 3.7.2.8. This information is incorporated into Subsection 3.7.2.8 of the COL FSAR. The staff found the response acceptable and in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 8.

Development of Amplified Seismic Input Motions for II/I Stability Evaluations

Turbine Building

The applicant used the site-specific SSE as seismic input for the TB. The input spectra are in COL FSAR Figures 3A-231a and 3A-232a. As discussed earlier, the staff found the use of the site-specific SSE as seismic input for the TB acceptable because being a heavy structure, the seismic input for the TB is not expected to be significantly affected by the surrounding buildings.

Radwaste Building

For the stability evaluation of the RWB, the applicant describes in COL FSAR Subsections 3.7.3.16 and 3H.3.5.3 the method for establishing the seismic input. In addition, Figures 3.7-44 through 3.7-46 depict the seismic input. The earthquake input used at the foundation level is the envelope of site-specific SSE response spectra and the induced acceleration response spectra due to a site-specific SSE that is determined from an SSI analysis, which accounts for the impact of the nearby RB. In this SSI analysis, five interaction nodes at ground surface are added to the 3-D SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the site-specific SSE spectra to determine the SSE input at the foundation level. The staff found the method acceptable, because the seismic input for the RWB considers the effect of the nearby RB.

Service Building

For the SB stability evaluation, the SSE input (see COL FSAR Figures 3.7-53 through 3.7-55) is the envelope of the site-specific SSE response spectra and the induced acceleration response spectra due to the site-specific SSE that is determined from an SSI analysis, which accounts for the impact of the nearby CB. In this SSI analysis, five interaction nodes at the ground surface are added to the 3-D SSI model of the CB. These five interaction nodes correspond to the four corners and the center of the SB foundation. The average response of these five interaction nodes is enveloped with the site-specific SSE spectra to determine the SSE input at the foundation level. The staff found the method acceptable, because the seismic input for the SB considers the effect of the nearby CB.

Control Building Annex

For the CBA stability evaluation, the SSE input (see COL FSAR Figures 3.7-47 through 3.7-49) is the envelope of the site-specific response spectra and the induced acceleration response spectra due to the site-specific SSE, which is determined from an SSI analysis that accounts for the impact of the nearby CB. In this SSI analysis, five interaction nodes at the depth corresponding to the bottom elevation of the CBA foundation are added to the three dimensional SSI model of the CB. These five interaction nodes correspond to the four corners and the center of the CBA foundation. The average response of these five interaction nodes is enveloped with the site-specific spectra to determine the SSE input at the CBA foundation level. The staff found the method acceptable, because the seismic input for the CBA considers the effect of the nearby CB.

Development of Amplified Seismic Input Motions for II/I Design

To develop the SSE seismic input for II/I design (see COL FSAR Subsection 3.7.3.16), the applicant used a method similar to the one described above for establishing the seismic input for the stability evaluations for the TB, RWB, SB, and CBA. However, with the exception of the TB, the SSE input at the foundation level is the envelope of 0.3g RG 1.60 response spectra and the induced acceleration response spectra due to the site-specific SSE, which is determined from an SSI analysis that accounts for the impact of the nearby heavy buildings. For the TB design, the input motion is the 0.3g RG 1.60 spectra. This is considered acceptable because the seismic input for the TB (being a heavy structure) is not expected to be significantly affected by the nearby buildings. In addition, the seismic input for II/I design is significantly higher than the site-specific spectra. The staff reviewed the design spectra at the foundation level of the other relatively lighter non-seismic Category I structures noted above. The staff concluded that these design spectra inputs are acceptable because

- The induced acceleration motion at the foundation level of the II/I structures from the site-specific SSI analysis of the nearby heavy structures is further enveloped by the 0.3g RG 1.60 spectra, which are significantly higher than the site-specific SSE.

3.7.2.4.9 Effects of Parameter Variation on Floor Response Spectra

RAI 03.07.02-8 asked the applicant to provide the procedure and amount of peak broadening used to account for the effects of expected variations of structural properties (including the effect of potential concrete cracking on structural stiffness); damping values; soil properties; and SSI effects for site-specific Category I structures in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 9.

In the response to **RAI 03.07.02-8 (ML092370556)**, the applicant states that the envelope of the ISRS obtained for LB, mean, and UB soil properties will be widened by ±15 percent to account for all uncertainties, with the exceptions of soil property variations (that are already accounted for by considering the LB, mean, and UB soil cases) and potential concrete cracking. To account for concrete cracking in addition to other uncertainties, the ISRS will be developed with structural properties based on cracked concrete stiffness and mean soil properties. These spectra will be enveloped with spectra from the un-cracked concrete analysis and then widened by ±15 percent to obtain the final ISRS for the design. The applicant provides this information in COL FSAR Subsection 3H.6.5.2.9. The staff found the response acceptable per the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 9.

3.7.2.4.10 Use of Equivalent Vertical Static Factors

In COL FSAR Subsection 3H.6.5.2.7, the applicant states that since a separate seismic analysis was performed for the vertical direction, equivalent static factors were not used to define the vertical seismic responses. The staff agreed with this observation.

3.7.2.4.11 Methods Used to Account for Torsional Effects

RAI 03.07.02-9 asked the applicant to further clarify the procedure used to account for torsional effects (including how an accidental torsional moment at a particular location is calculated) in the seismic analysis of site-specific Category I structures, in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 11. The applicant was also asked to describe how the torsional effects will be combined with other seismic demands in the structure.

In the response to **RAI 03.07.02-9 (ML092510038)**, the applicant states that inherent torsion (i.e., torsion resulting from eccentricity between the center of mass and center of rigidity) is accounted for in the seismic analysis. The applicant further states that because the structural model in the SSI analysis of the UHS/RSW pump house is a detailed 3-D finite element model incorporating torsional degrees-of-freedom and eccentricities, the SSI analysis did not account for accidental torsion. The applicant provides additional details of how accidental torsion is computed in COL FSAR Subsection 3H.6.5.2.11. The staff reviewed this information and found it acceptable per the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 11.

3.7.2.4.12 Comparison of Responses

Subsection 3H.6.5.2.12 of the COL FSAR states that since only frequency-domain analysis is performed, no comparison of responses is presented.

The staff noted that seismic analyses of all site-specific Category I structures are performed using a frequency-domain method in SASSI2000. The results of SASSI2000 in terms of maximum accelerations, ISRS, seismic soil pressures, and member forces and moments were obtained and used for the design. To ensure that the results of SASSI2000 are accurate and/or conservative for the design, during the March 14 through March 18 (2011) audit of the STP Units 3 and 4 seismic calculations, the staff reviewed the results of the SASSI2000 V&V. **RAI 03.07.02-29** requested the applicant to expand the scope of the SASSI2000 V&V results with additional test problems, thus checking the accuracy of the finite elements as well as the SM used in SASSI2000 to calculate seismic responses. Subsection 3.7.2.4.16 of this SER discusses the staff's review of the SASSI2000 V&V.

Because SASSI2000 member forces and moments are also used as a design basis, during the September 2011 NRC audit of STP Units 3 and 4 the staff requested, as part of the response to **RAI 03.07.01-29**, that the applicant perform a confirmatory analysis to provide further assurance that the section cut forces from the SASSI2000 are accurate and/or conservative. The applicant performed a benchmark study that showed that the section cut forces from the SSI analysis using the SASSI2000 program are accurate or conservative (see the Supplement 1 response to **RAI 03.07.01-29 [ML113250374])**. The staff's review of the bench mark study which the staff found acceptable is discussed later in Subsection 3.7.2.4.21, “Confirmatory Analysis of SASSI2000 Section Forces,” of this SER.

In addition to the above V&V studies, the staff also performed an independent seismic SSI analysis of the UHS/RSW pump house using MTR/SASSI to confirm the results of the STP Units 3 and 4 analysis, to assess the sensitivity of the results to the use of a high Poisson’s ratio of 0.495, mesh discretization, and to assess the impact of the use of the SM on the results of the SSI analysis. The results of the staff’s confirmatory SSI analyses are discussed later in Subsection 3.7.2.4.17, “Confirmatory SSI Analysis,” and in Subsection 3.7.2.4.18, “Poisson’s Ratio Confirmatory Analysis,” in this SER.

Based on the applicant’s V&V and confirmatory studies, and the staff’s confirmatory SSI analysis described in Subsections 3.7.2.4.16 to 3.7.2.4.18 and 3.7.2.4.21, the staff concluded that the use of SASSI2000 for the seismic response analysis of the site-specific Category I structures is acceptable for the STP Units 3 and 4 application.

3.7.2.4.13 Analysis Procedure for Damping

In Subsection 3H.6.5.2.13 of the COL FSAR, the applicant states that the SSI analysis accounts for the structural and soil-damping described in Subsection 3H.6.5.1.2 of the COL FSAR. The staff found that the applicant’s description is consistent with SRP Section 3.7.2 acceptance criteria and is therefore acceptable.

3.7.2.4.14 Determination of Seismic Overturning Moments and Sliding Forces for Seismic Category I Structures

The staff issued **RAI 03.07.02-10** requesting the applicant to clarify the procedure used to determine seismic overturning moments and sliding forces for site-specific Category I structures, in accordance with the guidance in SRP Section 3.7.2, SRP Acceptance Criterion 14. The applicant was also asked to include in the COL FSAR the calculated factors of safety against overturning and sliding, the coefficient of friction used in the calculation, and its basis.

In the response to **RAI 03.07.02-10 (ML092610377)**, the applicant states that the UHS/RSW pump house and RSW piping tunnel are the only site-specific seismic Category I structures. The applicant further states that the RSW piping tunnel (with the exception of the access shaft) is a buried structure. To check the sliding and overturning of the UHS/RSW pump house, the following criteria were used:

1. Loads and load combinations are in COL FSAR Section 3H.6.4.5, with loads defined in Subsection 3H.6.4.3.4.1. These loads and load combinations are in accordance with SRP Section 3.8.5 criteria.

2. As described in COL FSAR Subsection 3H.6.5.1.1.2, seismic loads from three seismic excitations (i.e., x, y horizontal, and z vertical) are applied considering the 100-40-40 percent combination rule, as shown below:

   ±100% x-excitation ±40% y-excitation +40% z-excitation
   ±40% x-excitation ±100% y-excitation +40% z-excitation

   (Note: Positive z-excitation is upward. Also, ±40% x-excitation ±40% y-excitation ±100% z-excitation is not critical.)

3. Only 90 percent of the dead load is considered in determining the resisting forces and moments.

4. A friction coefficient of 0.30 is assumed. Additional information regarding the water proofing membrane is in the response to **RAI 03.08.04-5**.

5. Passive pressure is not utilized; only at-rest soil pressure is considered for calculating the resisting sliding forces and overturning moments.

6. Calculated factors of safety against overturning, sliding, and flotation will be provided later in the response to **RAI 03.07.01-13**.

7. Figure 1 in the response to **RAI 03.07.02-10** shows how stability safety factors are calculated.

Items 1, 3, 4, 6, and 7 are reviewed in Subsection 3.8.4.4.2, Subpart A.5.2 of this SER.

Regarding the review of Item 2 in the response to **RAI 03.07.02-10**, the seismic loads from input motions applied separately in three orthogonal directions should be combined following the procedure outlined in Section 2 of RG 1.92, Revision 2. According to RG 1.92, the absolute maximum values of the co-directional forces obtained from the three input motions should be sorted by decreasing value and added by applying 1.0, 0.4, and 0.4 factors to the component quantities. In compliance with this guidance, Item 1 of **RAI 03.07.02-18** asked the applicant to clarify whether the procedure used complies with the provisions of RG 1.92, Revision 2 for combining the effects of three spatial components of an earthquake and, if not, to verify that the method used is conservative. In the response to the question in Item 1 of **RAI 03.07.02-18 (ML100550613)**, the applicant states that the procedure outlined in Section 2 of RG 1.92, Revision 2 is only one of several possible combinations, and it has been captured in the procedure used by the applicant. Since the procedure complies with the provisions of RG 1.92, Revision 2 for combining the effects of three spatial components of an earthquake, the staff found the response acceptable.

Regarding the review of Item 5 in the response to **RAI 03.07.02-10 (ML092610377)**, the applicant states that passive soil pressure to resist the foundation sliding and overturning was not utilized. This assumption is conservative for determining the factor of safety against sliding and overturning. However, soil pressure should be considered in the design of the soil-retaining walls. In addition, the magnitude and distribution of the passive soil pressure will depend on the rigidity of the wall and the amount of wall displacement and/or rotation against the soil. Therefore, Item 2 of **RAI 03.07.02-18** asked the applicant to clarify how the passive soil pressure was calculated and considered in the wall design. In the response to this question, the applicant states that full passive soil pressure cannot be developed without significant structural movement (in excess of six inches). Because the UHS/RSW pump house is shown to have adequate factors of safety against sliding, overturning, and flotation, there will be no significant structural movement. Therefore, no significant passive pressure will be mobilized. In the response to RAI 03.07.02-18 (ML100550613), the applicant tabulates the enveloping maximum displacements relative to the input motion at free field at various locations of buried walls for the UHS/RSW pump house. The response shows the maximum calculated displacements to be only 0.56 cm (0.22 in.). Since the movement is very small, full passive pressure was not mobilized. The mobilized passive pressure was considered in the stability evaluation and the wall design. Evaluation of stability and the wall design is discussed in Section 3.8 of this SER.

3.7.2.4.15 Assessment of Interaction due to Seismic Effects

In response to COL License Information Item 3.22, the applicant states in COL FSAR Subsection 3.7.5.4:

> Nonsafety-related SSCs that are located in the same room as safety-related SSCs will be reviewed to determine if their failure will impact the ability of the safety-related SSC to perform its safety function. Non-seismic Category 1 SSCs whose failure could jeopardize the function of a safety-related SSC will be analyzed to demonstrate that structural integrity will be maintained in an SSE.

The staff's review of this information item is documented in detail in Subsection 3.7.3.4 of this SER.

3.7.2.4.16 Computer Programs Verification and Validation Issues

For SSE ground motions, Appendix S to 10 CFR Part 50 requires that SSCs remain functional and within applicable stress, strain, and deformation limits and that the evaluation take into account SSI effects. Criterion III, "Design Control," of Appendix B to 10 CFR Part 50 states in part that:

> …[m]easures shall also be established for the selection and review for suitability of application of materials, parts, equipment, and processes that are essential to the safety related functions of the structures, systems and components.

Additionally, Criterion III states in part that:

> "…the design control measures shall provide for verifying or checking the adequacy of design…"

SRP Section 3.8.1, SRP Acceptance Criterion 4.F specifies that computer programs used in the design and analysis should be described and validated.

During the October 18 through October 22 (2010) audit, the staff reviewed the V&V documents for SASSI2000, SAP2000, and SHAKE2000 software used in the seismic analysis of Category I structures. Several issues were identified regarding the adequacy of these documents. **RAI 03.07.02.29** asked the applicant to address these V&V issues, as discussed below.

<u>SASSI2000</u>

During the October 2010 audit, the staff reviewed Sargent and Lundy (S&L) and SGH Associates (SGH) V&V documents for three SASSI2000 versions (S&L SASSI2000-v3.0, SGH SASSI2000-v3.0, and SGH SASSI2000-v3.0-SGH), which were used for SSI and SSSI analyses in the COL applications. The V&V documents of the three programs did not adequately address all of the program features that were used to calculate and obtain maximum accelerations, acceleration response spectra, and dynamic soil pressures. In particular, the scope of the test problems did not address the adequacy of the following program features used in the COL applications:

1. General direction of load applications in the model

2. General orientation of the elements in the model

3. Accuracy of the triangular elements (solid, shell, and plane-strain) that may be used in the model

4. Acceptable aspect ratio of rectangular elements (solid, shell, and plane-strain) to obtain accurate results, as used in the model

5. Required mesh refinement to output out-of-plane responses in the shell elements

6. Accuracy of the SM for calculating foundation impedance

In addition, the staff was concerned about potential numerical instabilities resulting from the use of a high Poisson's ratio for modeling saturated soil behavior in SASSI2000, as the Poisson's ratio approaches 0.5. Therefore, the SASSI2000-v3.0 limitations—with respect to capping the Poisson's ratio to avoid potential numerical instabilities—need to be validated and stated.

Significant differences in the out-of-plane acceleration response of thick versus thin shell element models was also observed in the results of the analysis, with the thick shell model producing lower responses. This finding also needs to be further evaluated for SASSI2000-v3.0, with respect to the adequacy and limitations of the specific shell element types.

Without further demonstration that includes a validation of the program features discussed above for the COL applications, the staff could not determine that the programs used in the seismic analysis would not adversely affect the results of the SSI analysis and would not meet the applicable regulations. As such, **RAI 03.07.02-29** also requested a demonstration of the acceptability of SASSI2000, with additional test problems addressing the issues discussed above.

In the response to **RAI 03.07.02-29 (ML110250368)**, the applicant states that all STP Units 3 and 4 SSI and SSSI analyses were performed using SGH SASSI2000-v3.0 and/or S&L SASSI2000-v3.0, with the exception of the SSI analysis of the UHS/RSW pump house with a more refined mesh, as described in the Supplement 2 response to **RAI 03.07.02-24 (ML103550646)**, which used SASSI2000-v3.0-SGH. In the response to **RAI 03.07.02-29**, the applicant also states that Items 1 and 2 features of the computer programs were not used in the STP Units 3 and 4 application. Therefore, no additional test problems to validate these items would be performed. Regarding Items 3 through 6 and the validation of thick shell elements, the applicant expanded the scope of test problems to address the V&V issues identified above. Regarding potential numerical instabilities in SASSI2000 using a very high Poisson's ratio, the applicant did not perform any validation studies but stated that detailed cautionary checks were already specified on pages 3 through 5 of the existing S&L SASSI2000 release memorandum, which requires program users to review the transfer functions and results for any sign of instability. To ensure that the results of STP Units 3 and 4 SSI analyses using SASSI2000 are unaffected by potential numerical instabilities associated with the use of a high Poisson's ratio, the staff performed a confirmatory SSI analysis of the UHS/RSW pump house with a high Poisson's ratio to compare against the applicant's results. This confirmatory analysis is discussed in Subsection 3.7.2.4.18, "Poisson's Ratio Confirmatory SSI Analysis," later in this SER.

A public meeting was held on February 2 and 3 in 2011 to discuss the analyses referenced in the RAI response. As a result of this meeting, the applicant submitted a revision to the original response to **RAI 03.07.02-29** dated March 15, 2011 **(ML110770440),** which further expanded the scope of the V&V test problems to address the accuracy of SASSI2000 using thick shell elements with a two-way slab action.

NRC staff followed up with an audit from March 14 through March 18 in 2011, to review the results of the new V&V test problems provided in the revised response to **RAI 03.07.02-29,** which the staff found acceptable with the following two exceptions:

a) *Shell Element Aspect Ratio (Audit Action Item 3.7-28 from March 14 through18, 2011 NRC Audit – ML111260469)*

This issue involved obtaining different results from S&L SASSI2000-v3.0 and SGH SASSI2000-v3.0 using very similar test problems to address the shell element aspect ratio for two-way slabs. To address this discrepancy, the applicant reanalyzed the S&L aspect ratio test problem using the corresponding SGH model. Comparisons of the results are in the Supplement 1 response to **RAI 03.07.02-29** dated May 9, 2011 **(ML1131A131)**. A review of the new results indicates that the discrepancies previously observed between the two versions were due to modeling differences and are now resolved. Based on this resolution, the staff accepted the validation of the shell element aspect ratio.

b) *Subtraction Method*

This issue involved validation of the SM used in SASSI2000 to calculate foundation impedance for SSI and SSSI analyses. In reviewing the SGH SASSI2000 V&V for the SM, the staff found that the test problems were not analyzed to frequencies sufficiently high enough to validate the stability and accuracy of the SM for a passing frequency of V_s/5h, where V_s is the lowest shear wave velocity of the foundation media and h is the largest element size in the soil model. In the case of the S&L SASSI2000 V&V test problems used to validate the SM, the analyses were carried out to frequencies sufficiently high enough to cover the passing frequency requirement of V_s/5h. However, the conclusion did not address the stability and accuracy of the SM used in the context of the calculated results. In both cases, the applicant was asked to revisit the test problems and provide validation that adequately addresses the stability and accuracy of the SM in relation to the acceptable passing frequency of V_s/5h, which is used in the STP Units 3 and 4 analysis and design.

In the Supplement 1 response to **RAI 03.07.02-29**, the applicant compares the response transfer functions of a circular embedded foundation over an elastic halfspace with theoretical solutions to validate the accuracy and stability of the SM in SASSI2000. The response comparisons covered a frequency range corresponding to a_o values of less than 3, where a_o is a dimensionless parameter equal to $2\pi fr/V_s$, f is the frequency of analysis, r is the radius of foundation, and V_s is the shear wave velocity of foundation media.

In reviewing the above results, the staff noted that the site-specific seismic Category I structures at the STP Units 3 and 4 site, such as the UHS basin/RSW pump house, are generally large structures with significant foundation footprints. For example, the UHS basin has a foundation footprint of about 91.4m × 73.2 m (300 ft × 240 ft). Assuming an average shear wave velocity of about 305 m/s (1,000 ft/s) and a cutoff frequency of about 22 Hz used in the SSI analysis, the corresponding a_o value for this structure is about 20, while the test problems are carried out to a_o values of less than 3. Because the shape of the response transfer function is strongly dependent on the a_o value, the applicant was asked to extend the results of these test problems in terms of foundation compliance and scattering response functions, calculated using the SM, to a_o values of up to at least 8 to ensure the accuracy of the SM for analyzing large embedded structures. Subsequently, the applicant performed sensitivity studies using the DM and MSM to address the accuracy of the SM for the seismic SSI analyses of STP Units 3 and 4 structures. The results of the above sensitivity study, the accuracy of the SASSI2000 SM, and its implications for STP Units 3 and 4 applications are reviewed in Subsection 3.7.2.4.20 later in this SER.

<u>SAP2000</u>

SAP2000 was used to calculate forces, moments, and stresses for the design of the site-specific seismic Category I structures, such as the UHS/RSW pump house and RSW piping tunnel. The forces and moments were calculated by integrating stresses across design sections. The thick shell element in SAP2000 was used for modeling and designing the slabs. Mesh sensitivity studies were also performed using the time history modal superposition method of fixed-base structures to assess the adequacy of the structural mesh refinement for calculating accelerations and acceleration response spectra. To that extent, the SAP2000 V&V did not provide adequate validations for the following items:

1. Accuracy of forces and moments calculated from section cuts in shell models

2. Accuracy of thick shell elements for calculating out-of-plane dynamic responses

3. Accuracy of time history modal analyses of fixed-base structures modeled using shell elements

Therefore, to complete the resolution of the concerns in **RAI 03.07.02-29,** the applicant was asked to supplement the SAP2000 V&V with additional test problems to address the items discussed above. The staff needed this information to be able to conclude that the use of SAP2000 in STP applications would not adversely affect calculation of seismic forces and moments and the evaluation of SSI effects for Category I structures.

In the response to **RAI 03.07.02-29**, the applicant expands the SAP2000 Versions 10.1 and 14.1 validation documents to provide additional test problems to validate above Items 1, 2, and 3. In this response, the applicant validates the SAP2000 spectra generation used to address the V&V issues in Items 1, 2, and 3. The staff reviewed the results of these test problems during the audit conducted March 14 through 18, 2011. The staff found that a 10 percent tolerance was used for validating forces and 15 percent was used for the spectra (these are the acceptance criteria). The applicant was also asked to justify the large tolerance used to validate the forces and spectra calculations. In the Supplement 1 response to **RAI 03.07.02-29**, the applicant states that the higher difference in forces and spectra was mainly due to differences in SAP2000 and ANSYS element formulations. As a result of these differences, the generated acceleration time histories from the two programs were somewhat different. The applicant further notes that the (dynamic) forces and spectra generated from SAP2000 were only used for mesh sensitivity studies and not for the design. Therefore, the acceptance criteria above have no impact on the STP Units 3 and 4 designs. The staff noted that only static forces and moments obtained from SAP2000 were used in the STP Units 3 and 4 designs. Because the static forces and moments had been validated, the staff found the V&V of SAP2000 acceptable for the STP application.

SHAKE2000

SHAKE2000 was used to calculate the SSE-based foundation motions for the SSI analysis of the UHS/RSW pump house and other seismic Category I structures. The SHAKE2000 V&V only tested soil models with up to eight soil layers, while the STP profile is a deep soil site that was modeled using a large number of soil layers. Therefore, **RAI 03.07.02-29** asked the applicant to further demonstrate the acceptability of SHAKE2000 with additional test problems that check the use of large numbers of soil layers to encompass the STP soil site. The staff needed this information to be able to conclude that the SSE-based foundation motion determined from SHAKE2000 is adequate for the STP application and meets the requirements of Appendix S of 10 CFR Part 50.

In the response to **RAI 03.07.02-29**, the applicant expanded the SHAKE2000 Version 3.5 validation document to include six additional test problems with 116 soil layers to demonstrate the acceptability of this program when using large numbers of soil layers. The applicant also stated that all STP Unit 3 and 4 SHAKE2000 analyses were performed using 100 or fewer soil layers. These six test problems cover both the program's in-profile (within) and outcrop input-output options. Based on the results of the new test problems, which the staff reviewed during the March 14 through 16, 2011 NRC audit, the staff found the SHAKE2000 V&V acceptable for the STP application.

3.7.2.4.17 Confirmatory SSI Analysis

For SSE ground motions, Appendix S of 10 CFR Part 50 requires that SSCs remain functional and within applicable stress, strain, and deformation limits and that the evaluation take into account SSI effects and the expected duration of vibratory motion. The STP is a deep, non-uniform soil site modeled with a large number of soil layers in the SSI analysis. Any use of the SASSI-based code for the STP application must be carefully verified and validated for project-specific applications. The applicant identified a numerical instability in the original SSI analysis using ACS SASSI NQA Version 2.2.1. In light of this numerical instability with ACS SASSI, the applicant decided to use SASSI2000 Version 3.0 for the SSI analysis and performed additional project-specific validations for this SASSI version. However, to gain greater confidence and assurance in the results of the SSI analysis, the staff performed a confirmatory analysis as part of the review of COL FSAR Section 3.7 to verify the acceptability of the design-basis results of the STP SSI analysis of the UHS/RSW pump house. The staff therefore issued **RAI 03.07.02-21** requesting the electronic input data files and other necessary information to carry out this confirmatory analysis.

Discussion of Results:

Using the data and information in the applicant's response to **RAI 03.07.02-21 (ML100890620)**, the staff performed a confirmatory SSI analysis of the UHS/RSW pump house for the mean soil case. The soil model consisted of 100 soil layers with a fixed thickness over a uniform halfspace modeled with 10 layers of variable thickness. The SSI analysis was performed using the MTR/SASSI Version 8.2.04 (x64) computer program licensed from MTR & Associates. MTR/SASSI is an enhanced version of the original SASSI computer program developed at the University of California, Berkeley. The program modifications include additional capabilities to handle detailed structural models and large numbers of soil layers. The un-interpolated transfer functions in the STP SASSI analysis were calculated for 68 frequencies with the exception of one frequency (14.16 Hz) in the z-direction that was not included. Based on the calculated frequencies, the cutoff frequency is 20.26 Hz. For the confirmatory SSI analysis using

MTR/SASSI, the un-interpolated transfer functions were calculated for 300 frequencies spaced at a uniform interval of 0.0625 Hz. This calculation ensures that a sufficient number of frequencies are analyzed for stable interpolated transfer functions. The results of this analysis and other modeling details are in the report, "*Confirmatory Seismic SSI Analysis of UHS and RSW Pump House for Units 3 and 4 of the South Texas Project, Prepared for the U.S. Nuclear Regulatory Commission by SC Solutions, dated June 1, 2010.*" Conclusions of this study are summarized below.

Based on the results of the UHS/RSW pump house confirmatory SSI analysis, the potential numerical instability resulting from the large number of soil layers used in SASSI to model the deep soil profile at the STP Units 3 and 4 site did not affect the results of the SASSI2000 SSI analysis reported by the applicant. This observation is based on comparisons of the maximum accelerations, acceleration response spectra, and transfer functions calculated at several representative locations in the structure; including nodes on the basemat, roofs, and different wall elevations both in the middle and at the periphery of the UHS/RSW pump house structures. The large discrepancy between the MTR/SASSI and STP SASSI2000 results at the center of the pump house roof in the z-direction is believed to be due to inadequate mesh refinement, which is insufficient to capture the roof's out-of-plane response.

The study also noted that this conclusion is based on a comparison of the results at only a few selected locations in the structure. To better ascertain the calculated results, the contours of the maximum accelerations in the UHS/RSW pump house (as calculated by MTR/SASSI) were plotted and shown to have a consistent pattern. Similar results from the STP SASSI2000 analysis were not available for comparison.

Several observations with respect to the applicant's SASSI modeling of the UHS/RSW pump house include:

1. The SASSI model assigned no hydrodynamic masses to the submerged columns inside the UHS basin. Because of the relatively large surface area of these columns, their responses can be significantly affected by the omission of hydrodynamic effects.

2. The columns inside the UHS basin are integral to the basin basemat. The basemat was modeled with solid elements with three translational degrees-of-freedom per node. To provide moment transfer at the column/basemat connections, the beams were extended into the solid elements with rigid massless beams. However, there were two columns where the beams were not extended into the solid elements. These columns were essentially pin-connected to the basemat, which caused higher accelerations compared to the rest of the columns. The modeling of these columns needs to be verified to ensure the intent of the design.

3. The refinement of the finite element mesh of the UHS/RSW pump house basemat and sidewalls below grade in the SASSI model was incapable of providing accurate calculations of the responses affected by frequencies above 13.75 Hz. This is based on the maximum passing frequency, $f_{max} = V_s/(5h)$ where V_s is the minimum shear wave velocity of foundation support media (V_s = 167 m/s [550 ft/s]) and h is the largest horizontal/vertical dimension of soil elements (h = 2.4 m [8 ft]).

4. Similarly, the level of model refinement in the structure was not capable of (1) accurately capturing the out-of-plane dynamic response of walls and slabs in SASSI, or (2) adequately calculating the design-basis stresses in the shell elements.

5. In the STP SASSI2000 analysis, one frequency response in the vertical direction was omitted from the results. The basis for discarding this frequency response and any other frequencies from the results of the analysis should be justified.

The resolution of Items 1 and 2 is discussed earlier under the topic “Hydrodynamic Effects of Water in the UHS Basin” in Subsection 3.7.2.4.3, “Procedures Used for Analytical Modeling,” of this SER. The resolution of Items 3 and 4 is discussed under the topic “Model Refinement and Passing Frequency” in Subsection 3.7.2.4.3 of this SER. With respect to Item 5, the applicant was asked in Item (b) of **RAI 03.07.02-24** to clarify why some frequencies were excluded from the calculation of un-interpolated transfer functions in certain directions. For example, the frequency 14.16 Hz was not included in the z-response analysis for the mean soil case and 9.521 Hz was not included in the z-response analysis for the UB soil case. The applicant was asked to provide the basis for selecting frequencies of analysis for the calculation of un-interpolated transfer functions and the basis for excluding frequencies from such calculations. In the response to Item (b) of **RAI 03.07.02-24 (ML102630145)**, the applicant states that the results of transfer functions are reviewed for each direction separately and additional frequencies are added as needed to stabilize the transfer functions. Because of the large number of cases being analyzed, the analyst will often not go back and include these additional frequencies in other case runs. The applicant adds that although the set of frequencies in the analysis are not the same in every case, this is not a result of excluding some calculated frequencies from the analysis. Based on the above explanation supported by the results of the staff’s confirmatory SSI analysis of the UHS/RSW pump house, which included 300 calculated frequency responses before interpolation and without excluding any specific frequency in any direction, the staff found the response acceptable (i.e., the Item [b] question in **RAI 03.07.02-24** corresponding to the Item 5 clarification above was resolved).

3.7.2.4.18 Poisson’s Ratio Confirmatory Analysis

The UHS/RSW pump house is founded on a deep non-uniform soil site. Due to a high ground water table at the site, the measured compression wave velocity in the saturated soil layers is close to the compression wave velocity in water (i.e., 1,463 m/s [4,800 ft/s]). As a result of strong seismic shaking, the soil layers experience significant shear deformations that cause the shear modulus to degrade while the constrained modulus remains fairly unaffected. This situation then causes the Poisson’s ratio for the saturated soils to rise and approach the maximum value of 0.5. The applicant’s original SSI analyses arbitrarily capped the Poisson’s ratio at 0.48, resulting in a lower calculated compression wave velocity than the measured values for saturated soils. The staff was concerned that this might not allow the high frequency components of the vertical motion to be transmitted into the structure, possibly producing a less conservative response. On the other hand, when Poisson’s ratio approaches 0.5, the analytical results could be affected by numerical instability.

To address this issue, the applicant performed a sensitivity study of the CB using a higher Poisson’s ratio of 0.495, with an LB soil profile as part of an investigation into the soil parameter’s departure from the ABWR DCD. This sensitivity study (using Poisson’s ratios of 0.48 and 0.495) for the LB soil profile indicated that, there are significant differences in the calculated transfer functions as well as in the vertical responses obtained from the two analyses. For the case with the higher Poisson’s ratio, the transfer functions showed a number of narrow peaks (spikes) that did not exist in the original analysis. The applicant did not provide a satisfactory explanation for these differences.

To accept the results of the SSI analysis for the site-specific Category I structures, the staff performed a Poisson's ratio confirmatory analysis to verify the applicant's results of the SSI analysis of the UHS/RSW pump house. The staff needed to ensure that the results of the SSI analysis used for the seismic design of the UHS/RSW pump house and other site-specific Category I structures are unaffected by potential numerical problems from the use of a high Poisson's ratio. The applicant provided the required data for this study.

Discussion of Results:

The applicant originally performed all SSI analyses using a coarse SSI model of the UHS/RSW pump house, with Poisson's ratio capped at 0.48. The applicant then performed two additional cases with Poisson's ratio capped at 0.495 using a refined SSI model of the UHS/RSW pump house. The latter two cases were only analyzed for the UB in situ soil case (with empty and full basin models). The applicant then enveloped, smoothed, and broadened the results of all three cases to develop the design spectra.

For this confirmatory analysis, the staff performed three analyses that included the empty-basin model with UB and LB in situ soil cases and the full-basin model with LB in situ soil case —in all three cases the Poisson's ratio was capped at 0.495. These three cases primarily control the ISRS for the UHS/RSW pump house structures. The refined SSI model of the UHS/RSW pump house was used for this study. All analyses were performed with the MTR/SASSI Version 8.2.04 (x64) computer program using the MSM (i.e., all the nodes at the surface of the excavated soil model were included as interaction nodes).

The empty basin model was first analyzed for the UB in situ soil case with Poisson's ratio capped at 0.495. The applicant analyzed this same model using SASSI2000 and the SM. Results from the two analyses were then compared and benchmarked. Both the empty- and full-basin models were analyzed for the LB in situ soil case with Poisson's ratio capped at 0.495 (the applicant did not analyze these cases). The results for all three cases were then compared with the STP Units 3 and 4 design spectra.

The staff compared the results of the analysis in terms of the computed transfer functions and the 5 percent damped raw acceleration response spectra at several key locations in the structure with those calculated by the applicant. A detailed discussion of the results of the analysis, as well as a resolution of the differences observed between the staff's results and those obtained by the applicant, are in the report, "*Poisson's Ratio Confirmatory Analysis, STP Units 3&4 UHS/RSW Pump House, prepared for U.S. Nuclear Regulatory Commission by SC Solutions, dated July 2011.*" Conclusions of this study are summarized below:

- The results of this study show that response transfer functions calculated using a high Poisson's ratio of 0.495 are stable, well-behaved, and unaffected by numerical instability within the frequency range of interest for this structure. Potential inaccuracies observed at frequencies above 18 to 22 Hz are beyond the amplified area of the input motion and will not significantly impact the computed ISRS.

- The results of the UB in situ soil case with the empty basin envelop those of the LB in situ soil case with empty and full basins.

- The STP Units 3 and 4 design spectra envelop the raw spectra from the three cases analyzed in this confirmatory analysis.

Based on the above findings, the staff concluded that the use of a high Poisson's ratio in the applicant's SSI analysis does not adversely impact the STP Units 3 and 4 design spectra for the UHS/RSW pump house. The staff's review of the accuracy of the SM and MSM used in SASSI2000 to calculate the seismic SSI response of the UHS/RSW pump house is in Subsection 3.7.2.4.20, which is located later in this SER.

3.7.2.4.19 Assessment of Fluor's Part 21 Evaluation of Main Steam Line Seismic Input Requirements

In a letter dated August 30, 2010 (ML102530168), Fluor Enterprises, Inc. notified the NRC regarding an exceedance of the ABWR DCD seismic design input requirements for the main steam line (MSL) seismic analysis of the TB for STP Units 3 and 4, in accordance with 10 CFR Part 21.

ABWR DCD Tier 2, Subsection 3.2.5.3 states that:

> … [t]he main steam piping, bypass line, and condenser are used to mitigate the consequences of an accident and are required to remain functional during and after an SSE."

It requires that:

> Dynamic input loads for the design of the main steam lines in the turbine building are derived as follows: For locations on the basemat, the ARS *[Amplified Response Spectrum]* shall be based upon the Regulatory Guide 1.60 Response spectra normalized to 0.6g (i.e., 2 times ARS of the site envelope). For locations at the operating deck level (either operating deck or turbine deck), the ARS used shall be the same as used at the reactor building end of the main steam tunnel. Seismic Anchor motions shall be similarly calculated.

The applicant has taken a Tier 2 departure (STP DEP 1.2-2) from the ABWR DCD TB design that affects the dimensions of the STP Units 3 and 4 TBs. In the course of a re-design, Fluor performed a dynamic analysis and generated FRS for the STP Units 3 and 4 TBs. Fluor compared the STP TB FRS with the FRS specified in ABWR DCD Tier 2, Subsection 3.2.5.3 for input to the MSL seismic analysis. The comparison revealed that the FRS generated for the STP TB exceeded the FRS required by ABWR DCD Subsection 3.2.5.3 for MSLs. Fluor indicated that this issue is being addressed in the STP Units 3 and 4 design by using the conservative FRS generated during a detailed design.

In the context of the above information, the staff issued **RAI 03.07.02-30** requesting the following information regarding the seismic input used for the design of the MSL and other important safety SSCs in the TB:

1. Dynamic input loads (such as the FRS, anchor motions, etc.) for the design of the MSLs in the TB including a description of the site-specific TB dynamic analysis model; corresponding SSE inputs; and computer programs used in the analysis.

2. An update to the appropriate sections of the STP COL application including the applicable Departure Report in Part 7; design descriptions or commitments identified in ABWR DCD Tier 1, Section 2.15.11; and corresponding ITAAC for the TB as a result of

> the STP-specific FRS being higher than the FRS specified in ABWR DCD Tier 2, Subsection 3.2.5.3 for input to the MSL seismic analysis.

The staff needed this information to confirm that the design of the MSLs and other important safety SSCs in the TB appropriately considers the SSE design loads in combination with other appropriate loads as required by the ABWR DCD, and to ensure that the COL FSAR reflects the corresponding design basis.

In the response to **RAI 03.07.02-30 (ML110250368)**, the applicant states that the ABWR DCD does not provide a detailed design or design analysis for the TB. Instead, the ABWR DCD includes a general arrangement for the TB, which utilizes a reinforced concrete construction from the basemat to the operating deck and uses shear walls to provide lateral support. Additionally, ABWR DCD Subsection 3.2.5.3 provides requirements for the dynamic input to the MSL analysis of the TB.

In accepting the requirements of the ABWR DCD, NUREG–1503 (FSER related to the ABWR) states on pages 3-45 and 3-46 that

> The staff concludes that the dynamic input loads for the design of the MSLs [main steam lines] inside the turbine building are acceptable because (1) a comparison of the response spectra at the RB foundation level with the RG 1.60 response spectra anchored to 0.6g ZPA shows that the RG 1.60 response spectra anchored to the same 0.6g ZPA envelop the response spectra at the RB foundation level and (2) the turbine operating deck is located at approximately the same elevation as the anchor point of the main steam line at the RB side and the response spectra at the RB end were generated using an acceptable analysis approach as discussed…

The NRC's acceptance, as noted above, is not based on a detailed design of the TB. The acceptance simply indicates that the turbine operating deck should be located at approximately the same elevation as the anchor point of the MSL at the RB side.

Critical features of the TB seismic design, as described above in the ABWR DCD, have not been altered by the Departure STP DEP 1.2-2. With this departure, the TB still utilizes a reinforced concrete construction from the basemat to the operating deck and uses shear walls to provide lateral support. The turbine operating deck is located at approximately the same elevation as the anchor point of the MSL at the RB side. Therefore, the basis for the ABWR DCD requirements of the dynamic input to the MSL design is still valid for the TB design, while accounting for Departure STP DEP 1.2-2.

The applicant further states that the detailed design of the STP Units 3 and 4 TBs is in progress. Fluor's analysis was based on a work-in-progress design. The applicant did not accept the TB design used by Fluor or its seismic analyses. The applicant continues to believe that the dynamic input for MSL analysis, as specified in ABWR DCD Subsection 3.2.5.3, is valid and the detailed design of the TB can be developed to be consistent with those provisions without any inconsistency or deficiency. The applicant also states that ABWR DCD Tier 1, Section 2.15.11 does not include requirements for seismic inputs to the MSL analysis and design. Therefore, this section is not affected by Fluor's Part 21 evaluation (i.e., even if the Part 21 report were accepted, the ITAAC would continue to be appropriate as currently written). In implementing ITAAC 2.15.11 and ITAAC 2.10.1 (related to the main steam system design), a dynamic analysis of the TB will be performed to confirm that the ABWR DCD dynamic input requirements for the MSL are satisfied for the TB final design. If the applicant ever determines that ABWR DCD Tier 2, Subsection 3.2.5.3 is not appropriate as applied to the STP Units 3 and 4 TBs, the applicant would take a departure in accordance with Section VIII.B.5 of the ABWR design certification rule (i.e., 10 CFR Part 52, Appendix A).

However, based on a review of the above response and in light of the Fluor Part 21 evaluation, the staff concluded that the applicant has not demonstrated that the dynamic input loads for the design of the MSL in the TB would be conservative at the STP site. The staff also noted that the referenced ITAAC do not specifically require verification of the DCD dynamic input requirements for the MSL. Therefore, **RAI 03.07.02-31** requested the applicant to provide site-specific ITAAC delineating that a dynamic analysis of the TB will be performed to confirm that the DCD dynamic input requirements for the MSL are satisfied for the final design of the TB. In the absence of the results for a final dynamic analysis of the TB, the staff needs these ITAAC to ensure that the design of the MSLs and other SSCs important to safety in the TB appropriately considers the SSE design loads in combination with other appropriate loads, as required by the ABWR DCD.

In response to **RAI 03.07.02-31 (ML110900339),** the applicant adds new site-specific ITAAC in COL application Part 9 specifically delineating that a dynamic analysis of the TB will be performed to confirm that the DCD dynamic input requirements for the MSL are satisfied for the final design of the TB. Based on staff's review of the response and the addition of the site-specific ITAAC, the staff found the applicant's response to address the Fluor Part 21 notice on the MSL acceptable.

3.7.2.4.20 DNFSB SASSI Subtraction Method Issues

The DNFSB identified a technical issue in SASSI: when the SM is used to analyze embedded structures, the results may be non-conservative. Because the SM was used for the STP Units 3 and 4 SSI/SSSI analyses, the staff issued **RAI 03.07.01-29** requesting the applicant to demonstrate the acceptability of the SM and the results or to provide a plan and schedule to ensure that the SSCs are designed to meet GDC 2 requirements. The RAI requested the following:

1. For all STP Unit 3 and 4 seismic Category I structures compare the ISRS, structural loads, and other design response quantities developed using the SM with those developed using the DM or MSM and evaluate the differences.

2. Demonstrate and justify that the differences identified in Item 1 have no impact on the design of seismic Category I structures, or revise the design to address these differences.

3. If the MSM is used to validate the SM, provide a validation program for the MSM.

4. Provide the COL FSAR markup, if any, in the response to document actions taken to address the resolution of the DNFSB's issues with SASSI versions used in the STP 3 and 4 analyses.

The staff needs this information to ensure that the STP design-basis loads and the ISRS will envelop the corresponding ISRS generated from either the DM or MSM.

In the response to **RAI 03.07.01-29 (ML11168A168)** and the Supplement 1 response to **RAI 03.07.01-29 (ML113250374)**, the applicant states that the SASSI2000 program was used to perform seismic analyses for seismic Category I structures. These seismic analyses consisted of the:

- SSI analysis
- SSSI analysis

The results of the above seismic analyses were used for the:

- Determination of amplified site-specific motions for light structures considering the influence of nearby heavy structures

- Generation of the ISRS using the acceleration time histories from the SSI analyses

- Structural design and stability evaluations of structures using:

    1. Maximum nodal accelerations and section cut forces from the SSI analyses
    2. Soil pressures from the SSI and SSSI analyses

- STP Units 3 and 4 application, which used the SM for all SSSI and some SSI analyses; the results of these analyses were used to address the design of the RB, CB, UHS, RSW pump house, DGFOSV, DGFOT, and RWB

For the RB and CB, the results were compared to the ABWR DCD design values to ensure that the ABWR DCD design envelops the results of these analyses.

Considering the above, the applicant states that an initial plan to address the issues identified by the DNFSB with the SM in the SASSI2000 computer program was discussed with NRC staff during NRC's May 23 through May 27 (2011) audit. The initial plan was submitted in the response to **RAI 03.07.01-29 (ML11168A168)**. During the execution of the initial plan and based on the results of various analyses and additional feedback from the staff, the applicant expanded the scope of the initial plan to further address the impact of the SM on the results of the analysis. The results of the revised plan were submitted in the Revision 1 of Supplement 1 response to **RAI 03.07.01-29 (ML113360516)** and are discussed below.

Project-Specific Verification of the MSM

The applicant performed a project-specific verification of the MSM. In the previous SSI analysis in support of the shear wave velocity departure, the CB SSI analysis was performed using the DM analysis. For this verification, the applicant re-analyzed the CB using the MSM. The applicant then compared the results of the SSI analyses with the DM and MSM to verify the MSM results. These comparisons of various calculated response quantities are discussed below.

a) *Comparison of In-Structure Response Spectra*

The Supplement 1 response to **RAI 03.07.01-29** compares the ISRS computed from the MSM and DM at several nodes of the CB model. The staff reviewed the above comparisons and found that the ISRS computed from the MSM agree well with those from DM.

b) Comparison of Transfer Functions

The Supplement 1 response to **RAI 03.07.01-29** (ML113250734) compares transfer functions computed from the MSM and DM for several nodes of the CB model. The staff reviewed these transfer functions and concluded that the transfer functions obtained from the MSM and DM computations show excellent agreement up to a frequency of about 10-13 Hz. Beyond this frequency and up to the maximum frequency of 30 Hz, the transfer functions calculated using the MSM and DM show some differences. However, the staff concluded that these differences should not significantly impact the final design quantities because:

- The observed differences in the transfer functions between 10 and 30 Hz are small.

- The transfer functions are only intermediate results. Therefore, when they are convolved with the input motion, which does not contain significant energy beyond 21 Hz, the impact on the final calculated ISRS will be very small, as shown in the Supplement 1 response to **RAI 03.07.01-29**.

- The structural stresses are generally controlled by the frequency content of the response at frequencies below 8 to 10 Hz, where the transfer functions show excellent agreement between the MSM and DM.

- The impacts on the maximum acceleration responses are also very small (see the next item).

c) Comparison of Maximum Accelerations

The Supplement 1 response to **RAI 03.07.01-29** compares the maximum accelerations in the x-, y- and z-directions. The staff reviewed these tables and concluded that the maximum accelerations from the DM and MSM compare very well. The maximum difference is less than 4 percent.

d) Comparison of Forces

The Supplement 1 response to **RAI 03.07.01-29** compares axial forces, shear forces, and bending moments for the beam elements of the CB model. The staff reviewed these tables and concluded that the forces and moments from the DM and MSM analyses compare very well. The maximum difference is less than 2 percent.

The Supplement 1 response to **RAI 03.07.01-29** compares axial forces, in-plane shear forces, and in-plane bending moments (with respect to model axes of symmetry) along four sections of the exterior walls of the model. These comparisons are provided for each individual excitation as well as the SRSS of all three excitations. The staff reviewed these results and concluded that the resultant axial forces, in-plane shears, and in-plane moments for the DM and MSM compare well. The maximum difference is about 4 percent.

Based on the above comparisons, the staff concluded that the applicant has adequately addressed the STP Units 3 and 4 project specific verification of the MSM with interaction nodes comprised of those at the soil-structure interface and the nodes at the top of excavated soil elements for the above response quantities.

<u>Generation of In-Structure Response Spectra</u>

The applicant generated the ISRS from the SSI analyses for the following seismic Category I structures:

- RSW piping tunnels
- DGFOSV
- DGFOT
- UHS/RSW pump house

The applicant notes that no ISRS were generated from the SSSI analyses.

a) *RSW Piping Tunnels*

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant states that the ISRS for the RSW piping tunnels were generated using the acceleration time histories obtained from a 2-D SSI model. This SSI analysis used the DM. The applicant therefore states that no further investigation is required for the ISRS of the RSW Piping tunnels. The staff concurred with this conclusion.

b) *Diesel Generator Fuel Oil Tunnels*

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant states that the ISRS for the DGFOT were initially generated using the acceleration time histories obtained from 2-D SSI analyses using the DM. However, in these SSI analyses, SSI soil pressures were not obtained. In order to obtain SSI soil pressures, the applicant revised the SSI model and repeated the analysis using the DM. In addition, in this revised analysis the ground water level was increased to 8.5 m (28 ft) MSL. The applicant used the results from this revised SSI analysis to generate the revised ISRS and DGFOT design (see FSAR Figures 3H.7-31 and 3H.7-32). Therefore, the applicant states that no further investigation is required for the ISRS of DGFOT. The staff concurred with this conclusion.

c) *Diesel Generator Fuel Oil Storage Vaults*

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant states that the ISRS for the DGFOSV were initially generated using the acceleration time histories obtained from 3-D SSI analyses using the SM. The applicant repeated this analysis using the MSM. In addition, in this revised analysis the ground water level was increased to 8.5 m (28 ft) MSL. The applicant used the results of this revised SSI analysis to generate the ISRS and DGFOSV design. Therefore, the applicant states that no further investigation is required for the ISRS of the DGFOSV. Because the analysis shows that the impact of the MSM on the ISRS is minimal for frequencies above 13 Hz for the STP Units 3 and 4 site, as evaluated earlier under the topic "Project-Specific Verification of MSM," the staff concluded that the use of the MSM to generate the ISRS for the DGFOSVs is acceptable.

d) *UHS/RSW Pump House*

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant states that the ISRS for the UHS/RSW pump house were initially generated using the acceleration time histories obtained from 3-D SSI analyses using the SM. The applicant further states that based on studies performed for mesh refinements, these spectra were amplified at several locations to account for the impact of the SSI model and structural mesh refinements, as explained in the Supplement 2 response to **RAI 03.07.02-24** (ML103550646). To address the DOE issue, the applicant modified and re-analyzed the full and empty basin SSI models for the UB in situ soil (same soil case used for the investigation of the SSI mesh refinement) using the MSM. In addition, in these revised analyses, the ground water level was increased to 8.5 m (28 ft) MSL.

The Supplement 1 response to **RAI 03.07.01-29** compares transfer functions from the SM and MSM for the UB in situ soil case at several locations for both empty and full basin cases. The comparison shows smoother transfer functions for the MSM. This response also compares the 5 percent-damped acceleration response spectra determined from using both the SM and MSM for the UB in situ soil case for both empty and full basin cases.

The applicant compared the resulting ISRS for the UB in situ soil case from the SM and MSM for both full and empty basin cases. On the basis of these comparisons, the applicant developed spectra amplification factors (always higher than or equal to 1) to account for the impact of the MSM on spectra generation. These amplification factors for full and empty basin cases are included in the RAI response. In addition, this RAI response provides the spectra amplification factors to account for the impact of structural and SSI mesh refinements on full and empty basin cases, respectively. The product of the modification factors for the MSM and the factor that was determined based on the envelope of the modification factors for structural mesh and SSI model refinements are selected as the final spectra amplification factors. The applicant further states that the final UHS/RSW pump house ISRS are determined in a manner similar to that described in the Supplement 2 response to **RAI 03.07.02-24**, which used the spectra amplification factors in the Supplement 1 response to **RAI 03.07.01-29**. FSAR Figures 3H.6-16 through 3H.6-39 depict the revised UHS/RSW pump house ISRS. Therefore, the applicant states that no further investigation is required for the ISRS of the UHS/RSW pump house. The staff reviewed the above information and concluded that the spectra amplification factors developed by the applicant to account for the impact of the MSM on the UHS/RSW pump house ISRS are conservative and are therefore acceptable.

Structural Design

The applicant obtained the following response quantities from the SSI and SSSI analyses, which were used in the design of STP Unit 3 and 4 structures:

- Maximum accelerations and section cut forces from SSI analyses
- Seismic soil pressures from SSSI analyses
- Seismic soil pressures from SSI analyses

The following discussions address each of the above items for all affected structures.

a) *Maximum accelerations and section cut forces*

The applicant states that all maximum accelerations and section cut forces for the structural design were obtained from the SSI analysis. The STP Unit 3 and 4 structures utilizing maximum accelerations and/or section cut forces from the SSI analysis include the following structures:

- RSW piping tunnels
- DGFOTs
- DGFOSVs

- UHS/RSW pump house

a.1) RSW Piping Tunnels

In the Supplement 1 response to **RAI 03.07.01-29** the applicant states that the SSI analyses of the RSW piping tunnels were performed using the DM. Therefore, the applicant concluded that no further investigation was required for the structural design of the RSW piping tunnels regarding maximum accelerations and section cut forces from the SSI analyses. The staff concurred with this conclusion.

a.2) Diesel Generator Fuel Oil Tunnels

In the Supplement 1 response to **RAI 03.07.01-29,** the applicant states that the SSI analyses of the DGFOT were performed using the DM. Based on these analyses, the applicant concluded that no further investigation was required for structural design of DGFOT in regards to maximum accelerations and section cut forces from SSI analyses. The staff concurred with this conclusion.

a.3) Diesel Generator Fuel Oil Storage Vaults

In the Supplement 1 response to **RAI 03.07.01-29,** the applicant states that the latest (final) SSI analyses of the DGFOSV were performed using the MSM. Based on these analyses, the applicant concluded that no further investigation was required for the structural design of the DGFOSV regarding the maximum accelerations and section cut forces from the SSI analyses. Because the calculated maximum accelerations using the MSM and DM compare reasonably well for the STP Units 3 and 4 site, as discussed earlier under the topic "Project-Specific Verification of MSM," the staff concluded that the use of the MSM to calculate the maximum accelerations for the DGFOSVs is acceptable.

a.4) UHS/RSW Pump House

In the Supplement 1 response to **RAI 03.07.01-29** the applicant states that the existing design of the UHS/RSW pump house was based on the SSI results using the SM. In this design, the seismic loads were determined using a conservative equivalent static method. For this equivalent static method of analysis, the elements of the structure were subdivided into nine major groups, and each group was further divided into panels (subgroups) resulting in a total of 208 panels. For each panel, the accelerations were determined based on the average of the maximum accelerations for all nodes within the panel.

In order to assess the impact of the SM, the applicant repeated the SSI analysis of the UHS/RSW pump house for the UB in situ soil case for both full and empty basin cases using the MSM. The maximum accelerations for the UB in situ soil case for empty and full basin cases from the SSI analyses — which used the SM and MSM — were compared to assess the impact of using the SM on the structural design.

The Supplement 1 response to **RAI 03.07.01-29** presents the percentage change in the magnitude of the panel accelerations for the MSM versus the SM for all 208 panels of this structure. The response shows that the majority of panel accelerations based on the MSM are lower than those based on the SM. However, some panels also see increases in acceleration with the MSM.

Considering that the majority of accelerations are reduced, and the fact that the existing design is based on the conservative equivalent static method, affirms the applicant's conclusion that the existing design based on the results of the SSI analysis using the SM is adequate. However, to confirm this finding, the applicant performed additional assessments that are described below.

a.4.1) Evaluation of UHS/RSW Pump House Walls and Slab Panels:

To assess the cumulative effects of the changes in acceleration for a number of section cuts, the applicant calculated the percentage difference between the SSI forces and the SM and MSM analyses and compared the results to the available demand margin in the section cut forces resulting from the use of the equivalent static method. The Supplement 1 response to **RAI 03.07.01-29** provides the results of this assessment for full and empty basin cases. On the basis of these results, the applicant makes the following observations:

- In the majority of cases, the individual force components along a section cut are lower in the MSM analysis than the respective force components in the SM analysis.

- With the exception of one case (discussed in the following bullet), those cases where an individual force component along a section cut in the MSM analysis exceeded the corresponding force in the SM analysis, the increase was bounded by the available demand margin resulting from the use of the conservative equivalent static method.

- For Section Cut #61 of the full basin case, the increase of the in-plane moment was 59.9 percent. This increase exceeded the available demand margin of 57 percent for the applied in-plane moment. The applicant states that this difference of 2.9 percent is of no consequence, for the following reasons:

    1. This deficit was only applicable to the in-plane moment. For the remaining force components (i.e., axial force, in-plane shear, out-of-plane shear, and out-of-plane moments), the MSM yielded lower force components.

    2. The design of in-plane reinforcement is a function of four force components: the axial force, in-plane shear, in-plane moment, and out-of-plane moment. The minimum margins for the axial force, in-plane shear, and out-of-plane moment are 60, 124, and 134 percent, respectively.

    3. The available margins noted in the tables in this response are conservative, because they are only based on the applied loads and do not consider any additional margin based on a provided reinforcement versus a calculated required reinforcement. Generally, provided reinforcement is at least 5 to 10 percent more than the calculated required reinforcement.

Based on the above information, the applicant concluded that all wall and slab panels of the UHS/RSW pump house design based on the SSI analysis using the SM would be adequate for the resulting forces when using the MSM of analysis.

The staff reviewed the results of the MSM of analysis and found that the applicant's justification for the adequacy of the design was based on a comparison of the structural stress demand (section cut forces and moments) calculated from the SASSI2000 versus the equivalent static force method results. In other words, the applicant used the margin in the structural demand that was calculated directly from the SSI analyses with SASSI2000 versus the demand from the equivalent static analysis calculated from SAP2000 (i.e., the original basis for the design) to justify any observed increase in the structural demand between the SM and MSM of analyses. The staff discussed this issue with the applicant during the September 27 – 30, 2011 audit (ML113140513). The applicant was requested to provide additional assurance that 1) the section cut forces from the SASSI2000 are accurate; and 2) the SSI mesh in the SASSI2000 SSI model is adequately refined to produce accurate section cut forces. The applicant performed two confirmatory analyses and provided the results in the Supplement 2 response to **RAI 03.07.01-29** (ML11364A098). The staff's assessment of the applicant's results of the confirmatory analyses shows that the existing design, which is based on the SM, is also adequate for the seismic demand determined from the SSI analysis using the MSM. The staff's evaluation of the applicant's confirmatory analyses is discussed in Subsection 3.7.2.4.21, "Confirmatory Analysis of SASSI2000 Section Forces," of this SER.

a.4.2) Evaluation of UHS Basin Columns and Beams:

The design of concrete beams and columns within the UHS basin is not only affected by the maximum accelerations, it can also be affected by the resulting ISRS at the top and bottom of the columns, which are used to account for the effect of hydrodynamic mass on the UHS columns. On the basis of the above information, the applicant provides a procedure described in the Supplement 1 response to **RAI 03.07.01-29** for assessing the UHS basin beams and columns. The applicant provides the results of this assessment in the Supplement 4 response to **RAI 03.08.04-30** (ML11181A002). On the basis of this assessment, the applicant states that all UHS basin concrete beams and columns that were designed based on the SSI analysis using the SM are adequate for the results of the SSI analysis using the MSM. The staff's review of this assessment is discussed in Subsection 3.8.4.4.4, (Item a) of this SER.

b) Seismic Soil Pressures

In the Supplement 1 response to **RAI 03.07.01-29,** the applicant states that the structural designs as well as the stability evaluations for the following STP Units 3 and 4 structures consider the seismic soil pressures from the SSSI and SSI analyses:

- RSW piping tunnels
- DGFOTs
- DGFOSVs
- UHS/RSW pump house
- RWB

The applicant addresses the impact on the stability evaluations in the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232), which is reviewed in Subsection 3.8.4.4.4, (Item c) of this SER.

b.1) Seismic Soil Pressures from the SSSI Analysis

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant states that the main purpose of the SSSI analyses was to determine the increase in soil pressures on the walls facing the adjacent structure from the adjacent structure. The RAI response describes the seven SSSI analyses using the SM (see Figure 03.07.01-29S1.79 in the Revision 1, Supplement 1 response to RAI 03.07.01-29 [ML113360516] for the locations of the sections).

Based on the discussions with NRC staff during the May 2011 NRC audit of STP Units 3 and 4 (ML12346A389), the applicant selected the RB + RSW Piping Tunnels + RWB model described in the RAI response for further evaluations of the dynamic soil pressures resulting from SSSI effects using the DM and MSM of analysis. The results of this evaluation were then used to address the SSSI results from the SM.

The bounding SSSI soil pressures for this SSSI analysis using the SM were determined considering the LB in situ, UB in situ, and UB backfill soil cases. The Supplement 1 response to **RAI 03.07.01-29** shows the governing soil case for soil pressures for each of the six walls in this SSSI model. The applicant made the following observations based on these results:

- Most (~ 85 percent) of these soil pressures are governed by the LB in situ and UB backfill soil cases

- Only a small percentage of soil pressures are governed by the UB in situ soil case. In these cases, the soil pressures from the UB in situ soil case are in general within 10 percent of those from the LB in situ or UB backfill soil cases.

Based on the above observations, the applicant conducted a re-analysis of this SSSI section using the DM and MSM with only the LB in situ and UB backfill soil cases. The staff reviewed the above comparisons and concluded that the LB in situ soil and UB backfill soil cases were adequate to address the soil pressure difference between the SM, MSM, and DM.

The results of the re-analysis for this SSSI section using the SM, MSM, and DM are reviewed below:

b.1.1) Investigation of SSSI Section 6 for Lower Bound In Situ Soil Cases

The following evaluations are based on the results from the SSSI analysis of Section 6 LB in situ soil cases using the SM, MSM, and DM.

b.1.1.1) Comparison of Maximum Absolute Soil Pressure Profiles

The Supplement 1 response to **RAI 03.07.01-29** compares the maximum absolute soil pressures in the SM, MSM, and DM of analysis for all walls in this model. These pressures represent the absolute value of the highest magnitude soil pressure experienced by each element during any point in the time history of the analysis. Based on the above results, the applicant makes the following observations:

- The maximum absolute pressure profiles obtained from the SM and the MSM are comparable for all walls.

- For the exterior walls of the SSSI model (i.e., the RB east wall, RB west wall below the RWB mud mat, and RWB west wall), the maximum absolute pressure profiles using the DM are different from those using the SM and MSM.

- For the interior walls (i.e., walls that face an adjacent structure), the maximum absolute pressure profiles from all three methods are comparable.

b.1.1.2) Comparison of Transfer Functions for Soil Pressures

The soil pressures are determined from soil element forces, which are dependent on element nodal displacements. Because the nodal displacements are calculated using the acceleration transfer functions, the soil pressures are related to the nodal acceleration transfer functions for the soil element used to calculate the soil pressures. For a comparison of the transfer functions for soil pressures, the applicant selected a total of six elements along the RB east, RB west, and RWB west walls where there is a significant difference between the soil pressures using the SM versus the DM of analysis. The location of these six elements is shown in the Revision 1 of Supplement 1 response to **RAI 03.07.01-29** (ML113360516). The RAI response also compares the transfer functions of these six elements using the SM, MSM, and DM of analysis.

The Supplement 1 response to RAI 03.07.01-29 shows that for low frequencies (i.e., up to about 7 Hz), there is reasonably good agreement among the transfer functions in all three methods of analysis. For higher frequencies, some significant differences are seen among the transfer functions in the three methods. The applicant states that the differences at higher frequencies are not significant for the structural design, because soil pressures mainly reflect low frequency responses. To demonstrate this finding, the applicant compares the maximum absolute soil pressures for the six walls in this SSSI model, which were computed to consider the responses from 0 to 5 Hz and 0 to 33 Hz. This comparison shows that most of the maximum soil pressures at any location along these walls reflect the responses to the 0 to 5 Hz frequencies.

The Supplement 1 response to **RAI 03.07.01-29** provides a more detailed comparison of the transfer functions with 0 to 5 Hz frequencies for these six elements, in addition to three more elements along the east and west walls of RSW Piping tunnel. The results show good agreement among the transfer functions in the SM, MSM, and DM of analysis for all nine elements.

b.1.1.3) Comparison of Total Soil Force Time Histories

The Supplement 1 response to **RAI 03.07.01-29** compares the total soil force time histories for all of the walls in this model using the SM, MSM, and DM of analysis. The results also show the magnitudes of the maximum total soil forces, the times corresponding to the maximum total soil forces, and the locations of the maximum total soil forces with respect to the grade elevation. Based on the above comparisons, the applicant states that the time history shapes of total soil forces acting on the walls are quite similar, and the differences in the magnitudes of the peaks and valleys are small.

b.1.1.4) Comparison of Soil Pressure Profiles

The Supplement 1 response to **RAI 03.07.01-29** also compares the soil pressure profiles for the maximum total soil forces, magnitudes of the maximum total soil forces, the times corresponding to the maximum total soil forces, and the locations of the maximum total soil forces with respect to the grade elevation for all of the walls in this model. Based on the above results, the applicant makes the following observations:

- The maximum total soil forces from the three methods of analysis are almost the same (i.e. within ±10 percent)

- For the same time, not only are the total soil forces from the three methods close (i.e. within ±10 percent), but the locations of the total soil forces from the three methods also are close. To demonstrate this point, the applicant compared instantaneous soil pressure profiles of the RB east and west walls for two equal times in the RAI response.

Based on the above observations, the applicant further states that the magnitudes and locations of the total soil forces obtained from the three methods of analysis are close (i.e., within ±10 percent).

b.1.2) Investigation of SSSI Section 6 for Upper Bound Backfill Soil Cases

The following discussion is based on the results from the SSSI analyses of the Section 6 UB backfill soil cases using the SM, MSM, and DM of analysis.

b.1.2.1) Comparison of the Maximum Absolute Soil Pressure Profiles

The Supplement 1 response to **RAI 03.07.01-29** compares the maximum absolute soil pressures of all walls in this model using the SM, MSM, and DM of analysis. Based on the above comparisons, the applicant makes the following observation:

- The maximum absolute pressure profiles in all three methods of analysis are comparable for all of the walls.

b.1.2.2) Comparison of Total Soil Force Time Histories

The Supplement 1 response to **RAI 03.07.01-29** compares the total soil force time histories for all of the walls in this model using the SM, MSM, and DM of analysis. This RAI response also shows the magnitudes of the maximum total soil forces, the times corresponding to the maximum total soil forces, and the locations of the maximum total soil forces with respect to the grade elevation. Based on this comparison, the applicant states that the time history shapes are quite similar, and the differences in the magnitudes of the peaks and valleys are small.

b.1.2.3) Comparison of Soil Pressure Profiles

The Supplement 1 response to **RAI 03.07.01-29** compares the soil pressure profiles for the maximum total soil forces, the magnitudes of the maximum total soil forces, the times corresponding to the maximum total soil forces, and the locations of the maximum total soil forces with respect to the grade elevation for all of the walls in this model. Based on the above comparison, the applicant states that:

- The maximum total soil forces obtained from the three methods of analysis are almost the same (i.e., within ±10 percent).

- For the same time, not only are the total soil forces obtained from the three methods close (i.e., within ±10 percent), but the locations of the total soil forces from the three methods are also close. To demonstrate this point, the applicant compares the instantaneous soil pressure profiles of the RB east and west walls for two equal times in the RAI response.

Based on the above findings, the applicant states that the magnitudes and locations of the total soil forces obtained from these three methods of analysis are close (i.e., within ±10 percent).

b.1.3) Conclusions Based on Investigation of SSSI Section 6

Based on the results presented above, the applicant considers the following conclusions to be applicable:

- The method of the SSSI analysis (SM, MSM, or DM) has a negligible impact on the total force due to seismic soil pressure (i.e., within ±10 percent).

- The method of the SSSI analysis (SM, MSM, or DM) has a negligible impact on location (i.e., the center of gravity [C.G.]) of the total force due to seismic soil pressure (i.e., within ±10 percent).

- The analytical results from the DM show some changes in the distribution of seismic soil pressures for the exterior walls.

- The method of the SSSI analysis (SM, MSM, or DM) has a negligible impact on the soil pressure distribution for the interior walls due to the fact that the interior walls, and the soil between them, collectively act as interior structural members and are not directly connected to the interaction nodes. Therefore, for the interior walls, there is no need for any additional investigation of the method of analysis that was used.

Because the total soil pressures and the point of application of the resultant soil pressure are not sensitive to the method of analysis used (i.e., SM, DM, and MSM), the staff found the above conclusions acceptable.

b.1.4) Evaluation of All SSSI Sections

The results from the STP Units 3 and 4 SSSI analyses were used to address the seismic soil pressures on the following structures:

- RB
- CB
- UHS/RSW pump house
- RSW piping tunnels
- DGFOSVs
- DGFOTs
- RWB

Among the above structures, the RB and CB are part of the certified design and, thus, the design of these structures was based on the 0.3g RG 1.60 response spectra. Therefore, for site-specific seismic input motions that are significantly lower than the 0.3g RG 1.60 response spectra, these structures possess a significant design margin. Therefore, the applicant has evaluated SSSI sections to assess the impact of the MSM analysis on the calculated soil pressures for the following structures:

- UHS/RSW pump house
- RSW piping tunnels
- DGFOSV
- DGFOT
- RWB

The staff agreed that not including the RB and CB in the above scope is appropriate because the impact of the MSM analysis on the calculated soil pressure will be bounded by the ABWR DCD seismic design pressures since the site-specific seismic motions are significantly lower than the ABWR DCD seismic input. The applicant's assessment and staff's evaluation of each SSSI section in the Supplement 1 response to **RAI 03.07.01-29** (ML11330516) is summarized below:

SSSI Sections 1 and 2

Section 1 is an E-W section analyzed by the applicant to address the crane foundation retaining wall (CFRW) effect on the RB. The CRFW is a non-seismic structure erected for construction but will be abandoned in place. Section 2 is also an E-W section analyzed by the applicant to address the CFRW effect on the CB. The applicant states that Section 1 does not include the DGFOT. Therefore, the applicant concluded that the results of the analysis for these two sections show no impact on the design of Category I site-specific structures, DGFOT, or the RWB. Therefore, no further investigation is required for these two sections. Based on review of Sections 1 and 2 SSSI models analyzed by the applicant, the staff verified that these Sections were not used to address any of the site-specific structures listed in the above scope. Because, these sections were only used for evaluation of the impact of the CFRW on the standard plant structures (RB and CB), the staff agreed with the conclusion that no further investigation is required for these two sections for the impact of the MSM.

SSSI Section 3

Section 3 is a N-S section of the TB + CB + RB. The applicant's confirmatory analysis evaluated the impact of site-specific input motions and soil properties on the ABWR DCD SSSI soil pressures. The applicant stated that the only SSSI soil pressures reported in the ABWR DCD are for the RB north wall and the CB south wall obtained from an SSSI analysis of RB + CB + TB. The applicant further stated that in this SSSI analysis the RB and CB walls are interior walls, so the method of analysis (SM, MSM, or DM) has a negligible impact on the total force due to seismic soil pressure or the distribution of seismic soil pressure and thus, no further investigation is required for this section. The staff reviewed the SSSI model depicted in Section 3 and verified that the RB north wall and the CB south wall traversed by this section cut can be considered as interior walls (inter-building walls separated by soil in between) and agreed with the conclusion that no further investigation is required for this section for the impact of the MSM.

SSSI Section 4

The applicant stated that this SSSI analysis is for the E-W CFRW + DGFOT + RB section. The walls of the DGFOT are interior walls, which are not impacted by the method of analysis. Therefore, no further investigation is required for this section. The staff noted that this section cut traversed through the CFRW, DGFOT, and RB. Because the impact of the MSM analysis on the calculated soil pressure on the RB will be bounded by the ABWR DCD seismic design pressures, the staff concluded that this section cut analysis will not affect the RB due to the method of analysis. The staff also reviewed the SSSI model depicted in Section 4 and verified that the walls of the DGFOT traversed by this section cut can be considered as interior walls (inter building walls separated by soil in between) and agreed with the conclusion that no further investigation is required for this section for the impact of the MSM.

SSSI Section 5

The applicant stated that this SSSI analysis is for the E-W CFRW + DGFOSV + DGFOT section. The east wall of the DGFOT and the east and part of the west walls of the DGFOSV are interior walls, which are not impacted by the method of analysis. The applicant also states that the DGFOSV walls—in addition to the SSSI and SSI soil pressures—are designed for the ASCE 4–98 seismic soil pressure as well as the full passive soil pressure (i.e., Kp = 3.0), which meets SRP Acceptance Criterion 4.H in SRP Section 3.8.4. Therefore, no further investigation is required for these walls. The staff reviewed the SSSI model depicted in Section 5 and verified that the east wall of the DGFOT and the east and part of the west walls of the DGFOSV are interior walls, which are not impacted by the method of analysis and agreed with the conclusion that no further investigation is required for this section for the impact of the MSM on the DGFOT east wall. In addition since the DGFOSV walls are designed for the full passive pressure which meets SRP 3.8.4 Acceptance Criterion 4.H, the staff agreed with the conclusion that no further investigation is required for this section for impact of MSM on the DGFOSV walls.

Because the west wall of the DGFOT is an exterior wall, the soil pressure distribution for this wall may be impacted by the method of analysis. The applicant states that this wall is designed for seismic soil pressures obtained from SSI, ASCE 4–98, SSSI (using the SM), as well as for a passive soil pressure. In the Supplement 1 response to **RAI 03.07.01-29**, the applicant compares the SSSI soil pressure with the seismic soil pressure used for the design of this wall. The response also compares out-of-plane shears and moments resulting from the SSSI and design seismic soil pressures for a typical panel of this wall. Based on the summary of the results in the RAI response, the applicant states that the minimum existing margin for the seismic soil pressure is 43 percent. The applicant stated that this margin will more than adequately account for any change in out-of-plane shear and moment due to seismic soil pressure distribution from the SSSI analysis using the DM. The staff reviewed the results presented in the response to RAI 03.07.01-29, Supplement 1, Revision 1. The staff agreed with this conclusion since the total seismic pressure load and the location of the resultant pressure load obtained using the SM, MSM, or DM remains approximately the same, and the effect of any variations in pressure distribution due to the method of SSSI analysis will be bounded by the existing margin.

<u>SSSI Section 6</u>

This SSSI analysis is for the E-W RWB + RSW Piping Tunnel + RB section. The original SASSI2000 SSSI analysis of this section used SM. The applicant subsequently reevaluated the analysis using the MSM and DM. Both the east and west walls of the RSW piping tunnel are interior walls, which are not impacted by the method of analysis. Therefore, the applicant concluded that no further investigation is required for these walls. The staff reviewed the SSSI model depicted in Section 6 and agreed with this conclusion.

The east wall of the RWB (a non-Category I structure) is an interior wall, which is not impacted by the method of analysis. The seismic soil pressure distribution for the west wall of the RWB is impacted by the method of the analysis. The Supplement 1 response to **RAI 03.07.01-29** compares the SSSI soil pressure with the seismic soil pressure used for the design of the west wall. The applicant stated that the SSSI soil pressure is from the DM of analysis. The RAI response shows the finite element model of this wall, which was used to determine out-of-plane shears and moments due to the SSSI and the design seismic soil pressures, and a comparison of contour plots for out-of-plane shears and moments. These comparisons show a minimum margin of 38 percent for the seismic soil pressure. The staff noted that for the evaluation of this section, DM of SSSI analysis was performed to verify that the seismic soil pressure demand used for the design is conservative. Since a DM of analysis was used for comparison purposes and a minimum margin of 38 percent for the seismic soil pressure exists, the staff found the seismic soil pressure used for the design to be acceptable.

SSSI Section 7

This SSSI analysis is for the N-S RB + DGFOSV No.1B + DGFOSV No.1C + RSW piping tunnel + UHS/RSW pump house section. The location selection of this SSSI section was based on discussions with the NRC staff during the August 2010 meeting in Rockville, Maryland (ML102250404). This SSSI section cuts through two DGFOSVs and was selected instead of another possible SSSI section to the east of it (which cuts through only one DGFOSV) for the following reasons:

1. For the section to the east of this section, the distance between the DGFOSV and the RB is in excess of 42.7 m (140 ft). Therefore, no significant impact from the RB is expected. The staff agreed with this assessment.

2. In addition to seismic soil pressures from the SSI and SSSI analyses, DGFOSV walls are designed for seismic soil pressures per ASCE 4–98 as well as for the full passive soil pressure (i.e., Kp = 3.0), which meets the guidance in SRP Acceptance Criterion 4.H in SRP Section 3.8.4.

3. The north wall of the RSW pump house is designed for seismic soil pressures obtained from the SSI and SSSI analyses, in addition to seismic soil pressures per ASCE 4–98 and the passive soil pressure (Kp = 1.2), without taking any credit for a lack of soil pressure at its juncture with the RSW piping tunnel.

In the Supplement 1 response to RAI 03.07.01-29, the applicant states that in this SSSI model the walls of the two DGFOSVs, the RSW piping tunnel, and the north wall of the UHS/RSW pump house (except for the bottom portion) are interior walls that are not impacted by method of analysis. The bottom portion of the north wall of the UHS/RSW pump house is exterior. However, the applicant stated that the north wall of the UHS/RSW pump house (in addition to the SSI and SSSI soil pressures) is designed for the seismic soil pressure per ASCE 4–98 and the passive soil pressure (Kp=1.2), without taking any credit for the lack of soil pressure at the juncture with the RSW piping tunnel. Therefore, no further investigation is required for these walls. The staff reviewed the SSSI model of the RB + DGFOSV No.1B + DGFOSV No.1C + RSW piping tunnel + UHS/RSW pump house (FSAR Figure 3H.6-211) and agreed with this conclusion since the seismic soil pressure on the interior walls are not affected by the SSSI analysis methods and the UHS/RSW pump house north wall is conservatively designed for the passive pressure without taking any credit for the lack of soil pressure at the juncture with the RSW tunnel.

The south wall of the UHS basin is an exterior wall. The applicant stated that while the distribution of the seismic soil pressure on the bottom 6.1 m (20 ft) (measured from the top of the basin basemat) of this wall may change due to the method of analysis, any change in the total force due to seismic soil pressure and the location (i.e., C.G.) of the resultant seismic soil force will be negligible. The applicant further states that this wall is designed to consider a conservative seismic soil pressure profile, which significantly envelops the seismic soil pressure from the SSSI analysis. The RAI response provides the contour plots for out-of-plane shears and moments due to the SSSI and the design seismic soil pressures based on the analysis of a typical section of this wall. Based on the summary results in the RAI response, the minimum existing margin for the seismic soil pressure is 243 percent. With this margin, the applicant states that the existing design of the wall more than adequately accounts for changes in the out-of-plane shear and moment due to seismic soil pressure distribution from the SSSI analysis using the DM. The staff agreed with this conclusion since the changes in the total seismic pressure load and the location of the resultant pressure load due to method of analysis is negligible, and any variations in pressure distribution due to the method of SSSI analysis will be bounded by the existing margin.

b.1.5) Conclusions Based on SSSI Investigations of All Sections

The applicant concludes that the existing STP Units 3 and 4 designs based on SSSI seismic soil pressures obtained from SSSI analyses using the SM will be adequate for SSSI seismic soil pressures obtained from SSSI analyses using the MSM and/or the DM.

Because the total soil pressures and the point of application of the resultant soil pressure are not sensitive to the method of analysis used (i.e., SM, DM, and MSM), the staff found the applicant's evaluation and resulting conclusions acceptable.

b.2) Seismic Soil Pressures from the SSI Analysis

The SSI seismic soil pressures from the STP Units 3 and 4 SSI analyses were used in the structural design of the following structures:

- RSW piping tunnels
- DGFOSV
- UHS/RSW pump house
- DGFOT

The evaluation of each of the above structures in regards to SSI seismic soil pressures is presented below:

b.2.1) RSW Piping Tunnels

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant compares the seismic soil pressures obtained from the SSI analysis based on maximum absolute pressures (conservative) with seismic soil pressures used for the design of the RSW piping tunnel east and west walls. The staff found that the SSI soil pressures were obtained from SSI analysis using the DM. Therefore, no further investigation is required for the SSI seismic soil pressures of the RSW piping tunnels.

b.2.2) Diesel Generator Fuel Oil Storage Vaults

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant compares seismic soil pressures obtained from the SSI analysis based on maximum absolute pressures (conservative) with seismic soil pressures used for the design of DGFOSV walls No. 1 through 4. The RAI response also provides wall numbering for the four walls of each DGFOSV. The SSI soil pressures are obtained from the SSI analysis using the MSM.

For each of the four walls of the DGFOSV, the applicant calculated the wall stresses using the SAP2000 to consider the SSI and design seismic soil pressures using the models in the RAI response. The RAI response presents the comparison of the contour plots for the resulting out-of-plane shears and moments due to the SSI with the design seismic soil pressures.

A summary of the above comparisons for the out-of-plane shears and moments of DGFOSV walls is in the RAI response. Based on the above comparison of shear and moments, the applicant concludes that there is a minimum margin of about 90 percent in the existing design for the seismic soil pressures obtained from the MSM of analysis.

For the DGFOSV, the applicant states that due to the size of the SSI model, obtaining the SSI seismic soil pressure using the DM of analysis is not feasible. However, from the investigation of SSSI soil pressures (described under topic b.1 above in this subsection), the SSI soil pressures from the DM are expected to have a soil pressure distribution that may differ from those obtained from the MSM of analysis. However, the total soil forces due to seismic soil pressures and the location (i.e., C.G.) of the resultant soil forces are expected to be similar to those obtained from the MSM, with only small variations (~10 percent). The applicant further notes that the margin for design seismic soil pressure shown above is about 90 percent. With this margin, the applicant concludes that the existing design will adequately account for any change in the seismic soil pressure distribution due to use of the DM.

Based on the above results, the staff found that the design of the DGFOSV using the design seismic soil pressures described above is adequate for the SSI seismic soil pressures, regardless of the method of analysis. Thus, no further investigation is required for the SSI seismic soil pressures of the DGFOSV.

b.2.3) UHS/RSW Pump House

The design-basis SSI analysis of the UHS/RSW pump house was originally performed using the SM of analysis. The applicant repeated the UHS/RSW pump house SSI analysis for the UB in situ soil case using the MSM.

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant compares SSI soil pressures (based on maximum of absolute soil pressures) from both the SM and MSM of analysis. Based on an examination of these comparisons, the applicant makes the following observations:

- The pressure profiles for the two methods are rather similar.
- The total soil force is close due to the seismic soil pressures obtained from the two methods. In addition, the total soil force from the SM of analysis either exceeds or is within 5 percent of the total soil force from the MSM.

The RAI response compares the design seismic soil pressure with the SSI soil pressures from the SM of analysis for all eight walls of the UHS/RSW pump house. These comparisons also show the total soil forces due to seismic soil pressures, as well as the location (i.e., C.G.) of the resultant total soil forces. The applicant concludes that the design seismic soil pressure considered for the design of the UHS/RSW pump house provides a minimum margin of about 220 percent against the SSI seismic soil pressures from the SM of analysis.

For the UHS/RSW pump house, the applicant states that due to the size of the SSI model, obtaining SSI seismic soil pressures using the DM is not feasible. However, from the investigation of the SSSI soil pressures (described under topic b.1 above in this subsection), the SSI soil pressures from the DM of analysis are expected to have a soil pressure distribution that may differ from that obtained from the SM. However, the total soil forces due to seismic soil pressures and the location (i.e., C.G.) of the resultant soil forces are expected to be similar to those obtained from the SM, with only small variations. The applicant states that as noted above, the minimum margin for the design seismic soil pressure is 220 percent. The applicant also states that this margin will adequately account for any change in the seismic soil pressure distribution from using the DM. Based on the above results, the staff found that the design of the UHS/RSW pump house using the design seismic soil pressures described above is adequate for the SSI seismic soil pressures, regardless of the method of analysis. Thus, no further investigation is required for the SSI seismic soil pressures of the UHS/RSW pump house.

b.2.4) Diesel Generator Fuel Oil Tunnels

In the Supplement 1 response to **RAI 03.07.01-29**, the applicant compares seismic soil pressures from the SSI analysis based on maximum absolute soil pressures (conservative) with the seismic soil pressures used for the design of the DGFOT walls. The SSI soil pressures were obtained from the SSI analysis that used the DM. These comparisons show that the seismic soil pressure profiles used for the design of these walls envelop the SSI seismic soil pressure profiles. Therefore, the staff concluded that no further investigation is required for the SSI seismic soil pressures of the DGFOT.

3.7.2.4.21 Confirmatory Analysis of SASSI2000 Section Forces

When addressing the adequacy of the walls and slabs of the UHS/RSW pump house for the cumulative effect of changes in the maximum accelerations from using the MSM of analysis, the applicant determined the percentage difference in the SSI forces for 19 section cuts of the UHS/RSW pump house obtained from both the SM and MSM analyses. The applicant then compared the percentage difference to the available margin in the section cut forces due to use of conservative equivalent static method.

During the September 2011 NRC audit of STP Units 3 and 4, the staff requested the applicant to perform two additional confirmatory analyses to provide further assurance that 1) the section cut forces from the SASSI2000 are accurate; and 2) the SSI mesh in the SASSI2000 SSI model is adequately refined to produce accurate section cut forces. The applicant performed these confirmatory analyses and provided the results in the Supplement 2 response to **RAI 03.07.01-29** (ML11364A098). The staff's review of the results of these studies is discussed below:

a) *Benchmark Study*

The purpose of the benchmark study was to show that the section cut forces from the SSI analysis using the SASSI2000 program were accurate or conservative. As noted in the Supplement 1 response to **RAI 03.07.01-29**, in order to benchmark the calculation of section cut forces from SASSI2000, the applicant repeated the dynamic analysis performed in SASSI2000 using the SAP2000 program with an identical input and a nearly identical model. The structural mesh of the models was identical to the so-called coarse mesh (also referred to as the original mesh) model used for the SSI analysis of the UHS/RSW pump house. The SAP2000 model was run as a fixed base, and the SASSI2000 model was modified by adding massless solid elements and fixing them at the base to simulate the fixed-base condition. Details of the SASSI2000 fixed-base modeling are in the Supplement 2 response to **RAI 03.07.01-29**. Other details of the dynamic analyses using SAP2000 and SASSI2000 are also in the same RAI response. It should be noted that only the full basin case was considered in this benchmark study.

In the Supplement 2 response to **RAI 03.07.01-29**, the applicant compares the section cut forces for the 19 section cuts from the SASSI2000 and SAP2000 fixed-base analyses. Based on this comparison, the staff concluded that in the majority of cases, the SASSI2000 results were higher than the corresponding SAP2000 results. For a few cases, the SASSI2000 results were lower than the SAP2000 results. For those panels where the SASSI2000 results were lower than the SAP2000 results, the applicant increased the corresponding section cut forces by the percentage difference. The staff's evaluation is in Subsection 3.8.4.4.4 (Item b) of this SER.

Based on the above results and the inherent difference between the two types of analyses (SASSI2000 based on frequency domain analysis and the SAP2000 based on the modal time history analysis), the staff concluded that the section cut forces calculated from the SASSI2000 analysis are conservative and reliable.

b) *Mesh Refinement Study*

The MSM analysis of the UHS/RSW pump house was performed using the original mesh (also called coarse mesh) that was used in the SM analysis. To examine the impact of mesh refinement on the SSI section cut forces, the applicant refined the fixed-base SSI model described in the benchmark study above to better approximate the more refined mesh used in the SAP2000 design model, which used the equivalent static method.

The applicant then analyzed the fixed-base SSI models with both the original mesh and refined mesh for the site-specific SSE ground motions that considered both the full and empty basin cases. The applicant combined the results from the three seismic excitations using the SRSS to determine section cut forces along the 19 section cuts. The Supplement 2 response to **RAI 03.07.01-29** compares the section cut forces for the full and empty basin cases. These comparisons show that in a majority of the cases, the section cut forces from the two meshes are within ±10 percent, except for some cases with higher percentage differences.

In order to account for the difference in the section cut forces due to the use of refined SSI mesh, the applicant increased the percentage difference in the SSI section cut forces obtained from the MSM of analysis by the percentage difference in the section cut forces with the use of refined mesh. This action was taken to obtain the total increase in the section cut forces resulting from the use of the MSM before comparing them to the available margins in the design section cut forces due to the use of the conservative equivalent static method. The Supplement 2 response to **RAI 03.07.01-29** provides the results of this comparison for the full and empty basin cases. Based on the applicant's comparisons of results in the response the staff found that in the majority of cases, the individual force components obtained from the MSM of analysis along a section cut are lower than the respective force components obtained from the SM of analysis. In those cases where the section cut forces obtained from the MSM of analysis exceed those obtained from the SM of analysis, the existing design margin is expected to accommodate the observed increase in section cut forces obtained from the MSM of analysis. Additional details of the comparison of the SM and MSM results are in the Supplement 2 response to **RAI 03.07.01-29**. The staff noted, however, that the applicant did not compare the out-of-plane moment for the roof panels calculated using the SM and MSM analyses. Therefore, the applicant was asked to provide further justification that the existing design of the UHS/RSW pump house roof slabs and operating floor are adequate for the resulting forces resulting from the use of the MSM of analysis. In the Supplement 3 response to **RAI 03.07.01-29** (**ML12103A369**), the applicant states that the existing design already accounts for the impact of the MSM analysis on the vertical accelerations used for the design of the RSW pump house roof and operating floor. FSAR Figures 3H.6-21 and 3H.6-24 show the vertical response spectra for the RSW pump house operating floor and roof, respectively. These response spectra were obtained from the SM SSI analysis and incorporate scaling factors from COL FSAR Table 3H.60-17 to compensate for the effect of mesh refinement and the MSM. The staff therefore found the spectra acceptable.

The staff concluded that the UHS/RSW pump house wall panels and slabs that were designed based on the results of the SSI analysis using the SM are adequate for the resulting forces due to a confirmation that used the MSM of analysis.

3.7.2.5 *Post Combined License Activities*

The applicant identifies the following commitments:

- Commitment (COM 3.7-2) – Develop a procedure to confirm that all nonsafety-related SSCs located in the same room as a safety-related SSC have been evaluated and correctly dispositioned for inspection of the as-built plant for II/I interactions. This will be developed in accordance with Section 13.5 and will be made available for inspection before fuel loading.

- Site-Specific ITAAC Table 3.0-18 to develop a dynamic analysis of the MSLs in the TB to generate ISRS.

3.7.2.6 *Conclusion*

The NRC staff's finding related to information incorporated by reference is in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the seismic system analysis that were incorporated by reference have been resolved.

The staff's review confirmed that the applicant has adequately addressed the Tier 1 departures relevant to this section in accordance with Section 3.7.2 of NUREG–0800.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations, the guidance in Section 3.7.2 of NUREG–0800, and other NRC RGs. The staff's review concluded that the applicant has adequately addressed COL License Information Item 3.22 and provided sufficient supplemental information to satisfy the NRC requirements and guidance in Section 3.7.2 of NUREG–0800.

3.7.3 Seismic Subsystem Analysis

3.7.3.1 *Introduction*

This FSAR section addresses the methods of the seismic analysis for ABWR seismic Category I SSCs that are not explicitly included in the structural models, when seismic analyses of the seismic Category I structures are performed. Such items include all seismic Category I cable trays and supports, conduits and supports, above-ground tanks, buried piping and tunnels, and structural elements that support other seismic Category I items (e.g., steel platforms, etc.). In addition, there is an evaluation of the seismic qualification of seismic Category I mechanical equipment, instrumentation, and electrical equipment that includes the seismic analysis of the piping systems.

The criteria and methods for the seismic analysis of safety-related SSCs and equipment include the following:

- Methods used for the seismic analysis
- Procedures used for analytical modeling
- Analytical procedures for damping
- Three components of earthquake motion
- A combination of modal responses
- Use of equivalent vertical static factors
- Buried seismic Category I piping, conduits, and tunnels
- Methods used for the seismic analysis of Category I concrete dams
- Methods used for the seismic analysis of above-ground tanks

3.7.3.2 *Summary of Application*

Section 3.7.3 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.7.3 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in COL FSAR Section 3.7.3, the applicant provides the following:

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

This departure updates information in FSAR Table 1.8-21 and replaces the International Code Council (ICC), “2006 International Building Code,” (IBC) for the International Council of Building Officials, “1991 Uniform Building Code” (UBC). This change incorporates the requirements of the Texas Building Code, which adopted the 2006 IBC.

COL License Information Items

- COL License Information Item 3.21 — Piping Analysis, Modeling of Piping Supports

In Subsection 3.7.5.3 of the ABWR DCD, the COL applicant is required to provide justification for methods used (other than those described in ABWR DCD Subsection 3.7.3.3.1.6) for determining pipe support stiffness in the piping analysis.

- COL License Information Item 3.22 — Assessment of Interaction Due to Seismic Effects

In Subsection 3.7.5.4 of the ABWR DCD, the COL applicant is required to describe the process for completing the design of the balance-of-plant and nonsafety-related systems to minimize II/I interactions and proposed procedures for an inspection of the as-built plant for II/I interactions. (COM 3.7-2)

- COL License Information Item in ABWR DCD Subsection 3.7.3.3.1.8 — Response Spectra Amplification at Support Attachment Points

To address the modeling of piping system supports, Subsection 3.7.3.3.1.8 of the ABWR DCD requires the COL applicant to ensure that the drywell equipment and pipe support structure (DEPSS) meet the criteria in ABWR DCD Subsection 3.7.3.3.4. If these criteria cannot be met, the COL applicant will generate the amplitude response spectrum (ARS) at piping attachment points that consider the DEPSS to be part of the structure using the methods of the dynamic analysis described in Section 3.7.2; or the COL applicant will analyze the piping systems that consider the DEPSS to be part of the pipe supports.

- COL License Information Item in ABWR DCD Subsection 3.7.3.3.1.7 Modeling of Special Engineered Pipe Supports

To address the modeling of specially engineered pipe supports, Subsection 3.7.3.3.1.7 of the ABWR DCD requires the COL applicant to ensure that modifications to the normal linear-elastic piping analysis methodology used with conventional pipe supports are required to calculate the loads acting on the supports and on the piping components when the specially engineered pipe supports are used as described in ABWR DCD Subsection 3.9.3.4.1(6).

Supplemental Information

As a result of Departure STD DEP 1.8-1 in COL FSAR Subsection 3.7.3.16, the applicant refers to the IBC and deletes references to the UBC. The applicant also deletes the ABWR DCD statement, "the seismic zone shall be Zone 3," and replaces it with "the seismic acceleration shall be the SSE ground acceleration."

3.7.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the seismic subsystem analysis, and the associated acceptance criteria, are in Section 3.7.3 of NUREG–0800.

In accordance with Section VIII, "Processes and Changes and Departures," of, "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies one Tier 2* departure. This departure requires prior NRC Approval and is subject to the requirements of 10 CFR 52 Appendix A, Section VIII.B.6.

Other regulatory guidance and design standards supporting the seismic design of safety-related SSCs and equipment include the following:

- NUREG–1061, "Report of the U.S. Nuclear Regulatory Commission Piping Review Committee," on the seismic analysis of piping

- ASCE 4–98 on the seismic analysis of safety-related nuclear structures and components

3.7.3.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.7.3 of the certified ABWR DCD. The staff reviewed Section 3.7.3 of the STP Units 3 and 4 COL FSAR Revision 7, and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

The staff reviewed the information in the COL FSAR:

The applicant has performed a site-specific SSI analysis of the seismic Category I structures, as discussed in Appendix 3A of the STP Units 3 and 4 COL FSAR, to confirm that the standard plant results included in the ABWR DCD will envelop the results of the site-specific SSI. The staff's evaluation of this analysis is in Section 3.7.2 of this SER.

The review of this application is limited to the COL license information items and the supplementary information in COL FSAR Section 3.7.3.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

The portion of this departure that applies to COL FSAR Subsection 3.7.3.16 for the design of non-seismic Category I structures updates Tier 2* information on codes and standards to refer to the 2006 IBC, deleting the 1991 UBC. This change incorporates the requirements of the Texas Building Code, which adopted the 2006 IBC. In addition, COL FSAR Subsection 3.7.3.16 states that for non-seismic Category I structures that are required to be designed to withstand a SSE, the seismic acceleration shall be the SSE ground acceleration. Since SSE ground accelerations will be used as seismic input, the staff concluded that the applicant's use of the IBC with the SSE ground acceleration as input to the seismic analysis of non-seismic Category I structures that are required to withstand a SSE is acceptable. The details of the staff's evaluation of the seismic methods of analysis and the design of non-Category I structures with a potential to interact with the Category I structures, including the use of the IBC instead of the UBC, are discussed under Tier 2* Departure STD DEP 1.8-1 in Subsection 3.8.4.4 of this SER.

COL License Information Items

- COL License Information Item 3.21 Piping Analysis, Modeling of Piping Supports

ABWR DCD Subsection 3.7.5.3 requires the COL applicant to justify the methods used, other than those described in ABWR DCD Subsection 3.7.3.3.1.6. In COL FSAR Subsection 3.7.5.3, the applicant indicates that the method described in ABWR DCD Subsection 3.7.3.3.1.6 will be used to determine pipe support stiffness. Since the applicant does not indicate the use of any methods other than those described in ABWR DCD Subsection 3.7.3.3.1.6, the staff found the applicant's response acceptable and no further justification is required.

- COL License Information Item 3.22 Assessment of Interaction Due To Seismic Effects

ABWR DCD Subsection 3.7.5.4 requires the COL applicant to describe the process for completing the design of the balance-of-plant and nonsafety-related systems to minimize II/I interactions and to propose procedures for an inspection of the as-built plant for II/I interactions. In response to this information item, the applicant in FSAR Subsection 3.7.5.4 states that nonsafety-related SSCs that are located in the same room as safety-related SSCs will be reviewed to determine whether their failure will impact the ability of the safety-related SSCs to perform their safety functions. The applicant also states that the non-seismic Category 1 SSCs whose failure could jeopardize the function of a safety-related SSC will be analyzed to demonstrate that structural integrity will be maintained in an SSE.

However, this description does not provide sufficient information for the staff to evaluate how the process for assessing interactions from a seismic effect will be implemented. Therefore, the staff issued **RAI 03.07.02-20** requesting additional details on completing this process.

In the response to **RAI 03.07.02-20** dated February 4, 2010 (**ML100480204**), the applicant provides additional information on the (a) process for completing the design of balance-of-plant and nonsafety-related systems to minimize II/I interactions; (b) criteria to be used for determining whether the failure of nonsafety-related SSCs will impact the ability of the safety-related SSCs to perform their safety functions; and (c) criteria to be used for demonstrating structural integrity of non-seismic Category I SSCs. Concerning item (a), the applicant indicates that nonsafety-related commodities—including their supports and support anchorages—are designed to preclude a failure under SSE seismic loading. Layout guidelines specify minimum seismic separation criteria between commodities. Concerning item (b), the applicant states that no criteria have been developed for determining the impact of the failure of nonsafety-related SSCs on safety-related SSCs because the nonsafety-related SSCs in the same room with safety-related SSCs are designed to preclude a failure under SSE seismic loading. Concerning item (c), the applicant states that:

- nonsafety-related piping and instrument lines inside any Category I structures will be designed to withstand any SSE event with pipe stresses limited to faulted allowable stresses

- support span criteria used for nonsafety-related cable trays, conduits, and HVAC ducts inside any Category I structure will be the same as for the safety-related SSCs

- supports for nonsafety-related piping, instrument lines, cable trays, conduits, and HVAC ducts inside any Category I structures will be designed for loads that include SSE loads and self-excitation loads during any SSE event

- anchorages for nonsafety-related commodity supports, equipment, and components inside any Category I structure will be designed for loads that include SSE loads

- within the Category I structures, both embedments and post-installed anchors are safety related and are designed to the requirements for Category I components, regardless of the classification of the component attached to the structure.

In addition, the applicant indicates that a procedure to confirm that all nonsafety-related SSCs located in the same room as a safety-related SSC were evaluated and correctly dispositioned for inspection of the as-built plant for II/I interactions will be developed. In the Supplement 1 response to **RAI 03.07.02-20** dated April 11, 2011 **(ML111050565),** the applicant provides a site-specific ITAAC (Table 3.0-19) in COL application Part 9 to verify that the as-built configuration is consistent with the Seismic II/I Interaction analysis.

The staff found the applicant’s responses to **RAI 03.07.02-20** acceptable in view of the following:

- The applicant’s proposed process for completing the design of the balance-of-plant and nonsafety-related systems to minimize II/I interactions is consistent with the standard engineering practice.

- Nonsafety-related SSCs and their anchorages in the same room with safety-related SSCs are designed to preclude failures under the SSE.

- The site-specific ITAAC on the Seismic II/I Interaction in COL application Part 9 provides reasonable assurance that a Seismic II/I Interaction analysis is performed and the as-built configuration is consistent with the Seismic II/I Interaction analysis.

The staff also confirmed that the applicant has made appropriate changes to COL application Part 9 and COL FSAR Subsection 3.7.5.4, and RAI 03.07.02-20 is therefore closed. The staff concluded that the applicant has adequately addressed COL License Information Item 3.22.

- COL License Information Item in ABWR DCD Subsection 3.7.3.3.1.8

ABWR DCD Subsection 3.7.3.3.1.8 indicates that if the DEPSS does not meet the criteria in ABWR DCD Subsection 3.7.3.3.4, the COL applicant will generate the acceleration response spectra at the piping attachment points and will consider the DEPSS as part of the structure using the methods of a dynamic analysis described in ABWR DCD Section 3.7.2; or the COL applicant will analyze the piping systems and will consider the DEPSS as part of the pipe supports. In COL FSAR Subsection 3.7.5.5, the applicant indicates that the acceleration response spectra at the piping attachment points are generated with the consideration that the drywell equipment and pipe support structure are part of the structure using the methods of a dynamic analysis, which are described in ABWR DCD Section 3.7.2. Since the applicant uses a method of analysis specifically identified in ABWR DCD Subsection 3.7.3.3.1.8, the staff found that the applicant has adequately addressed this COL license information item.

- COL License Information Item in ABWR DCD Subsection 3.7.3.3.1.7

ABWR DCD Subsection 3.7.3.3.1.7 indicates that when the specially engineered supports described in ABWR DCD Subsection 3.9.3.4.1(6) are used, additional information with regard to the modeling of these supports—including the information required by RG 1.84, "Design, Fabrication, and Materials Code Case Acceptability, ASME Section III"—should be included in the COL application. In COL FSAR Section 3.7.5.6, the applicant indicates that no specially engineered pipe supports described in ABWR DCD Subsection 3.9.3.4.1(6) will be used. Since the applicant indicates that no specially engineered supports described in ABWR DCD Subsection 3.9.3.4.1(6) will be used, the staff concluded that no additional information in this regard is required.

Supplemental Information

The applicant in COL FSAR Subsection 3.7.3.16 refers to the IBC and deletes the UBC. The applicant also deletes the ABWR DCD statement, "The seismic zone shall be Zone 3," and replaces it with "The seismic acceleration shall be the SSE ground acceleration." As discussed above under the evaluation of Departure STD DEP 1.8-1, the staff found these changes acceptable.

Departure STD DEP 1.2-1 establishes a new non-seismic Category 1 CBA adjacent to the CB. COL FSAR Subsection 3.7.3.16 specifies the analytical procedure for non-seismic structures. The IBC code and the SSE level ground acceleration is used to design non-seismic structures required to be designed to withstand any SSE. Because of the proximity of the RB, CB, and the TB to the CBA, the seismic response of the CBA may be affected by the surrounding buildings due to the structure-to-structure interaction effect. Therefore, the staff issued **RAI 03.07.03-01** requesting the applicant to address whether or not the effects of the structure-to-structure interaction are considered when establishing the SSE acceleration level at the foundation of the

CBA during an SSE event. If not, the applicant was asked to justify not including them and to explain what impact this omission could have on the seismic interaction evaluation of the CBA.

In the response to **RAI 03.07.03-01** dated September 15, 2009 **(ML092610377)**, the applicant indicates that the induced acceleration at the foundation level of the CBA during any SSE event may be amplified due to the presence of the nearby heavy structures, particularly due to its close proximity to the CB. The applicant states that the SSE input at the foundation level is the envelope of the 0.3g RG 1.60 response spectra and the induced acceleration response spectra due to a site-specific SSE, which is determined from an SSI analysis that accounts for the impact from the nearby CB. In this SSI analysis, five interaction nodes at the depth corresponding to the bottom elevation of the CBA foundation are added to the three dimensional SSI model of the CB. These five interaction nodes correspond to the four corners and the center of the CBA foundation. The average response of these five interaction nodes is enveloped within the 0.3g RG 1.60 response spectra to determine the SSE input at the CBA foundation level.

Since the seismic input to the CBA includes the effects from the presence of the adjacent CB, the staff found the applicant's response to **RAI 03.07.03-01** acceptable. However, the staff also issued **RAI 03.07.03-03** requesting the results of the SSI analysis described in the **RAI 03.07.03-1** response, together with the resulting seismic input motion for the CBA. In the Revision 1 response to **RAI 03.07.03-3** dated December 16, 2009 **(ML093520627)**, the applicant provides the requested information on the results of the SSI analysis. The staff noted that the new results in the Revision 1 response were slightly modified due to the revised SSI analysis. The staff reviewed the revised results and found them acceptable. The staff also confirmed that appropriate changes to COL FSAR Subsection 3.7.3.16 have been made. Therefore, the staff concluded that the applicant has adequately addressed this issue, and **RAIs 03.07.03-01** and **03.07.03-3** are closed.

The supplementary information in COL FSAR Section 3.7.3 does not include procedures for a seismic subsystem analysis of site-specific seismic Category I substructures (e.g., platforms, support frame structures, buried piping, tunnels, above ground tanks, etc.). Therefore, the staff issued **RAI 03.07.03-2** requesting the applicant to provide the seismic inputs and procedures used to analyze the site-specific subsystems per the guidance specified in SRP Section 3.7.3, with a level of detail comparable to ABWR DCD Section 3.7.3.

In the response to **RAI 03.07.03-02** dated September 3, 2009 **(ML092510038)**, the applicant states that the analysis and design of site-specific seismic Category I substructures (e.g., platforms, support frame structures, buried piping, tunnels, etc.) are in accordance with ABWR DCD Tier 2, Section 3.7.3, except that the site-specific SSE is used as seismic input. The applicant adds that there is no site-specific seismic Category I above the ground tank at the STP Units 3 and 4 site. Since the analysis and design procedure of site-specific seismic Category I substructures are in accordance with ABWR DCD Tier 2, Section 3.7.3, and the site-specific SSE is used as the seismic input, the staff found the applicant's response to **RAI 03.07.03-2** acceptable. The staff also confirmed that COL FSAR Subsection 3H.6.5.2.16 appropriately reflects the applicant's response. Therefore, **RAI 03.07.03-02** is closed.

3.7.3.5 *Post Combined License Activities*

The applicant identifies the following commitment:

- Commitment (COM 3.7-2) – Develop a procedure to confirm that all nonsafety-related SSCs located in the same room as a safety-related SSC have been evaluated and correctly dispositioned for inspection of the as-built plant for II/I interactions in accordance with Section 13.5 and to make them available for inspection before fuel loading

In addition, the applicant identifies the following site-specific ITAAC for II/I interaction verification:

- ITAAC Table 3.0-19, "Seismic II/I Interaction,"

3.7.3.6 *Conclusion*

The NRC staff's finding related to information incorporated by reference is in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the seismic subsystem analysis that were incorporated by reference have been resolved.

In addition, the staff compared the additional information in the COL application to the relevant NRC regulations, the guidance in Section 3.7.3 of NUREG–0800, and other NRC RGs. The staff's review concluded that the applicant has adequately addressed the COL information items and provided sufficient supplemental information in the COL application for developing seismic input for the analysis of non-seismic structures required to withstand any SSE and to assess Seismic Category II/I interaction effects. The staff found that this approach meets the guidance in Section 3.7.3 of NUREG–0800. The applicant also provides a site-specific ITAAC in COL application Part 9 to verify that the as-built configuration is consistent with the seismic Category II/I interaction analysis. The ITAAC provides reasonable assurance that nonsafety-related SSCs will not compromise the safety functions of the safety-related SSCs during any SSE.

3.7.4 Seismic Instrumentation

3.7.4.1 *Introduction*

This FSAR section addresses the installation of instrumentation that is capable of adequately measuring the effects of an earthquake at the plant site.

The criteria for the seismic instrumentation include the following:

- Comparison with RG 1.12, Revision 2, "Nuclear Power Plant Instrumentation for Earthquakes."

- Location and description of instrumentation.

- Control room operator notification.

- Comparison with RG 1.166, "Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Postearthquake Actions

- Instrument surveillance.

- Program implementation.

3.7.4.2 *Summary of Application*

Section 3.7.4 of the STP COL FSAR, Revision 9, incorporates by reference Section 3.7.4 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52, Appendix A, with no departures. In addition, in FSAR Section 3.7.5, "COL License Information," the applicant provides the following:

<u>*COL License Information Item*</u>

- COL License Information Item 3.20 Pre-Earthquake Planning and Post-Earthquake Actions

This COL license information item addresses the requirement to develop procedures for pre-earthquake planning and post-earthquake actions. The applicant commits (COM 3.7-1) to develop the needed procedures before fuel loading.

3.7.4.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is NUREG–1503. In addition, the relevant requirements of the Commission regulations for the seismic instrumentation, and the associated acceptance criteria, are in Section 3.7.4 of NUREG–0800. Specific requirements include:

- 10 CFR Part 50, Appendix A, GDC 2, "Design bases for protection against natural phenomena"
- 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants"

3.7.4.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.7.4 of the certified ABWR DCD. The staff reviewed Section 3.7.4 of the STP Units 3 and 4 COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the following information in the COL FSAR:

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

COL License Information Item

- COL License Information Item 3.20 Pre-Earthquake Planning and Post-Earthquake Actions

In Subsection 3.7.5.2, the applicant provides supplemental information to address COL License Information Item 3.20 and commits (COM 3.7-1) to develop procedures for pre-earthquake planning and post-earthquake actions.

DCD Subsection 3.7.5.2 directs the COL applicant to submit to the NRC as part of the COL application the procedures for pre-earthquake planning and post-earthquake actions. It states that these procedures shall implement the seismic instrumentation program that follow the guidelines recommended in EPRI Report NP-6695 with exceptions. The EPRI report reflects the guidelines described in RG 1.12, RG 1.166, and RG 1.167, “Restart of a Nuclear Power Plant Shut Down by a Seismic Event.” Therefore, NRC staff found the applicant’s response acceptable.

3.7.4.5 *Post Combined License Activities*

The applicant identifies the following commitment:

- Commitment (COM 3.7-1) – Develop the procedures for pre-earthquake planning and post-earthquake actions before fuel loading, in accordance with Section 3.7.4 and Section 13.5. The procedures will implement the Seismic Instrumentation Program specified in Section 3.7.4 and will follow the guidelines recommended in EPRI Report NP-6695, with the exceptions listed in Subsection 3.7.5.2 of the referenced DCD.

3.7.4.6 *Conclusion*

The NRC staff’s finding related to information incorporated by reference is in NUREG–1503. The staff reviewed the application and checked the referenced DCD. The staff’s review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to the seismic instrumentation that were incorporated by reference have been resolved.

The staff reviewed the supplemental information addressing COL License Information Item 3.20 and concluded that the applicant has provided sufficient information in accordance with the NRC requirements of Appendix S to 10 CFR Part 50.

3.8 Seismic Category I Structures

3.8.1 Concrete Containment

3.8.1.1 *Introduction*

This FSAR section addresses the design of the containment as a reinforced concrete cylindrical shell structure with an internal steel liner made of carbon steel, except for wetted surfaces where stainless steel or carbon steel with stainless steel cladding will be used. The containment is divided by the diaphragm floor and the reactor pedestal into an upper drywell chamber, a lower drywell chamber, and a suppression chamber. The containment is surrounded by and structurally integral to the RB through the RB floor slabs and the spent fuel pool structures. The containment is designed to resist various combinations of dead loads, live loads, and environmental loads including those resulting from wind, tornados, and earthquakes as well as from normal operating loads, and loads generated by a postulated LOCA. The design, fabrication, construction, and testing of the containment are in accordance with Subsection CC of the ASME Code, Section III, Division 2, "Code for Concrete Containments."

The criteria for the concrete containment design include the following:

- Description of the containment
- Applicable codes, standard, and specifications
- Loads and load combinations
- Design and analysis procedures
- Structural acceptance criteria
- Materials, quality control, and special construction techniques
- Testing and in-service inspection requirements

3.8.1.2 *Summary of Application*

Section 3.8.1 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.8.1 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in FSAR Section 3.8.1 and Appendices 3H, 3B, and 3G, the applicant provides the following:

Tier 1 Departures

- STP DEP T1 5.0-1 Site Parameters

The applicant identifies four specific departures of the site-specific parameters from generic site parameters used in the referenced ABWR DCD. The applicant revises the certified design site parameter for the site flooding from 30.5 cm (1 ft) below grade to 182.9 cm (6 ft) above grade, in order to satisfy a design-basis condition that assumes the main cooling reservoir failure for the STP site. The maximum design precipitation rate for rainfall increases from 49.3 to 50.3 cm/h (1.62 to 1.65 ft/h). The humidity at the site, as represented by the wet bulb temperature, increases from that specified in the DCD. In addition, the shear wave velocity is less than the 304.8 m/s (1,000 ft/s) minimum stated in the DCD. This departure has no separate impact on the design of the concrete containment other than through the RB, which surrounds the concrete containment. For a detailed discussion and evaluation of this departure, see Subsection 3.8.4.4 of this SER.

- STD DEP T1 2.15-1 — Re-classification of Radwaste Building Substructure from Seismic Category I to Non-Seismic

This departure revises the seismic category of the RWB substructure from seismic Category I to non-seismic. This departure has no effect on the concrete containment discussed in this section, other than an editorial change in the reference to the RWB. Subsection 3.8.4.4 of this SER evaluates this departure.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

This departure identifies Tier 2* items that are being updated to more current revisions or editions. Tables 1.8-20 and 1.8-21 of the COL FSAR show the new changes.

Tier 2 Departure Requiring Prior NRC Approval

- STD DEP 3B-2 — Revised Pool Swell Analysis

This departure updates the hydrodynamic loads analysis to incorporate a new method of analysis for the pool swell compared to the method described in the DCD.

Tier 2 Departure Not Requiring Prior NRC Approval

- STD DEP 3H-1 — Liner Anchor

This departure corrects the containment liner anchor material identified in FSAR Subsection 3H.1.4.4.3 to SA-36.

- STD DEP 3B-1 — Equation Error in Containment Impact Load

ABWR DCD Appendix 3B, Subsection 3B.4.2.3, provides two equations for calculating the pulse duration for a flat target. The multiplying factor used in one of the equations is incorrect because its dimensions are seconds per foot instead of seconds per meter, as required in this case. This departure corrects the multiplying factor from 0.0016 seconds per foot to 0.0052 seconds per meter. The change affects a multiplying factor for the correct application of units; it does not affect the structural design of the containment, because the correct loads are used for the structural analyses to show that the structures and components withstand the loads adequately without failures.

COL License Information Item

- COL License Information Item 3.25 — Structural Integrity Test Result

The applicant states that the “structural integrity test (SIT) of the containments will be performed in accordance with Subsection 3.8.1.7.1 and ITAAC Table 2.14.1, Item #3. The Unit 3 containment will be considered a prototype and its SIT performed accordingly." The details of the test and the required instrumentation for the test are in FSAR Subsection 3.8.6.3.

<u>Supplemental Information</u>

In FSAR Subsection 3.8.1.7.3, "Preservice and Inservice Inspection," the applicant describes the pre-service and ISI program requirements for ASME B&PV Code Class CC and MC pressure retaining components of the containment structure and their integral attachments. The applicant also describes those programs that implement the requirements of ASME B&PV Code Section XI (ASME Section XI), Subsection IWE, and Subsection IWL.

3.8.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the concrete containment, and the associated acceptance criteria, are in Section 3.8.1 of NUREG–0800.

In addition, in accordance with Section VIII, "Processes for Changes and Departures," of, "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies two Tier 1 departures and one Tier 2* departure. Tier 1 departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4. Tier 2* departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.B.6. Tier 2 departures not requiring prior NRC approval are subject to the requirements in Section VIII.B.5, which are similar to the requirements in 10 CFR 50.59.

The review and acceptability of the COL license information items are based on meeting the applicable acceptance criteria and guidance in SRP Section 3.8.1.

3.8.1.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.8.1 of the certified ABWR DCD. The staff reviewed Section 3.8.1 of the STP Units 3 and 4 COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

Tier 1 Departures

- STP DEP T1 5.0-1 — Site Parameters
- STD DEP T1 2.15-1 — Re-classification of Radwaste Building Substructure from Seismic Category I to Non-Seismic

These departures are evaluated in Subsection 3.8.4.4 of this SER.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

In FSAR Section 3.8, the applicant references Departure STD DEP 1.8-1. In this departure, the applicant updates the versions of several codes and standards. One change references ASME Code Section III, Division 2, 2001 Edition with the 2003 addenda and identifies certain limitations. The ABWR DCD specifies the use of the ASME Code, version 1989. NUREG-1503, page 3-49 specifies that any change to the use of the ASME Code (1989 Edition) for the design and construction of reinforced concrete containment structural elements requires NRC review and approval before implementation. In **RAI 03.08.04-33** and follow-up **RAI 03.08.04-36**, the staff requested the applicant to provide a detailed comparison of the differences between the code editions noted above, in order to demonstrate that the safety margins are not reduced and to evaluate any potential impact from the provisions in the newer codes that may be more restrictive or result in a more robust design. The evaluation of this departure is discussed under Tier 2* Departure STD DEP 1.8-1 in Subsection 3.8.4.4 of this SER.

Tier 2 Departure Requiring Prior NRC Approval

- STD DEP 3B-2 — Revised Pool Swell Analysis

In FSAR Appendix 3G, "Response of Structures to Containment Loads," the applicant states that the information in this section is incorporated by reference from the ABWR DCD. However, a review of Appendix 3B, "Containment Hydrodynamic Loads," Table 3B-1, "Pool Swell Calculated Values," indicates a significant increase in pool swell (PS) height and pressure loads for STP Units 3 and 4 compared to the loads reported in the ABWR DCD. Therefore, **RAI 03.08.01-5** asks the applicant to confirm that the results of the "Response of Structures to Containment Loads" reported in ABWR DCD Appendix 3G are unaffected by the containment hydrodynamic loads reported in Appendix 3B of STP Units 3 and 4 and are appropriately incorporated by reference. The applicant's response to **RAI 03.08.01-5** dated September 15, 2009 (ML092610377), states that the impact of changes in the loads on the internal structures of the containment from the increase in pool swell height and pressure will be addressed during the detailed design phase. The staff's evaluation noted that ABWR DCD Subsection 3H.1.5.5.2 describes the design of the containment internal structures, the load combination (including pool swell loads), and the analysis and design results, which are all incorporated by reference in FSAR Appendix 3H. Also, the pool swell loads are used in loading combinations for the design of the containment structure, and the analysis and design results for the containment structure are reported in Appendix 3H. The staff also noted that the impact of the changes in the loads from the increases in pool swell height and pressure on the concrete containment and the containment internal structures were not addressed in the response. Therefore, in

RAI 03.08.01-8, the staff asked the applicant to provide a quantitative evaluation confirming that the increased pool swell height and pressure will not have an adverse impact on the design of the concrete containment and the containment internal structures, and that it is appropriate to incorporate by reference the analysis of and design results for the containment and the containment internal structure reported in Appendix 3H of ABWR DCD. The staff reviewed the applicant's supplemental response to **RAI 03.08.01-8** dated April 14, 2010 (ML101090143), which concludes that the loads from increased pool swell height and pressure are enveloped by the condensation oscillation (CO) loads reported in the ABWR DCD. The response states that the design of the reinforced concrete containment vessel (RCCV) and the internal structures as described in ABWR DCD Section 3H.1.5 considered the selected load combinations, as determined in ABWR DCD Tables 3H.1-5a and 3H.1-5b. These load combinations considered the CO to be the controlling LOCA (CO, chugging [CH], or PS) loads, because these three LOCA loads do not occur simultaneously. For the containment and containment internal structures described in ABWR DCD Subsections 3H.1.5.5.1, 3H.1.5.5.2, and 3H.1.5.5.3, the load combinations including the CO + Pa loads [Pa is the containment pressure associated with the LOCA] governed over the load combinations including the PS + Pa loads. The response also includes a table comparing the pressures on the diaphragm floor, the RCCV wall, the reactor pressure vessel pedestal, and the basemat resulting from the revised PS + Pa loads with the DCD PS + Pa and CO + Pa loads, which confirm the applicant's conclusion. The staff found the applicant's response acceptable because the PS and CO loads do not occur simultaneously and the CO + Pa loads used in the ABWR design envelop the revised PS + Pa loads and thus govern the design. The response also includes a markup of Table 6.2-8 showing that the location of the RCCV penetrations will assure that the bottom of the penetration sleeve is above the revised pool swell impact zone (7,700 mm [275.6 in.]). In COL application Part 7, Section 2.3, the applicant revises the Departure STD DEP 3B-2 discussions to include a reference to Table 6.2-8. The FSAR was subsequently revised to incorporate these changes, and **RAI 03.08.01-5** and **RAI 03.08.01-8** are therefore closed. The detailed evaluation of Departure STD DEP 3B-2 is in Section 6.2 of this SER. The staff accepts the applicant's conclusion that the increased pool swell height and pressure will not have an adverse impact on the design of the concrete containment and the containment internal structures.

Tier 2 Departures Not Requiring Prior NRC Approval

- STD DEP 3H-1 Liner Anchor

This departure corrects the containment liner anchor material identified in Subsection 3H.1.4.4.3 to SA-36. ABWR DCD Tier 2, Subsection 3H.1.4.4.3 incorrectly identifies the containment liner anchor material as ASTM A-633, "Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates," Grade C, which is not an ASME Code allowable material. This change does not affect Tier 1, Tier 2*, the technical specifications, the bases for the technical specifications, or the operational requirements. The applicant evaluates this change pursuant to the requirements set forth in 10 CFR Part 52, Appendix A, Section VIII.B.5. The applicant states in this evaluation that DCD Tier 2, Subsection 3H.1.5.1 indicates that the liner anchors are considered rigid links, and they are not explicitly evaluated in the containment analysis. However, Appendix 19F of ABWR DCD Tier 2 includes an assessment of the containment liner and liner anchors. According to the Subsection 19F.3.2.1 analysis, the material properties used for the liner anchors are for ASTM A-36, "Standard Specification for Carbon Structural Steel." Permitted material in ASME SA-36 meets the requirements of ASTM A-36. Furthermore, Bechtel Topical Report BC-TOP-1 "Containment Building Liner Plate Design Report," Revision 1 issued December 1972, demonstrates that the A-36 liner anchor material is acceptable.

The applicant's evaluation determined that this departure does not require prior NRC approval in accordance with 10 CFR Part 52, Appendix A, Section VIII.B.5. Within the scope of the review in this section, the staff found it reasonable that the departure does not require prior NRC approval. The applicant's process for evaluating departures and other changes to the DCD is subject to NRC inspections.

- STD DEP 3B-1 Equation Error in Containment Impact Load

As stated above in the summary of the application, the correction of this error affects a multiplying factor for the correct application of units. The correction does not affect the structural design of the containment, because the correct loads are used for the structural analyses to show that the structures and components adequately withstand the loads with no failures.

The applicant's evaluation determined that this departure does not require prior NRC approval in accordance with 10 CFR Part 52, Appendix A, Section VIII.B.5. Within the scope of the review in this section, the staff found it reasonable that the departure does not require prior NRC approval. The applicant's process for evaluating departures and other changes to the DCD is subject to NRC inspections.

COL License Information Item

- COL License Information Item 3.25 Structural Integrity Test (SIT) Result

ABWR DCD Subsection 3.8.6.3 states that each COL applicant will perform a SIT on the ABWR containment in accordance with Subsection 3.8.1.7.1. Additionally, the first ABWR containment is considered as a prototype, and the details of the test and the instrumentation of its SIT will be provided by the first COL applicant for NRC review and approval. Because STP Units 3 and 4 represent the first such ABWR containment, the applicant needs to document the details of the SIT and the instrumentation for the test. The FSAR states that the details of the test and the required instrumentation will be provided to the NRC for approval. However, the FSAR does not include any details of the test, the required instrumentation, or a timeline for providing the information to the NRC for review. Therefore, **RAI 03.08.04-6** asked the applicant to include this information in the FSAR or to indicate when this information will be available for review. The RAI also asked the applicant to identify an appropriate tracking information to ensure compliance with RG 1.206, Regulatory Position C.III.4.3 such that the requirements in COL license information 3.25 are met.

The applicant’s response to **RAI 03.08.04-6** dated September 15, 2009 (ML092610377), provides a summary of SIT requirements for STP Units 3 and 4 based on Article CC-6000 of ASME Section III, Division 2. The applicant states that the details of the SIT and the instrumentation plan to be used (such as specific locations designated for recording displacements, strains, and temperature during the test) will be defined in the construction specifications. The STP Unit 3 primary containment vessel is classified as a prototype containment. Therefore, the test and instrument plan for the Unit 3 SIT will conform to the requirements for a prototype of containments as delineated in Article CC-6000 of ASME Section III, Division 2. The test and instrument plan for the Unit 4 SIT will conform to the requirements for non-prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2. However, the applicant does not indicate when the details of the test and instrumentation plan will be available to the NRC for review. Because COL License Information Item 3.25 requires the applicant to provide the details of the SIT and the instrumentation for

NRC review and approval, **RAI 03.08.04-27** asked the applicant to either provide the information for the staff to review or provide plans to meet the requirements of the COL license information item using the guidance in RG 1.206, Regulatory Position C.III.4.3.

The applicant's response to **RAI 03.08.04-27** dated February 10, 2010 (ML100550613), states that the details of the SIT and the instrumentation required for the test will be provided in the ASME Construction Specification. The applicant refers to RG 1.206, Regulatory Position C.III.4.3, Situation 4, for resolving the COL information item 6 months before the performance of the test. The staff's evaluation noted that according to RG 1.206, Regulatory Position C.III.4.3, the applicant should justify why the item cannot be resolved before the issuance of the license. Therefore, **RAI 03.08.04-32** asked the applicant to provide a detailed explanation to justify why any part or all of the information pertaining to the COL information item cannot be provided at this time and to clearly address all parts of the COL license information item. Also, the applicant was asked to confirm in Chapter 1 of the FSAR whether the COL information item could be completely resolved before the NRC issues the COL.

In the revised response to **RAI 03.08.04-32** dated March 7, 2011 (ML110730067), which superseded all earlier responses to **RAI 03.08.04-6, RAI 03.08.04-27,** and **RAI 03.08.04-32,** the applicant provides details of the Test and Instrument Plan for the SIT; the Unit 3 RCCV is still classified as a prototype containment. The applicant states that the Test and Instrument Plan for the Unit 3 SIT was developed to conform to the requirements of a containment prototype delineated in Article CC-6000 of ASME Section III, Division 2. In addition, the Test and Instrument Plan for the Unit 4 SIT will conform to the requirements for a containment that is not a prototype delineated in Article CC-6000 of ASME Section III, Division 2. The response provides a detailed description of the test and its objective; a description of the test parameters for both the pressurization and depressurization of the RCCV at 1.15 times the containment design pressure of 0.41 MPa (45 psig); and the test parameters for the differential pressurization/depressurization of the of drywell and the suppression chamber. The response also provides a detailed description of the instrumentation plan for the test and the evaluation of test results that includes the following items:

- Plans and elevations of the containment showing the proposed locations for measuring displacements and strains.
- Confirmation that the ranges selected for the instrumentation are consistent with the predicted deformation.
- Detailed description of how test results will be evaluated to ensure full compliance with the acceptance criteria in Subarticle CC-6400 of ASME Section III, Division 2 and in RG 1.136, *"Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,"* Revision 3..
- How and when crack mapping locations are determined.
- When the calculation for the predictive analyses will be performed.

The applicant also states that the results of the SIT will be included in a report which will meet the requirements of Sub-article CC-6530. The staff reviewed the applicant's response and found it acceptable, because the response adequately addresses COL License Information Item 3.25 by providing details of the SIT test and the instrumentation required for the test that are in accordance with the requirements of ASME Section III, Division 2, Article CC-6000. The applicant's response also includes markups of FSAR Subsection 3.8.6.3 that adequately describe details of the SIT and the proposed instrumentation plan, in addition to a plan for evaluating the test results and recording them in a report. The proposed markups to FSAR Subsection 3.8.6.3 were subsequently incorporated into a later revision of the FSAR and **RAI 03.08.04-6,** follow-up **RAI 03.08.04-27,** and **RAI 03.08.04-32** are therefore resolved.

Supplemental Information

10 CFR 50.55a(b)(2)(vi) states that "licensees may use either the 1992 Edition with the 1992 Addenda or the 1995 Edition with the 1996 Addenda of Subsection IWE and Subsection IWL as modified and supplemented by the requirements in paragraphs (b)(2)(viii) and (b)(2)(ix) of this section when implementing the initial 120-month inspection interval for the containment inservice inspection requirements of this section." In addition, RG 1.206 and; Regulatory Positions C.III.1 and Section C.I.5.2.4.1 define the ISI Program as an operational program as described in SECY-05-0197, "Review of Operational Programs in a Combined License Application and General Emergency Planning Inspections, Tests, Analyses, and Acceptance Criteria," so that the program and its implementation milestones will be fully described in terms of the scope and the level of detail that will allow for a finding of acceptability.

During the review, the staff noted that the ABWR DCD states that the containment ISI requirements are in Subsection 3.8.1.7, whereas none are addressed in this section of the FSAR. In addition, Section 6.6 of the DCD does not address ISI of the containment structure. Similarly, the staff noted that the program describing containment ISI is not addressed in either of the STP FSAR Sections 3.8.2 or 6.6. Therefore, in **RAI 03.08.01-11**, the staff asked the applicant to discuss the Containment ISI Program in either of these sections in sufficient detail for the staff to obtain a reasonable assurance of the acceptability of the ISI inspections for the containment.

In the response to **RAI 03.08.01-11** dated November 15, 2011 (ML113250040), the applicant provides FSAR markups of Subsection 3.8.1.7.3 which describes the scope of containment Preservice and ISI Program requirements of the ASME B&PV Code, Class CC and Class MC pressure-retaining components of the containment structure and their integral attachments. The applicant states that the programs implement the requirements of the ASME Boiler and Pressure Vessel (B&PV) Code Section XI (ASME Section XI), Subsections IWE and IWL. Subsection IWE of ASME Section XI applies to Class MC components and metallic shell and penetration liners of Class CC pressure retaining components and their integral attachments. Subsection IWL of ASME Section XI applies to the Class CC reinforced concrete containment structure.

The applicant also states that the Preservice and ISI Program plans are based on ASME Section XI, Edition and Addenda per the requirements of 10 CFR 50.55a. The actual Edition of ASME Section XI to be used is specified based on the procurement date of the component per 10 CFR 50.55a. The containment structure is designed to provide access for the examinations required by ASME Section XI, IWE-2500 and IWL-2500. The applicant also includes details of the ASME Code requirements based on 2004 Edition of ASME Section XI, and the supplemental requirements provided in 10 CFR 50.55a for the 2004 Edition of the ASME Code for information.

Overall, the applicant's response provides details of various aspects of the Preservice and ISI Program including identifying components excluded from inspection, accessibility for examination, preservice examination plan, visual examination methodology, ultrasonic examination when needed, provision for alternative examination techniques, qualification of personnel, ISI schedule, evaluation of examination results, system pressure test, and evaluation of inaccessible areas following the provisions of IWE and IWL. The staff concluded that the applicant’s response is acceptable since it provides details of the ISI Program consistent with the requirements of the ASME Code Section XI, Subsection IWE and Subsection IWL.

3.8.1.5 *Post Combined License Activities*

There are no post COL activities related to this section.

3.8.1.6 *Conclusion*

The NRC staff's findings related to information incorporated by reference are in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff’s review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to concrete containment that were incorporated by reference have been resolved.

In addition, the staff compared the COL license and supplemental information in the application to the relevant NRC regulations, the guidance in Section 3.8.1 of NUREG–0800, and other NRC RGs. The staff’s review concluded that the applicant has adequately addressed the COL license information item and provided sufficient information on the Preservice and ISI Program, in compliance with NRC regulations.

The staff’s review confirmed that the applicant has adequately addressed the Tier 2 departure requiring prior NRC approval and the Tier 2* departure relevant to this section in accordance with Section 3.8.1 of NUREG–0800. The staff also found it reasonable that the identified Tier 2 departures are characterized as not requiring prior NRC approval per 10 CFR Part 52, Appendix A, Section VIII.B.5.

3.8.2 Steel Components of the Reinforced Concrete Containment

3.8.2.1 *Introduction*

This FSAR section describes the following steel components of the concrete containment vessel:

- Personnel air locks; two air locks providing access to the upper and lower drywell.

- Equipment hatches; three hatches serving the upper and lower drywell and suppression chamber.

- Penetrations; piping (hot and cold) and electrical RCCV penetrations.

- Drywell head; steel cover for the opening in the upper drywell top slab over the RPV.

The design criteria for the steel components include the following:

- Description of the components
- Applicable codes, standards, and specifications
- Loads and load combinations

- Design and analysis procedures
- Structural acceptance criteria
- Materials, quality control, and special construction techniques
- Testing and in-service inspection requirements
- Welding methods and acceptance criteria
- Shop testing requirements

3.8.2.2 *Summary of Application*

Section 3.8.2 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.8.2 of the certified ABWR DCD, Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in FSAR Section 3.8, the applicant provides the following:

Tier 1 Departure

- STD DEP T1 2.15-1 — Re-classification of Radwaste Building Substructure from Seismic Category I to Non-Seismic

This departure revises the seismic category of the RWB substructure from seismic Category I to non-seismic. This departure has no effect on the steel components of the reinforced concrete containment discussed in this subsection. Subsection 3.8.4.4 of this SER evaluates this departure.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

This departure identifies Tier 2* codes, standards, and RGs that are being updated to more current revisions or editions. The changes are in FSAR Tables 1.8-20 and 1.8-21.

Tier 2 Departure Not Requiring Prior NRC Approval

- STD DEP 12.3-3 — Steam Tunnel Blowout Panels

This departure removes the discussion concerning blowout panels and relief and release pathways associated with the steam tunnel. The description of the panels is in DCD Section 3.8.4 and Subsection 3.12.1.3. The applicant states that this departure does not have any adverse impact, does not change any plant physical feature, nor does it influence the design basis or the safety analysis. Therefore, this departure has no effect on the steel components of the reinforced concrete containment discussed in this subsection. For additional details about this departure, see Subsection 3.8.4.4 of this SER.

Administrative Departures Not Requiring Prior NRC Approval

- STD DEP Admin — Administrative minor correction

The applicant defines administrative departures as minor corrections—such as editorial or administrative errors in the referenced ABWR DCD (e.g., misspellings, incorrect references, table headings, etc.).

3.8.2.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the steel components of the reinforced concrete containment, and the associated acceptance criteria, are in Section 3.8.2 of NUREG–0800.

In accordance with Section VIII, "Processes for Changes and Departures," of, "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies one Tier 1 departure and one Tier 2* departure. Tier 1 departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4. Tier 2* departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.B.6. Tier 2 departures not requiring prior NRC approval, are subject to the requirements in Section VIII.B.5, which are similar to the requirements in 10 CFR 50.59.

3.8.2.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.8.2 of the certified ABWR DCD. The staff reviewed Section 3.8.2 of the STP Units 3 and 4 COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

Tier 1 Departure

- STD DEP T1 2.15-1 — Re-classification of Radwaste Building Substructure from Seismic Category I to Non-Seismic

This departure is evaluated in Subsection 3.8.4.4 of this SER.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

According to NUREG–1503, the major steel components of the concrete containment will be fabricated and tested as Class MC Components, in accordance with the 1989 Edition of Subsection NE of ASME Code Section III, Division 1. According to COL FSAR Section 1.8, Table 1.8-21, the applicable code version of ASME Code Section III, Division 1 (1989 Edition) remains unchanged. Therefore, Departure STD DEP 1.8-1 has no effect on the design and analysis of the steel components of the reinforced concrete containment. The applicant took no exceptions to Section 3.8.2 of the generic DCD for the standard ABWR plant design, and there is no outstanding COL license information item related to this section.

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1 See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

Tier 2 Departure Not Requiring Prior NRC Approval

- STD DEP 12.3-3 Steam Tunnel Blowout Panels

This departure is evaluated in Subsection 3.8.4.4 of this SER.

Administrative Departure Not Requiring Prior NRC Approval

- STD DEP Admin Administrative Minor Correction

Administrative departures do not impact any design function or method of performing or controlling a design function. Therefore, this departure has no effect on the steel components of the reinforced concrete containment discussed in this subsection.

The applicant's evaluation determined that this departure does not require prior NRC approval, in accordance with 10 CFR Part 52, Appendix A, Section VIII.B.5. Within the review scope of this section, the staff found it reasonable that this departure does not require prior NRC approval.

3.8.2.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

3.8.2.6 *Conclusion*

The NRC staff's findings related to information incorporated by reference are in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to steel components of the reinforced concrete containment that were incorporated by reference have been resolved.

In addition, the staff compared the additional information in the application to the relevant NRC regulations, the guidance in Section 3.8.2 of NUREG–0800, and other NRC RGs. The staff's review concluded that the applicant has adequately addressed the Tier 2* departure as it related to this area of review, in compliance with NRC regulations.

The staff's review confirmed that the applicant has adequately addressed the Tier 2* departure as it related to this area of review in accordance with Section 3.8.2 of NUREG–0800. The staff found it reasonable that the identified Tier 2 departures are characterized as not requiring prior NRC approval, per 10 CFR 52 Appendix A, Section VIII.B.5.

3.8.3 Concrete and Steel Internal Structures of the Concrete Containment

3.8.3.1 *Introduction*

This FSAR section describes the containment internal structures whose functions include (1) support of the reactor vessel radiation shielding, (2) support of piping and equipment, and (3) formation of the pressure suppression boundary. The containment internal structures are constructed of reinforced concrete and structural steel. The internal structures that are considered include the following:

- Diaphragm floor, which serves as a barrier between the upper drywell and the suppression chamber.

- Reactor pedestal, which provides support for the RPV and consists of a ledge on a cylindrical shell that forms the reactor cavity and extends from the bottom of the diaphragm to the top of the containment foundation slab.

- Reactor shield wall, which attenuates the radiation emanating from the RPV.

- Drywell and equipment pipe support structure, which supports the piping, pipe whip restraints, and mechanical and electrical equipment.

- Miscellaneous platforms, which allow access to and provide support for piping and equipment.

- Lower drywell equipment tunnel, which provides equipment access to the lower drywell from the RB.

- Lower drywell personnel tunnel, which provides personnel access to the lower drywell from the RB.

The major code used in the design of concrete internal structures is ACI 349 (1980 Edition). The ABWR DCD used the ANSI/AISC N690-1984, for the design of all steel internal structures.

The concrete and steel internal structures of the containment are designed to resist various combinations of dead and live loads, accident-induced loads (including pressure and jet impact), and seismic loads. The load combinations cover those cases most likely to occur and include all loads that may act simultaneously. The internal structures of the containment are designed and proportioned to remain within the limits in accordance with SRP Section 3.8.3 for the various load combinations. These limits are based on ACI 349 (1980 Edition), and ANSI/AISC N690 (1984 Edition) for concrete and steel structures, respectively, modified as appropriate for load combinations that are considered extreme.

The design criteria for the concrete and steel internal structures include the following:

- Description of the concrete and steel internal structures
- Applicable codes, standards, and specifications
- Loads and load combinations
- Design and analytical procedures
- Structural acceptance criteria
- Materials, quality control, and special construction techniques
- Testing and in-service inspection requirements
- Welding methods and acceptance criteria for structural and building steel

3.8.3.2 *Summary of Application*

Section 3.8.3 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.8.3 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in FSAR Section 3.8, the applicant provides the following:

<u>*Tier 2* Departure Requiring Prior NRC Approval*</u>

- STD DEP 1.8-1 — Tier 2* Codes, Standards and Regulatory Guide Edition Changes

This departure identifies Tier 2* items that are being updated to more current revisions or editions. The new changes are in FSAR Tables 1.8-20 and 1.8-21.

3.8.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the concrete and steel internal structures of the concrete containment, and the associated acceptance criteria, are in Section 3.8.3 of NUREG–0800.

In accordance with Section VIII, "Processes for Changes and Departures," of, "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies one Tier 2* departure. Tier 2* departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.B.6.

3.8.3.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.8.3 of the certified ABWR DCD. The staff reviewed Section 3.8.3 of the STP Units 3 and 4 COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

<u>*Tier 2* Departure Requiring Prior NRC Approval*</u>

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

According to NUREG–1503, the criteria used in the design, analysis, and construction of the internal structures of the containment conform with the following codes, standards, and specifications: RG 1.57 Revision 0, "Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components"; RG 1.94, Revision 1, "Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

Steel during the Construction Phase of Nuclear Power Plants"; and RG 1.142, Revision 1. The industry standards include ACI-349 (1980); ASME B&PV Code (1989 Edition), Section III Division 2, "Code for Concrete Reactor Vessels and Containments", and B&PV Code Section III Division 1, Subsection NE; ANSI/AISC N690 (1984); and ANSI N45.2.5-1974, "Supplementary Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants." According to COL FSAR Table 1.8-21, the applicable version of ASME Code Section III, Division 2 was changed to the 2001 Edition with the 2003 Addenda; and the ANSI/ACI 349 version was changed to the 1997 Edition. According to FSAR Table 1.8-20, RG 1.142 was changed to Revision 2, which endorses ACI 349–97 (with the exception of Appendix B, "Anchoring to Concrete"). RG 1.57 was updated to Revision 1 and is intended for the evaluation of metal containments and its components, but without requiring any backfitting of existing designs. As documented in FSAR Table 1.8-21a, the ANSI/AISC N690 code was changed to the 1994 version and is only applicable to site-specific SSCs. Hence, it does not affect the SSCs considered in this section.

NUREG–1503 states that any change to the use of ANSI/AISC N690 (1984 Edition) and ACI 349 (1980 Edition) for the design and construction of containment internal structural elements would constitute an unreviewed safety question and, therefore, would require NRC review and approval before implementation. In **RAI 03.08.04-33** and follow-up **RAI 03.08.04-36**, the staff asked the applicant to provide a detailed comparison of the differences between the code editions described above, in order to demonstrate that the safety margins are not reduced and also to evaluate any potential impact of the provisions of the newer codes that are more restrictive or result in a more robust design. The evaluation of these responses is discussed under the Tier 2* Departure STD DEP 1.8-1 in Subsection 3.8.4.4 of this SER.

3.8.3.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

3.8.3.6 *Conclusion*

The NRC staff's findings related to information incorporated by reference are in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to concrete and steel internal structures of the concrete containment that were incorporated by reference have been resolved.

In addition, the staff compared the information in the application to the relevant NRC regulations, the guidance in Section 3.8.3 of NUREG–0800, and other NRC RGs and industry standards. The staff's review concluded that the applicant has adequately addressed the Tier 2* departure relevant to this section, which is in compliance with NRC regulations.

3.8.4 Other Seismic Category I Structures

3.8.4.1 *Introduction*

This FSAR section addresses the RB, CB, and DGFOT as other seismic Category I structures that constitute the ABWR standard plant. Non-Category I structures that could interact with these structures are the RWB, SB, CBA, the plant stack on the RB roof, and the TB. With the exception of the plant stack, these structures are structurally separated from the other ABWR standard plant buildings. Of the above non-Category I structures, only the CBA was not included in the ABWR standard plant.

The applicant reclassified the RWB substructure from seismic Category I to non-seismic (under Departure STD DEP T1 2.15-1) and also relocated the RWB closer to the RB. In FSAR Subsection 3.8.6.4, the applicant identifies the UHS, the RSW piping tunnel, and the DGFOSV as three additional site-specific seismic Category I structures. A description of these structures is in FSAR Section 3H.6. Details of the DGFOT are in Section 3H.7.

Seismic Category I masonry walls are not used in the design. The ABWR standard plant does not contain seismic Category I pipelines buried in soil.

The criteria for the design of the other seismic Category I structures include the following:

- Description of the seismic Category I structures
- Applicable codes, standards, and specifications
- Loads and load combinations
- Design and analytical procedures
- Structural acceptance criteria
- Materials, quality control, and special construction techniques
- Testing and in-service inspection requirements

3.8.4.2 *Summary of Application*

Section 3.8.4 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.8.4 of the certified ABWR DCD Revision 4, referenced in 10 CFR Part 52, Appendix A. In addition, in FSAR Section 3.8.4 and Appendix 3H, the applicant provides the following:

<u>*Tier 1 Departures*</u>

- STD DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category I to Non-Seismic

This departure revises the seismic category of the RWB substructure from seismic Category I to non-seismic.

- STP DEP T1 5.0-1 — Site Parameters

The applicant identifies four site-specific departures from the generic site parameters used in the referenced ABWR DCD.

- The certified design site parameter for the site flooding was changed from 30.5 cm (1 ft) below grade to 182.9 cm (6 ft) above grade to account for the failure of the main cooling reservoir, which is a design-basis event for the STP site.

- The maximum design precipitation rate for rainfall was increased from 49.3 cm/h (1.62 ft/h) to 50.3 cm/h (1.65 ft/h).

- The humidity at the site, as represented by the wet bulb temperature, was increased from the specified humidity in the DCD.

- The shear wave velocity is less than the 305 m/s (1,000 ft/s) minimum specified in the DCD.

Tier 2 Departure Requiring Prior NRC Approval*

- STD DEP 1.8-1 Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

This departure identifies Tier 2* codes, standards, and regulations that are being updated to more current revisions or editions. The changes are in Tables 1.8-20 and 1.8-21 of the COL FSAR. This departure updates ACI 349 to the 1997 Edition and updates the ASME Section III, Division 2 Code to the 2001 Edition with the 2003 Addenda.

Tier 2 Departures Not Requiring Prior NRC Approval

- STD DEP 12.3-3 Steam Tunnel Blowout Panels

Departure STD DEP 12.3.3 moves the blowout panels from the RHR pump and heat exchanger room to the main steam tunnel. This departure also removes the tunnel's function to serve as a relief and release pathway for high energy events in the RB.

- STP DEP 3.5-2 Hurricane Generated Missile Protection

Departure STP DEP 3.5-2 modifies DCD Tier 2, Subsection 3.5.1.4 to incorporate the guidance in RG 1.221, "Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants," dated October 2011. Some of the hurricane parameters exceed the tornado-based parameters used in the design of standard plants. Section 3H.11 discusses the effects on the RB and CB.

- STD DEP 3.8-1 Resizing the Radwaste Building

Departure STD DEP 3.8-1 changes the overall dimensions and arrangement of the RWB resulting from process changes to the radioactive waste treatment systems.

- STD DEP 11.2-1 Liquid Radwaste Process Equipment

Departure STD DEP 11.2-1 changes the liquid radwaste management system (LWMS) to replace the permanently installed forced-circulation concentrator system with the mobile liquid radwaste processing system.

- STD DEP 11.4-1 Radioactive Solid waste Update

Departure STD DEP 11.4-1 changes the solid radwaste management system (SWMS) to replace the permanently installed system with the mobile system components.

COL License Information Item

- COL License Information Item 3.26 — Identification of Seismic Category 1 Structures

FSAR Section 3.8 incorporates by reference ABWR DCD Section 3.8, including all subsections. In FSAR Subsection 3.8.6.4, the applicant references Table 3.2-1 for a complete list of seismic Category I SSCs. These include the following site-specific seismic Category I structures:

- UHS/RSW pump house
- RSW piping tunnel
- DGFOSV

Supplemental Information

- DNFSB Issue — Resolution of Issues with Subtraction Method of Analysis Identified by DNFSB

In FSAR Section 3H.10, the applicant addresses the technical issue identified by DNFSB/DOE, which states that results may be non-conservative when analyzing embedded structures using the SM of analysis in SASSI.

- Hurricane Wind Design — Design for Site-Specific Hurricane Winds and Missiles

In FSAR Section 3H.11, "Design for Site-Specific Hurricane Winds and Missiles," the applicant addresses the impact of site-specific hurricane winds and missiles according to the guidance in RG 1.221 on the standard plant and site-specific structures at STP Units 3 and 4.

3.8.4.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for the other seismic Category I structures, and the associated acceptance criteria, are in Section 3.8.4 of NUREG-0800.

In accordance with Section VIII, "Processes and Changes and Departures," of, "Appendix A to Part 52--Design Certification Rule for the U.S. Advanced Boiling Water Reactor," the applicant identifies Tier 1, Tier 2*, and Tier 2 departures. Tier 1 and Tier 2* departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4. Tier 2 departures not requiring prior NRC approval are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.B.5, which are similar to the requirements in 10 CFR 50.59.

The review and acceptability of COL License Information Item 3.26 is based on meeting the applicable acceptance criteria and guidance in SRP Section 3.8.4.

3.8.4.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.8.4 of the standard DCD for the ABWR design. The staff reviewed Section 3.8.4 of the STP Units 3 and 4 FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

<u>Tier 1 Departure</u>

- STD DEP T1 2.15-1 Reclassification of Radwaste Building from Seismic Category 1 to Non-Seismic

In FSAR Section 3.8.4, the applicant references Departure STD DEP T1 2.15-1 that reclassifies the RWB substructure from seismic Category I to non-seismic, and commits to RG 1.143, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," for design of the radwaste processing systems, structures, and components. The applicant also removed all design information of RWB structure from FSAR Section 3.8.4 and Section 3H.3. The staff agrees that with the reclassification of the RWB substructure as non-seismic, the RWB design information need not be included as seismic Category I structures. However, the staff believes that the design information for the RWB still needs to be included in the FSAR, in order for the staff to ensure that the design of the RWB structure is performed in accordance with the guidance in RG 1.143, Revision 2, and thus meets the regulatory requirements in GDC 2, "Design basis for protection against natural phenomena", and GDC 60, "Control of releases of radioactive materials to the environment," of 10 CFR Part 50, Appendix A. Therefore, the staff asked the applicant in **RAI 03.08.04-2** to include design information for the RWB in the FSAR.

In the response to **RAI 03.08.04-2** dated September 15, 2009 (ML092610377), the applicant states that the RWB is a reinforced concrete structure located about 6.1 m (20 ft) west of the RB and is designed to RG 1.143, Revision 2. Also, since the above grade height of the RWB exceeds the distance to the RB, the RWB design shall satisfy II/I requirements to ensure that the integrity of the RB is maintained (i.e., the RWB cannot collapse or come in contact with the RB under SSE and design-basis tornado loads). The RWB is classified as RW-IIb (hazardous), in accordance with RG 1.143, Revision 2. In this response, the applicant also provides the basic criteria for the design and II/I evaluation of the RWB by including general references to RG 1.143 and to other RGs and industry codes and standards. The applicant adds that the analysis and design results will be available for review following the completion of the initial RWB design. However, the applicant's response did not provide any specific information about the RWB structure and its analysis and design. Therefore, the staff issued **RAI 03.08.04-18** to track the issue identified in **RAI 03.08.04-2**.

RAI 03.08.04-18 asked the applicant to provide design information for the RWB, and to include in the FSAR sufficient design information following the guidance in SRP Section 3.8.4. This RAI requested the applicant to provide information such as a detailed description of the structure, applicable codes, standards, specifications, loads and load combinations, procedures for the design and analysis, structural acceptance criteria, materials and quality control, design of the critical sections, and stability evaluation. In the Revision 1 response to **RAI 03.08.04-18** dated June 2, 2010 (ML101580248), the applicant states that the RWB is classified as RW-IIb (hazardous) for STP Units 3 and 4, per Section 5 of RG 1.143, Revision 2. However, the

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

applicant points out that the RWB is designed conservatively for earthquake, tornado, and wind loadings and meets or exceeds the requirements for the RW-IIa classification. A design for other loads is based on the requirements for the RW-IIb classification. The codes and standards that are used to determine loads, load combinations, load factors, and acceptance criteria meet or exceed those noted in Tables 1 through 4 of RG 1.143, Revision 2. The input motion for the seismic design is one-half of the DCD SSE defined in Tier 1, Table 5.0. Similarly, the design-basis tornado parameters are equal to three-fifths of the Region I tornado parameters defined in Table 1 of R.G. 1.76, Revision 1, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants." In addition, the tornado missiles are in accordance with Table 2 of RG 1.143, Revision 2 for the RW-IIa classification. The input motion for the II/I design is the envelope of the 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum resulting from the site-specific SSE, which is determined from an SSI analysis that accounts for the impact from the nearby RB. The applicant also states that the seismic II/I stability evaluations of the RWB structure for sliding, overturning, and floatation are in accordance with the criteria in the response to **RAI 03.07.02-13** (ML100550613), and the required safety factors are the same as those specified in SRP Section 3.8.5. The applicant submitted the markup of FSAR Section 3H.3 and Tables 3H.3-3, -4, and -5 that included the details of structural descriptions, loads, load combinations, the methodology of the analysis and design, and the seismic II/I evaluation.

The staff reviewed the information in the FSAR markup for Section 3H.3. The staff found that although the information in the FSAR generally meets the guidance in SRP Section 3.8.4 and the design criteria in RG 1.143, Revision 2, some additional clarifications were needed. During the audit in May 2011 (ML12346A233), the staff reviewed the calculations for the seismic analysis, design, and stability evaluation. The staff confirmed that the loads, load combinations, and acceptance criteria in the FSAR were used appropriately in the analysis and design of the RWB and in its stability evaluation. The staff discussed with the applicant the additional clarifications needed, which are described below:

1) The description of the structure in Section 3H.3.3 does not include any plans and sections that define the primary elements of the structure.

2) Soil parameters described in Subsection 3H.3.4.2.1 provide only the bearing capacity safety factors and do not include the actual bearing pressures, the ultimate bearing capacities, and the basis for calculating the bearing capacities.

3) The extreme environmental load combination for structural steel in Subsection 3H.3.4.3.4.2 does not clarify that the stress limit coefficient for the shear must be limited to 1.4.

4) FSAR Subsections 3H.3.5.1 and 3H.3.5.2 describe two models for seismic analyses. One is a fixed-base stick model, and the other is a SAP2000 3-D finite element model. It is not clear how the two types of models for seismic analyses were used in the design of the RWB.

5) The seismic II/I evaluation described in Subsection 3H.3.5.3 does not clearly explain how to perform the seismic analysis for the seismic II/I design.

6) The description of the stability evaluation in Subsection 3H.3.5.3 does not clearly explain how to determine the seismic demand or the methodology followed for the stability evaluation.

7) Flood design parameters for the II/I evaluation described in Subsection 3H.3.5.3 need to include all components of flood loads; namely, the hydrodynamic loads and loading due to floating debris.

In addition to the above information, the staff discussed the complexity of the various extreme environmental design parameters for the RWB as well as those same parameters that were applicable to all site-specific structures. Furthermore, the staff noted that the large openings in the RWB are not protected against tornado missiles. Although tornado missile protection is not required for the RW-IIb classification of the RWB, the applicant stated that the RWB is designed for tornado using the criteria for RW-IIa classification. The staff considered it necessary to clarify that such design excluded tornado missile protection for the large openings in the building. The applicant subsequently addressed all of the above issues in the following RAI responses and revised the FSAR:

- Supplement 1 Revision 1 response to **RAI 03.08.04-18** dated September 15, 2010 **(**ML102630145)**;** Supplement 2 Revision 1 response to **RAI 03.08.04-18** dated March 15, 2011 (ML110770440)**;** and the Supplement 3 response to **RAI 03.08.04-18** dated July 27, 2011 (ML11213A094)**;**

- Supplement 4, 5, and 6 responses to **RAI 03.07.02-13** dated November 28, 2011 **(**ML11335A232)**;** April 4, 2012 **(**ML12103A369); and May 29, 2012 (ML12153A101), respectively,

- Supplement 1 response to **RAI 02.03.01-24** dated April 10, 2012 **(**ML12103A368);

The applicant's clarifications of the above issues are discussed below:

1) The applicant's markup of FSAR Figures 3H.3-54 through 3H.3-60 includes plans and sections of the RWB structure showing the layout and dimensions of the major structural elements that meet the guidance in SRP Section 3.8.4. Therefore, the issue is considered to be closed.

2) The applicant's updated FSAR Subsection 3H.3.4.2.1 includes the actual bearing pressures and the ultimate bearing pressures. The update also clarifies that the soil bearing pressure capacities are determined using the methodology described in Section 2.5S.4. The staff accepted the methodology for determining the bearing capacity of soil in Section 2.5S.4. Therefore, the issue is resolved.

3) The applicant's addition of Note 1 to Subsection 3H.3.4.3.4.2 clarifies that the stress limit coefficient in the shear shall not exceed 1.4 for the extreme environmental loading combination. The issue is therefore resolved.

4) The applicant's updated FSAR Subsections 3H.3.5.1 and 3H.43.5.2 clarify the two types of seismic analyses of the RWB. The analysis and design of the RWB as well as the II/I design were performed using the RSA of a SAP2000 3-D finite element model. The II/I stability evaluation of the RWB used the base shears and moments obtained from the RSA of a fixed-base stick model. This proposed FSAR revision provided the requested clarification, and the issue is therefore resolved.

5) The applicant revised FSAR Subsection 3H.3.5.3 to clarify that the RWB analysis and design of the II/I design were performed using a SAP2000 3-D finite element model with shell and frame elements, as shown in Figures 3H.3-5 through 3H.3-7. This revision provided the requested clarification, and the issue is therefore resolved.

6) The applicant revised FSAR Subsection 3H.3.5.3 to clarify that the II/I stability evaluations for sliding and overturning were performed using the seismic input motions described in Subsections 3.7.2.8 and 3.7.3.16, and the seismic demands were determined by the RSA of the fixed-base stick model described in Subsection 3H.3.5.1. Also, Figure 3H.3-52 was added to the FSAR to outline the methodology followed for the seismic II/I stability evaluation of the RWB that is considered to be acceptable by the staff. This revision provided the requested clarification, and the issue is therefore resolved.

7) The applicant revised FSAR Subsection 3H.3.5.3 to clarify that the flood design parameters for the II/I evaluation included the hydrodynamic and flood debris loading per Section 3.4.2 for 12.2 m (40 ft) MSL flood level caused by the main cooling reservoir dike breach. This revision confirmed that the applicant has considered all of the components of flood loading for the II/I evaluation of the RWB. The issue is therefore resolved.

In addition to the above information, the applicant included the extreme environmental design parameters for the RWB in FSAR Table 3H.9-1 to clearly describe the various parameters used for the design, the stability evaluation, and the II/I design of the RWB. In the Supplement 1 response to **RAI 02.03.01-24** (ML12103A368), the applicant adds Note 6 to FSAR Table 3H.9-1 to clarify that the RWB structure is designed for tornado missiles and hurricane missiles, and that the large openings at and above grade are not missile protected. In the Supplement 4 Revision 1 response to **RAI 03.08.04-18** dated August 28, 2012 (ML12249A035), the applicant revises FSAR Subsection 3H.3.5.2 and states that the RWB finite element model includes uniform foundation soil springs and provides the values of the static and dynamic subgrade reaction moduli for the springs used in the model. The applicant states that the use of uniform soil springs is appropriate considering the fact that the RWB basemat is 6.1 m (12 ft) thick and is stiffened in both horizontal directions, with the interior shear walls arranged approximately every 9.1 m (30 ft), and no significant dishing of the mat is expected. The staff agreed with the applicant's assertion that no significant dishing of the RWB basemat is expected, because the basemat is very rigid and the assumption of uniform soil springs is considered to be a reasonable and practical approach for the design of the RWB basemat. The proposed changes to the FSAR were incorporated in Revision 9 of the FSAR and this issue is closed and resolved.

Based on the above discussions, the staff concluded that the applicant has addressed all of the staff's questions and requests for additional information about the design of the RWB.

The staff reviewed the design information for the RWB in FSAR Section 3H.3, including the FSAR markups in the RAI responses, as stated in the preceding paragraphs. FSAR Section 3H.3.1 describes the objective and scope of the RWB design. The description states that the RWB is a non-seismic Category I structure and is classified as RW-IIb, in accordance with RG 1.143, Revision 2. The RWB is designed to meet or exceed the design criteria per RG 1.143, Revision 2. Because the RWB is located close to safety-related seismic Category I structures, it is also designed to ensure that it does not collapse onto the nearby safety-related buildings (II/I design criteria). The staff found that the applicant has appropriately identified the scope of RWB design by meeting the relevant design criteria per RG 1.143, Revision 2 for the design of the structure. The RWB also meets the II/I design criteria to avoid any unacceptable interactions with adjacent seismic category I structures. FSAR Table 3H.9-1 provides the extreme environmental loads used to meet these design requirements.

FSAR Section 3H.3.3 describes the RWB structure. Figures 3H.3-54 through 3H.3-60 show the general layout of the RWB structure and the dimensions of the primary structural elements. The staff found that the information in this section adequately meets the guidance in SRP Section 3.8.4 and is therefore acceptable.

FSAR Section 3H.3.4 provides the criteria for the design of the RWB structure. As discussed before, the RWB is designed conservatively for earthquake, tornado, and wind loadings that are consistent with the RW-IIa classification. The staff noted that the codes and standards used to design the RWB are listed in this section. ACI 349-97, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," and RG 1.142, Revision 2 are used for the concrete design. ANSI/AISC N690–1984, "Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities," is used for design of the structural steel. These codes and standards that were used to meet the design requirements are in accordance with the guidance of RG 1.143, Revision 2 and are therefore acceptable. The structural steel and concrete materials that were used are consistent with the steel and concrete codes used for the design and are therefore acceptable.

FSAR Subsection 3H.3.4.2 describes the site design parameters used for the design and the evaluation of the RWB. The description shows that the static and dynamic soil bearing capacity factors of safety are in excess of 6.5. The design ground water level is at 9.75 m (32 ft) MSL per DCD and bounds the STP Units 3 and 4 site groundwater levels discussed in FSAR Section 2.4S.12. The design flood level is 10 m (33 ft) MSL per DCD and is above the level resulting from one-half of the PMF described in Section 2.4S.3, in accordance with RG 1.143 for RW-IIa classification. Similarly, the roof snow load of 2.39 kPa (50 psf) per DCD is very conservative for the STP Units 3 and 4 site, and exceeds the criteria in RG 1.143 for RW-IIa classification. In COL application Part 2, Tier 1, Table 5.0 shows that the design rainfall is 50.3 cm/h (1.65 ft/h).

FSAR Subsection 3H.3.4.3 describes the design loads and load combinations used in the RWB design. The selected basic wind speed of 202.7 km/h (126 mph) envelops the value derived from ASCE 7–95 in conformance to RG 1.143. The wind loads are calculated using the provisions of Chapter 6 of ASCE 7-95. The earthquake loads are due to one-half of the SSE defined in DCD Tier 1, Table 5.0. This corresponds to the RG 1.60 response spectra anchored to 0.15g. This design earthquake load envelops RG 1.143 for RW-IIa classification. The tornado parameters are equal to three-fifths of the Region I tornado parameters defined in Table 1 of RG 1.76, Revision 1. Therefore, the maximum tornado wind speed and pressure drop for the RWB design are 222 km/h (138 mph) and 8.27 kPa (1.2 psi), respectively. These findings, in addition to the tornado missile parameters, are in accordance with Table 2 of RG 1.143, Revision 2, for the RW-IIa classification.

The load combinations for the structural steel and reinforced concrete design conform to RG 1.143. Earthquake and wind loads are considered severe environmental loads, and the tornado load is treated as an extreme environmental load for the purpose of assigning load factors and allowable stresses.

As described in FSAR Subsection 3H.3.5.2, the analysis and design of the RWB are performed using a SAP2000 3-D finite element model with shell and frame elements, as shown in FSAR

Figures 3H.3-5 through 3H.3-7. The seismic loads are obtained from the RSA of this model. The input motion for this RSA is the 0.15 g RG 1.60 response spectrum. The seismic analysis method used (i.e., the finite element model and response spectrum method) is acceptable because it meets the SRP Section 3.7.2 acceptance criteria.

All concrete and steel designs are in accordance with the ACI 349-97 and ASNI/AISC N690-1984 code requirements, respectively. This finding meets the guidance in Table 1 of RG 1.143, Revision 2.

FSAR Subsection 3H.3.5.3 describes the seismic II/I evaluation that is performed to ensure that the RWB will not collapse on the nearby seismic Category I structures. The analysis and design for II/I are performed using a SAP2000 3-D finite element model with shell and frame elements. The seismic loads are obtained from the RSA of this model. The earthquake input used at the foundation level is the envelope of the 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to a site-specific SSE that is determined from an SSI analysis that accounts for the impact of the nearby RB. The lateral load resisting system of the RWB is designed to remain elastic for the seismic, flood, and tornado loads. For tornado parameters, including the missiles, the same parameters are used as those defined in DCD Tier I, Table 5.0. For a flood, the extreme flood level of 12.2 m (40 ft) MSL caused by the main cooling reservoir dike breach is used. The evaluation requirements for this flood, including hydrodynamic and flood debris loading, are described in FSAR Section 3.4.2. In the Supplement 1 response to RAI 02.03.01-24 dated April 10, 2012 (ML12103A368), the applicant proposes a site-specific ITAAC (Table 3.0-23) for the RWB to ensure that the lateral load resisting system of the RWB is designed to remain elastic under extreme environmental loads to prevent it from impacting the adjacent RB. The SSE loading is combined with other loads for structural steel and reinforced concrete elements as an extreme environmental load combination. The staff found this II/I evaluation method acceptable, because the analytical procedure, loads, load combinations, and use of design codes meet the SRP Section 3.8.4 criteria.

Stability evaluation of the RWB structure is performed considering the amplified input motions obtained from the Modified SM SSI analysis of the RB (see the Supplement 4 response to **RAI 03.07.02-13** in ML11335A232) and other site-specific parameters such as soil properties. The II/I stability evaluation of the RWB is performed using the base shears and moments obtained from the RSA of a fixed-base stick model. This fixed-base stick model is also used to obtain the seismic in-plane shears and moments of the exterior walls reported in Table 3H.3-1 and the structural frequencies reported in Table 3H.3-2. In the fixed-base stick model, the structure is represented by a lumped-mass model consisting of structural masses lumped at selected nodes that are connected by massless elements representing the stiffness properties of the shear walls between the nodes. The building masses are lumped at elevations where the building weights are concentrated, such as the floors and roof. For modeling reinforced concrete shear wall elements, the shear walls in each particular vibration direction are identified. The stiffness of a shear wall along its length consists of a combination of its shear stiffness and its flexural stiffness, both of which are calculated individually and combined to obtain the stiffness of the wall. This response spectrum analytical method is acceptable because it meets the SRP acceptance criteria. Table 3H.6-14 shows that the safety factors for sliding, overturning, and floating exceed the minimum safety factors specified in SRP Section 3.8.5.

In summary, the applicant committed to design the RWB to comply with the guidance in RG 1.143, Revision 2. For earthquake, wind, and tornado loads, the applicant designed the structure for the RW-IIa classification requirements. The calculated stability factors of safety against sliding, overturning, and floating exceed the criteria in SRP Section 3.8.5. Because the applicant provided an ITAAC to ensure that the lateral load resisting system of the RWB is designed to remain elastic under the extreme environmental loads, there is reasonable assurance that the RWB will not adversely impact the RB during such events. Also, the demonstration of the overall stability of the RWB using the guidelines in SRP Section 3.8.5 provides further assurance against the RWB impacting the RB. Based on the review of RAI responses, FSAR and COL application updates, and audit meetings, the staff found that the RWB design meets the SRP criteria and is hence acceptable.

During the review and approval of Tier 1 Departure STD DEP T1 2.15-1, the staff noted that the FSAR did not include any discussion of how the guidance in RG 1.143, Revision 2, was implemented to establish the RW-IIb classification of the RWB. In RAI 03.08.04-37, the staff asked the applicant to demonstrate that the RW-IIb classification of the RWB is appropriate, including the basis for supporting the classification process in Regulatory Position C.5 of RG 1.143, Revision 2 and to update the FSAR, accordingly. Subsequently, RAI 03.08.04-38 and RAI 03.08.04-39 were issued to address the classification of the radwaste system and the RWB. In the Revision 1 response to RAI 03.08.04-39 (ML13037A595), the applicant proposes to change the classification of the RWB to RW-IIa. The applicant compares the design criteria for RW-IIa per RG 1.143, Revision 2, to the criteria used for the STP Units 3 and 4 RWB design. According to this comparison, the RWB design criteria for earthquakes, winds, tornadoes, tornado missiles, floods, and precipitation (rain, snow, etc.) meet the design criteria for the RW-IIa classification per RG 1.143, Revision 2. This response also includes a revision to FSAR Table 3H.9-1 that deletes Note 6 and adds Note 7, which confirms that all exterior doors of the RWB are normally closed and missile protected. FSAR Subsections 3H.3.4.3.3.2, 3H.3.4.3.3.3, and 3H.3.4.3.3.4 were added to describe the design for a malevolent vehicle assault, an accidental explosion, and a small aircraft crash, respectively. The RWB protection from a malevolent vehicle assault is in accordance with RG 5.68. Using the guidance in RG 1.91, accidental explosion hazards were evaluated and found not to pose any hazards to the RWB. As discussed in FSAR Subsection 2.2S.2.7, the applicant used the methodology described in NUREG–0800, Subsection 3.5.1.6; RG 1.117, Revision 3 “Tornado Design Classification,” and DOE-STD-3014-96, “Accident Analysis for Aircraft Crash into Hazardous Facilities,” determine that the risks from aircraft hazards are sufficiently low at the STP Units 3 and 4 site and are thus not considered in the design of the SSCs.

The staff reviewed the applicant’s RAI response and noted that the RWB structure is already designed for earthquakes, winds, tornadoes, and tornado missiles according to the criteria for the RW-IIa classification. Therefore, the exterior RWB doors are missile protected. The RWB design-basis flood level of 33 ft MSL is higher than the site PMF of 26.3 ft, which is higher than one-half of the PMF that is required by RG 1.143 for the RW-IIa classification. The roof snow load design of 50 psf is very conservative for the STP Units 3 and 4 site—as discussed in FSAR Subsection 3H.6.4.3.3.5—and is more than what the RW-IIa classification requires per RG 1.143, Revision 2. The applicant also designed the RWB for a malevolent vehicle assault, accidental explosions, and a small aircraft crash per the guidance in RG 1.143, Revision 2. Based on the above information, the staff agreed that the RWB design meets the design criteria for the RW-IIa classification per RG 1.143, Revision 2. The response to RAI 03.08.04-39 is acceptable pending confirmation of FSAR update. The evaluation of radwaste processing systems in RAIs 03.08.04-38 and 03.08.04-39 will be in the Chapter 11 SER.

- STP DEP T1 5.0-1 Site Parameters

In Appendix 3H, the applicant references Departure STP DEP T1 5.0-1, which includes the following four site-specific departures from generic site parameters used in the referenced ABWR DCD: the design-basis flood level, maximum design precipitation, minimum shear wave velocity, and the maximum humidity. Of the four departures, the increase in humidity for the STP site is not considered to have any adverse impact on the structures. The following subsection evaluates the impact of the other three departures on the structures.

3.8.4.4.1 Evaluation of Site Parameters

A. Design-Basis Flood Level

The certified design site parameter for site flooding changes from 30.5 cm (1 ft) below grade to 182.9 cm (6 ft) above grade, in order to account for a potential breach in the main cooling reservoir as a design-basis event for the STP site. This change impacts the design of standard plant structures, as well as the site-specific structures in terms of the design of exterior doors and walls of these structures and their stability.

A.1. Watertight Doors and Barriers

In Part 7 of the COL application, the applicant evaluates Departure STP DEP T1 5.0-1 stating that the STP Units 3 and 4 safety-related SSCs are designed for or protected from a flooding event by watertight doors, which prevent the entry of water into the RB and the CB in case of a flood. The applicant also states that the exterior doors located below the maximum flood elevation on the 12,300 floor of the RB and the CB are revised to be watertight doors. Because these doors play a significant role in protecting safety-related SSCs and constitute a special design feature, the staff issued **RAI 03.08.01-3** requesting the applicant to include in the FSAR design information about these doors including locations, seismic and other design criteria, the seismic classification, redundancy features (if any), and to identify whether these doors will be used for normal access and egress to and from the RB and the CB. In the September 15 (2009) and June 29 (2010) responses to this RAI (ML092610377 and ML101830420, respectively), the applicant provides additional supplemental information to FSAR Subsection 3.8.6.4, which states that the watertight doors are seismic Category I and are designed to be leak tight. These responses also provide locations of the watertight doors and include some general design criteria. The staff noted that the responses did not include sufficient details about loads, load combinations and acceptance criteria for the design of these doors. Also, it was not clear from the response how the closure of normally open watertight doors would be ensured upon the indication of an imminent flood, because the applicant did not provide any details of an in-service inspection of these doors or station procedures for controlling operator actions. In addition, the staff noted that the access doors between the CB and the RB were not designated as watertight. Because there is a gap between these buildings, it is not clear how the ingress of flood water through these access areas was prevented. The applicant's response also did not address whether there was any redundancy feature considered for the normally open watertight doors. Therefore, the staff issued **RAI 03.08.01-6,** which was later followed by **RAI 03.08.01-9.** The RAIs asked the applicant to address or clarify the various issues considered necessary to demonstrate the adequacy of the watertight doors. The applicant's response to the staff's questions is discussed below.

1) *Description of Structures*

a. *Seismic Classification:* **RAI 03.08.01-6** asked the applicant to specify the seismic classification of the watertight doors in other relevant sections of the FSAR (e.g., Table 3.2-1), in order to ensure that these doors—including all components of the doors—will be appropriately treated for design, construction, installation, quality control, and maintenance. In the revised response to **RAI 03.08.01-6** dated June 29, 2010 (ML101830420), the applicant proposes to revise Table 3.2-1 of the COL FSAR by adding footnote (ii) to the RB and CB rows stating that watertight doors that protect the safety-related equipment from an external flood are designated as seismic Category I. The applicant incorporated the proposed change into the FSAR Revision 6. However, during a subsequent review, the staff observed that the above note was not added to the DGFOSV, which is also a seismic Category I structure with an external watertight door. Subsequently, in the Supplement 5 response to **RAI 03.07.02-13** dated April 10, 2012 (ML12103A369), the applicant provides a markup of Table 3.2-1 that includes the above footnote for the DGFOSV. These responses resolved the staff's concerns, and this issue in RAI 03.08.01-6 is resolved. The applicant incorporated the proposed changes into the FSAR Revision 8.

b. *Layout Plan:* **RAI 03.08.01-9** asked the applicant to provide a layout plan clearly identifying the location of all seismic Category I watertight doors. In the response to this RAI dated September 15, 2010 (ML102630145), the applicant refers to the Revision 3 response to **RAI 03.04.02-6** (ML110730067) proposing a revision to FSAR Subsections 3.4.3.1 and 3.4.3.3, including in the markup of a list and the location of the watertight doors or barriers in the RB, CB, and DGFOSV. The staff found that the applicant's information adequately identifies and locates the watertight doors. The applicant subsequently incorporated the proposed changes into FSAR Subsections 3.4.3.1 and 3.4.3.3, Revision 6. Therefore, this issue in **RAI 03.08.01-9** is resolved.

c. *Access Doors Between the CB and RB:* **RAI 03.08.01-6** asked the applicant to clarify details regarding the access doors between the CB and the RB. From the layout plan, these doors appear to be exterior doors and are not identified as watertight doors. In the revised response to **RAI 03.08.01-6** dated June 29, 2010 (ML101830420), the applicant states that the access doors between the RB and CB are not required to be watertight, because both buildings are separately protected from the design-basis flood by sealing the gap between these buildings. The staff found that the applicant's response adequately addresses the issue because these doors are not exposed to floodwater, and the gap between these buildings is sealed for protection against flood propagation. Therefore, the issue is considered resolved and closed. The adequacy of the seals is discussed in item "e" below.

d. *Availability of Normally Open Doors*: **RAI 03.08.01-6** asked the applicant to describe whether any redundancy features were considered for the watertight doors, particularly those doors that are normally open. The staff also asked the applicant to explain how the availability of the normally open watertight doors during a flood event is ensured considering that these doors will need to be closed upon the indication of an imminent flood. In the revised response to **RAI 03.08.01-6** (ML101830420), the applicant states that single failure assumptions are not typically imposed on design-basis external event calculations because they are conservative by design. The applicant adds that redundancy was not considered for the external or internal watertight doors used to control the effects of flooding. The response did not include a detailed description of the procedures and their validation to close the open doors. **RAI 03.08.01-9** asked the applicant to explain whether the mechanism in place to ensure that the requirement for the normally open watertight doors to be closed upon the indication of an imminent flood will be included in the station procedures. The RAI also asked the applicant to confirm whether there was an evaluation of the adequacy of the station procedures to effectively close these doors when needed. In the Revision 2 response to **RAI 03.08.01-9** dated March 7, 2011 (ML110730067), the applicant states that all doors protecting against the design-basis flood will be normally closed. This change is specified in a proposed revision to FSAR Subsection 3.4.3.1, which is included in the Revision 3 response to **RAI 03.04.02-6**. The staff found this response acceptable, because maintaining the watertight doors closed at all times does not require any additional operational actions during a flood. The staff also agreed that it is not necessary to consider redundancy for normally closed watertight doors. Therefore, this issue in **RAI 03.08.01-6** is resolved.

e. <u>*Description of the Seals*</u>: In the revised response to **RAI 03.08.01-6,** the applicant states that the gaps between the RB and the CB will be sealed as described in the response to **RAI 03.08.04-15** dated October 5, 2009 (ML092810321). The staff noted that this response provided only a conceptual detail of a joint seal between the buried RSW tunnels, the RSW pump house, and the CB. Also, in the February 10, 2010, response to a follow-up question in **RAI 03.08.04-25** (ML100550613) about the above referenced joint seal, the applicant had explained that if these seals were to fail, any in-leakage of groundwater would be at a very low rate. Because the seals for the gaps between the RB and the CB are credited for preventing the ingress of above-ground floodwater, the applicant's reference to the joint seals used for the buried RSW tunnels did not adequately address the issue. Therefore, follow-up **RAI 03.08.01-9** asked the applicant to describe the seal between the RB and the CB, including information about seismic classification; performance demand; and in-service inspection of the seal. The applicant provided more detailed information in the Revision 2 response to **RAI 03.08.01-9** (ML110730067) and the Supplement 1 response to **RAI 03.08.01-9** (ML111050565 dated April 11, 2011). The applicant stated that the joint seals between the RB and the CB below the design-basis flood level will be made using a polyurethane foam impregnated with a waterproof sealing compound between the concrete surfaces and an interior redundant water stop. This information is also included in the markup of a proposed revision to FSAR Subsection 3.4.3.1 submitted with the Revision 3 response to **RAI 03.04.02-6**. That response states that the seal material and joint seal assembly shall be tested to verify their watertight capability when subjected to the maximum anticipated hydrostatic head. The testing program will demonstrate the following:

- The seal material can withstand a movement of ±25 percent of the gap size or the expected long-term settlement, whichever is larger, in any resultant direction and still be watertight.
- The seal material can compress to one-third of its thickness without developing more than 25 psi pressure on the adjacent structures.
- The entire joint seal assembly, including the watertight joint seal and the redundant water stop, can prevent the total leakage during an SSE event from exceeding that which will cause internal flooding to exceed the height of the flood protection curbs or raised equipment pads. The total permitted leakage for the joint seal assembly shall be determined for the entire duration of the SSE when simultaneously subjected, to the maximum anticipated hydrostatic pressure, the maximum differential displacements due to long term settlement or tilt, and the maximum differential displacements due to the SSE.

- The seal material can function as a watertight barrier after being subjected to the maximum displacements from the SSE, and the redundant water stop on the interior side of the joint can withstand the SSE maximum displacements without degradation.

In addition, the applicant states that the foregoing requirements will demonstrate that the material is capable of being watertight after the effects of long-term settlement or tilt, as well as during normal operating vibratory loading such as SRV actuation and not impact the adjacent structures. The applicant also states that the joint seal and the interior water stop are classified as seismic Category I with respect to their ability to remain in place, and an in-service inspection program will ensure that the joint seal materials do not significantly degrade.

In the Revision 2 response to **RAI 03.08.01-9** (ML110730067), the applicant states that the gap size is determined by the displacement under the SSE load, in addition to the long-term settlement similar to the joints discussed in the response to **RAI 03.08.04-25** dated May 13, 2010 (ML101340651). The gap size in the above response is at least 50 percent larger than the calculated maximum displacements. The staff reviewed the applicant's proposed design and testing requirements for the seals between the CB and the RB. The staff agreed with the applicant that the gap size for the seal will ensure that the gap will not be compressed to more than one-third of the total gap from the settlement, tilt, and SSE. This gap size will ensure that the seal does not develop more than 172.4 kPa (25 psi) pressure on the adjacent structures. Also, testing the seals to withstand a movement of ±25 percent of the gap size or the expected long term settlement, whichever is larger, will ensure that the seal material will maintain its integrity and will be able to withstand the maximum long-term settlements and differential displacements during normal operating vibratory loads such as SRV actuation. Testing the seal material for total leakage during a SSE to confirm that the seal material will be within an acceptable limit under maximum anticipated hydrostatic pressure, maximum differential displacement from settlement and tilt, and maximum differential displacement from the SSE will demonstrate that the seal material remains in place to stop significant water leakage into the safety-related buildings during and after a seismic event. The staff had also asked the applicant about the capability of the RB and CB walls to withstand the additional pressure of 172.4 kPa (25psi) exerted by the filler material at the seismic joints. In the Supplement 6 response to **RAI 03.07.02-13** dated May 29, 2012 (ML12153A101), the applicant states that below grade CB walls are designed for minimum 598 kPa (86.7 psi) lateral seismic soil pressures that envelop the 25 psi (3.6 kilo pound-force [kips] per square foot) pressure from the filler material. Since the areas where the below grade tunnels meet the CB are not exposed to below grade lateral soil pressure, the CB walls are already designed for a lateral pressure higher than what is exerted by the tunnel seals. Similarly, the staff noted that the below grade RB walls are also designed for a lateral seismic soil pressure greater than 172.4 kPa (25 psi). For above grade, the only seismic gap is at the junction of the main steam tunnel. The total load from the filler material at this joint is conservatively estimated to be no more than 2,670 kilonewtons (kN) (600 kips), assuming 0.30-m (1-ft) wide filler material over a 48.77-m (160-ft) perimeter of the tunnel (i.e., 160 x 3.6 = 576 kips < 600 kips). This load is transferred to the 1.6-m (5.25-ft) thick slabs and walls of the main steam tunnel as in-plane loads. On the CB side, the main steam tunnel walls are part of two of the main shear walls of the CB. On the RB side, the load is transferred as in-plane loads to slabs and walls that are connected to the RCCV shell. These loads are considered to have a negligible impact on the RB and CB walls and slabs. The staff agreed with the above explanation provided by the applicant to address the adequacy of the RB and CB walls to withstand the additional pressure exerted by the joint filler material. Therefore, the staff concluded that the applicant has adequately addressed the issues related to sealing the gap between the RB and the CB to protect against design-basis flooding. The FSAR was subsequently updated to incorporate the proposed markups in the Revision 3 response to **RAI 03.04.02-6**. Therefore, this issue is considered resolved and closed.

2) *Applicable Codes, Standards, and Specifications*

RAI 03.08.01-6 asked the applicant to clearly state in the FSAR the codes and standards used to design the watertight doors. The applicant's Revision 1 response to **RAI 03.08.01-6** (ML101830420) refers to the response to **RAI 03.04.02-6**, which includes the markup of FSAR Subsection 3.4.3.1 stating that the watertight doors, frames, and all components are designed to the requirements of ANSI/AISC N690 and SRP Section 3.8.4; that the fabrication of the doors meets the requirements of ANSI/AISC N690; and that welding meets the requirements of nondestructive testing, personnel qualifications, and acceptance criteria in American Welding Society (AWS) D1.1, "Structural Welding Code – Steel." The response did not include any information about which version of ANSI/AISC N690 the applicant would use for the design of the watertight doors. **RAI 03.08.01-9** asked the applicant to confirm that the version of ANSI/AISC N690 used in design is the same as the referenced version in SRP Section 3.8.4 (i.e., AISC N690-1994) or justify using any other version of the code. In the Revision 2 response to **RAI 03.08.01-9** (ML110730067), the applicant states that the applicable version of ANSI/AISC N690 for the site-specific DGFOSV is 1994 with Supplement 2, in accordance with SRP Section 3.8.4, Revision 2 (the revision applicable to site-specific structures). The applicant also states that COL application Table 1.8-21a will be revised to include this revision of the code for site-specific applications, as shown in the Supplement 1 response to **RAI 03.08.04-33** dated July 27, 2011 (ML11213A094). The applicant notes that for the RB and CB, the applicable version of ANSI/AISC N690 is 1984 as listed in DCD Table 1.8-21; and these versions will be used in the design of the doors when applicable. The staff found that the response adequately addressed the issue by specifying the versions of the code that apply to each Category I structure. Using the version of the code that is specified in the DCD for the RB and CB is considered acceptable, because these structures are included in the DCD. The applicant also appropriately used the version of the code referenced in SRP Section 3.8.4 for the DGFOSV, which is a site-specific structure. The applicant subsequently incorporated the revision into Tables 1.8-21 and 1.8-21a in the FSAR to reflect the use of the code versions stated above. Therefore, the issue is considered resolved.

3) *Loads and Loading Combinations*

In **RAI 03.08.01-6,** the staff asked the applicant to clearly state in the FSAR the loads and load combinations used for the design of the watertight doors. The applicant's Revision 1 response to **RAI 03.08.01-6** refers to the response to **RAI 03.04.02-6**, which includes the markup of FSAR Subsection 3.4.3.1 that describes the loads and load combinations to be used in the design. However, the response did not completely address the various issues pertaining to loads and load combinations. **RAI 03.08.01-9** asked the applicant to (1) describe how the flood loads and the loading combination were determined, including the load factors used in loading combinations involving the flood load because ANSI/AISC N690 and ACI 349 do not specifically address flood loads; (2) include in Subsection 3H.6.4.3.3.4, "Extreme Environmental Flood (FL)," a description of the various components of flood loading (e.g., hydrostatic load, hydrodynamic load, impact load from debris transported by floodwater, etc.) and the corresponding design values that were used; and (3) explain why only hydrostatic or differential pressure load 'P' needs to be considered for the design of the watertight doors and not the other components of a flood (e.g., hydrodynamic load and the load from debris transported by the flood. In the Revision 2 response to **RAI 03.08.01-9** (ML110730067), the applicant states that the controlling flood event at STP Units 3 and 4 results from the main cooling reservoir dike break and, as such, is considered an extreme environmental load. The applicant also states that according to Commentary R9.2.7 of ACI 349-97, other extreme environmental loads can be replaced for tornado loadings in the load combinations, which was incorporated to design the doors and barriers. Regarding the flood load components, the applicant included the hydrostatic load from the flood elevation at 12.2 m (40 ft) MSL; the associated drag effects of 2.11 kPa (44 psf); the impact from the floating debris per Section 3.4.2 and the hydrodynamic load resulting from the wind-generated wave action per Figure 3.4-1 (Figure 3.4-1 is only used to calculate the hydrodynamic load from the wind-generated wave action). The applicant also noted that the weight of the water (above ground) due to the flood loads is 1.023 grams per cubic centimeter (g/cc) (63.85 pounds per cubic foot [pcf]) in order to include the effects of suspended sediments in the water. The applicant subsequently included the above information in Section 3.4.2, Section 3.4.3, and Subsection 3H.6.4.3.3.4 of the FSAR. The staff reviewed the response and other referenced documents and determined that the response adequately addresses the issues pertaining to flood loads and load combinations by following accepted engineering practices in defining the components of flood loads and loading combinations. The applicant also included in the FSAR all components of flood loading and appropriate loading combinations. Therefore, the issue is considered resolved.

4) *Design and Analysis Procedures*

In the response to **RAI 03.08.01-6** (ML101830420) regarding the analysis and design procedures of watertight doors, the applicant states that "the design of the doors will be performed in accordance with SRP Section 3.8.4." The staff did not consider that merely referencing SRP Section 3.8.4 provides sufficient information about the analysis and design procedure used by the applicant. Therefore, **RAI 03.08.01-9** asked the applicant to describe the analysis and design procedures, including how seismic loads are determined for the watertight doors. The applicant's Revision 2 response to **RAI 03.08.01-9** (ML110730067) states that the watertight doors will be designed by vendors in accordance with specific requirements given in the procurement specification, and that the procurement specification will include that the detailed analysis and design should comply with the guidance in the applicable revision of SRP Section 3.8.4 and ANSI/AISC N690. The response also states that the seismic loads will be determined using the applicable response spectra, and that the method of analysis for evaluating seismic and other reactor building vibratory loadings, if applicable, will be the static equivalent method as described in DCD Subsection 3.7.3.8.1.5. The staff reviewed the response and determined that the analysis and design procedures described by the applicant meet the guidance in SRP Section 3.8.4 and are accepted engineering practices. The information included in the FSAR is also considered adequate for the design of these doors. Therefore, the issue is considered resolved.

5) *Structural Acceptance Criteria*

FSAR Subsection 3.4.3.1 includes the structural acceptance criteria for each load combination. The acceptance criteria are considered adequate because they meet the requirements of ANSI/AISC N690 referenced in SRP Section 3.8.4.

6) *Materials Quality Control, Special Construction Techniques, and Quality Assurance*

In the Revision 1 response to **RAI 03.08.01-6,** the applicant refers to the proposed revision to FSAR Subsection 3.4.3.1 submitted in the response to **RAI 03.04.02-6**. In the Revision 3 response to **RAI 03.04.02-6,** the applicant provides an FSAR markup stating that the structural steel used for the watertight doors conforms to either ASTM A36, or ASTM A992, or ASTM

A500 Grade B. The faceplate conforms to ASTM A606, Type 4; and the rubber gasket conforms to ASTM D1056 Type 2, Class D. The steel materials proposed for the watertight doors conform to specification ANSI/AISC N690-1994 Supplements 1 and 2; and ASTM D1056 Type 2, Class D is a commonly used gasket material for extreme temperature resistance and is considered appropriate. The FSAR was subsequently updated to reflect the proposed changes and these issues are considered resolved.

7) *Testing and In-service Surveillance Programs*

RAI 03.08.01-6 asked the applicant to clearly state in the FSAR the testing and in-service surveillance programs for the watertight doors. In the January 14, 2010, response to **RAI 03.08.01-6** (ML100191524)**,** the applicant states that the water retaining capability of the doors shall be demonstrated by qualification tests for the water head levels before shipment of the doors. In this response, the applicant also provides the FSAR markup that includes a description of the tests and allowable leakage through the door seals. The staff needed several clarifications regarding the response, which is described below.

In the response to **RAI 03.08.01-6**, the applicant states that watertight doors will allow a slight seepage that satisfies a Type 2 closure per U.S. Army Corps of Engineers EP 1165-2-314, "Flood Proofing Regulations" (1992). Because it was not clear whether the proposed leakage associated with a Type 2 closure was in line with internal flooding considerations, **RAI 03.08.01-9** asked the applicant to justify why the applicant considers the leakage of 1.24 liters per meter (1/10 gallon per linear foot) of gasket per hour to meet the criterion for a Type 2 closure and to provide the basis for the assumption that such leakage will not compromise the functionality of any safety-related commodity or any other design-basis parameters. In the Revision 2 response to **RAI 03.08.01-9,** the applicant states that there are less than 304.8 m (1,000 ft) of gasket material for all of the watertight doors used for protection against external flooding; that a leakage rate of 1.24 liters per meter (1/10 gallon per linear foot) of gasket per hour equates to 0.0063 cubic meters per minute (m^3/min) (100 gallons/hour), which is far less than the 1.34 m^3/min (354 gpm) accepted for internal flooding in the RB at Floor 400 (1F) described in DCD Subsection 3.4.1.1.2.1.4 and the 12.0 m^3/min (3,170 gpm) accepted for internal flooding in the CB per DCD Subsection 3.4.1.1.2.2 due to internal pipe leakage. The response also states that the safety-related equipment potentially subjected to external flooding is protected by curbs and raised equipment pads, similar to the safety-related equipment potentially subjected to internal flooding. The staff found that the applicant has adequately addressed the issue by demonstrating that the proposed seal leakage is enveloped by the design limits established for internal flooding. Therefore, this issue is resolved.

In the Revision 1 response to **RAI 03.08.01-6,** the applicant refers to the proposed revision to FSAR Subsection 3.4.3.1 submitted in the response to **RAI 03.04.02-6**, which states that the watertight doors are seated such that the force of the water helps to maintain the watertight seal. The response also states that the water-retaining capability of the doors shall be demonstrated by qualification tests for the water head levels. The response included a list of testing parameters such as the gasket material and cross-section, its retainers, and the anvil configuration identical to that of the full-size doors to ensure leak-tightness. Because the applicant's test description considered only maximum hydrostatic pressures, **RAI 03.08.01-9** asked the applicant to justify that testing these doors against the maximum water pressure only is adequate and will envelop the performance of the seals during lower hydrostatic pressure. In the Revision 2 response to **RAI 03.08.01-9** (ML110730067), the applicant states that during the test, the hydrostatic head will be raised at a rate of not more than 0.305 m/min (1 ft/min) to a level 25 percent higher than the flood level, and any leaks that occur during this time will be detected. If the leakage rate begins to diminish as the hydrostatic head increases, the assembly will be tested at a lower hydrostatic head. The response also states that this requirement was added to the COL FSAR markup in the Revision 3 response to **RAI 03.04.02-6** (ML110730067). The staff found that the applicant's response has adequately addressed the issue of testing the watertight doors, because the proposed test procedure and parameters simulate the conditions these doors may be subject to during flooding. No special in-service inspection other than what would be needed to meet the requirements of the Maintenance Rule is considered necessary, because these doors will be normally closed. The proposed FSAR markups were included in a subsequent revision of the FSAR, and the issue is therefore resolved.

As described above, the applicant's response has adequately addressed the staff's requests for additional information and clarification in **RAI 03.08.01-3** and follow-up **RAI 03.08.01-6** and **RAI 03.08.01-9.** Therefore, these RAIs are closed.

A.2. <u>Evaluation of Standard Plant Structures for Site-Specific Flood Loads</u>

FSAR Appendix 3H, Section 3H.1.6, "Site Specific Structural Evaluation," addresses the effect of an increased maximum flood level for STP Units 3 and 4 on the design of the RB. However, the FSAR text included only a statement that the load due to the revised flood level on the RB is less than the ABWR Standard Plant RB seismic load. The staff considered the evaluation from the applicant to be very qualitative, and it did not adequately address all of the issues associated with the increased flood level. Therefore, in **RAI 03.08.01-4** and subsequently in **RAI 03.08.01-7** and **RAI 03.08.01-10**, staff asked the applicant to provide a quantitative evaluation of the RB that considered all of the effects resulting from the increased flood level. These effects include, but are not limited to, wave effects, loadings due to flow and drag, and overall stability of the structure considering floatation. The above RAIs also asked the applicant to address the same issues for the CB. The resolution of the various issues pertaining to the evaluation of standard plant structures for site-specific flood loads are described below.

1) *Loads and Loading Combinations*

<u>*Load Combination*</u>: In response to the staff's question regarding the basis for the loading combination used for flood loading, with references to applicable industry codes and standards, in the November 29, 2010, Revision 1 response to **RAI 03.08.01-10** (ML103360074) the applicant states that the load combination used for flood loading is based on the provisions in Section 9.2.7 of ACI 349-97 stating that "If resistance to other extreme environmental loads such as extreme floods is specified for the plant, then an additional load combination shall be included with the additional extreme environmental load substituted for Wt in Load Combination 5 of 9.2.1". The staff found that the applicant's response has adequately addressed the issue by demonstrating compliance with the applicable concrete code. Therefore, this issue is resolved.

<u>*Debris Loading*</u>: The staff noted that in the response to **RAI 03.08.04-22** (ML100550613), the applicant had used a water density of 1.28 g/cc (80 pcf) to account for the effect of debris in flood water. Since the basis for this assumption was not clear, **RAI 03.08.01-10** asked the applicant to justify, with references to industry standards and codes, the assumption to use 1.28 g/cc (80 pcf) unit weight of water to account for debris loading on the exterior walls of the RB and CB. In the Revision 1 response to **RAI 03.08.01-10**, the applicant states that in order to account for the impact of floating debris, the guidance in Section C5 of the Commentary to ASCE 7-05 was used and accordingly, the impact from a floating piece of debris weighing 226.8

kg (500 lbs) traveling at a maximum flood water velocity of 1.44 m/s (4.72 ft/s) was considered. The applicant also states that the flood water density, considering the maximum sediment concentration, is 1.023 g/cc (63.85 pcf) per COL FSAR Subsection 2.4S.4.2.2.4.3 and the density of 1.44 g/cc (80 pcf) noted in the response to **RAI 03.08.04-22** is a conservatively assumed value that was revised to 1.023 g/cc (63.85 pcf) considering the maximum sediment concentration in the Revision 1 response to **RAI 03.08.04-22** dated September 15, 2010 (ML102630145). The staff found that the applicant has adequately addressed the issue of debris loading by following the guidance in an accepted engineering standard such as ASCE 7-05. Therefore, this issue is resolved.

2) *Design and Analysis Procedures*

In the Revision 1 response to **RAI 03.08.01-4** dated September 15, 2010 (ML102630145), regarding the quantitative evaluation of the effects of a site-specific flood on the RB and the CB, the applicant compares out-of-plane shear and moment demands for above grade wall elements of the RB and the CB resulting from flood loads with those resulting from seismic loads. The comparison shows that the demands from the seismic loads are higher. However, it was not clear from the response how the representative wall elements were selected, and how the shear and moment demands were determined for both flood and seismic loads. Therefore, **RAI 03.08.01-7** asked the applicant to provide a detailed description of how the representative wall elements for the RB and the CB were selected for the evaluation of out-of-plane shears and moments; how the reported shear and moment demands for flood and seismic loads were determined, including consideration of the impact from floating debris during the flood. In the November 29, 2010, Revision 2 response to **RAI 03.08.01-7** (ML103360074), the applicant provides details of the computation of out-of-plane shear and moment demands on the most critical exterior above ground walls of the RB and CB. The shear and moment demand from the flood load were calculated considering a hydrostatic pressure of 1.83 m (6 ft) high flood water above grade, a hydrodynamic drag load of 2.11 kPa (44 psf), and a hydrodynamic force from wind-generated waves 897.4 kg/m (603 lbs/ft). The hydrodynamic forces are described in FSAR Subsection 2.4S.4.2.2.4.3. The above demands were shown to be less than the corresponding demands from the tornado wind load without considering the tornado-generated missile impact. The applicant also states that the impact from a 226.8-kg (500-lb) floating debris during a flood is significantly less than the impact from the tornado missile considered in the design of the RB and the CB. Based on the above details, the applicant concludes that the design of the above grade exterior walls of the RB and CB for tornado wind pressures in conjunction with tornado generated missiles bounds the design for flood loading. The staff found that the applicant’s justification for the design of the above grade exterior RB and CB walls acceptable because all components of the flood loading were considered in the evaluation, and the results were shown to be enveloped by the results from tornado loading. For the below grade walls, the applicant compared the increase in the out-of-plane loads on the RB and CB walls from a flood level of 1.83 m (6 ft) above grade with the seismic lateral soil pressures on the corresponding walls. Since the maximum ground water level is 0.61 m (2 ft) below grade, a flood elevation of 1.83 m (6 ft) above grade would cause an increase in hydrostatic pressure on the below ground walls corresponding to a 2.44-m (8-ft) height of water. In the Revision 2 response to **RAI 03.08.01-7**, the applicant calculated the increase in wall pressure based on a 7-ft height of water. In the April 11, 2011 Supplement 1 response to **RAI 03.08.01-7** (ML111050565), the applicant revises the below ground wall calculation to consider a water level of 2.44 m (8ft) (instead of 2.13 m [7 ft]) above ground water level and stated that the resulting hydrostatic pressure on the walls is still under the lateral seismic soil pressure used in the design and concluded the DCD design is adequate for the site-specific flooding event. The staff found the applicant’s evaluation acceptable, since the below grade walls may be considered to be subjected to only the additional lateral hydrostatic pressure from flooding and the increases in the wall pressures are less than the seismic lateral soil pressures considered in the design of these walls in the ABWR DCD. Therefore, this issue is resolved.

3) *Stability Evaluation*

RAI 03.08.01-4 asked the applicant to address the effect of the increased flood level on the overall stability of the RB and the CB. **RAI 03.08.01-7** specifically asked the applicant to explain why the safety factors for stability reported in Table 3H.1-23 (RB) and Table 3H.2-5 (CB) of the ABWR DCD are not considered affected by the increased flood load. Also, the staff asked the applicant to provide a quantitative evaluation of sliding and overturning stability from flooding that considers the hydrodynamic loads and the buoyancy effects on the structures and to update the FSAR accordingly. In the Revision 2 response to **RAI 03.08.01-7** dated November 29, 2010 (ML103360074), the applicant states that the SRP Section 3.8.5 criteria for a stability evaluation are included in load combinations "A" through "E" of SRP Acceptance Criterion 3, and they do not include any flood loads except for the buoyant force of the design-basis flood. The applicant provides a qualitative evaluation in the response stating that since the flood level is only 1.83 m (6 ft) above grade, the unbalanced forces due to the design-basis flood will be negligible compared to the unbalanced loads due to the seismic SSE. In this response, the applicant also calculates the new flotation safety factors considering the design-basis flood level of 12.2 m (40 ft) MSL for the RB and the CB to be 2.24 and 1.3, respectively, The applicant also provides a markup of FSAR Sections 3H.1.6 and 3H.2.6 and ABWR DCD Tables 3H.1-23 and 3H.2-5. The staff agreed with the applicant's position that SRP Section 3.8.5 does not specifically address a stability evaluation that considers a flooding scenario similar to the design-basis flood at the STP site, except for considering buoyancy effects from a steady-state flooding condition. However, the staff maintained that such evaluation must be carried out to demonstrate that the structure is available to perform its intended function during the design-basis flood. The staff also agreed with the applicant's evaluation that the unbalanced forces from 1.83-m (6-ft) high flowing flood water would be significantly less than the unbalanced forces from the SSE. The applicant also revised the floatation factors of safety for the RB and the CB considering site-specific flood loads, and the revised factors of safety are larger than the minimum value of 1.1 per SRP Section 3.8.5. The applicant has subsequently revised the FSAR to incorporate the changes in with the response. Based on the above details, the staff concluded that the applicant has adequately addressed the issue of stability of standard plant structures for site-specific flooding. Therefore, this issue is resolved.

As described above, applicant's responses adequately addressed all issues pertaining to the evaluation of standard plant structures for a site-specific flood. Therefore, **RAI 03.08.01-4** and follow-up **RAI 03.08.01-7** and **RAI 03.08.01-10** are closed. The resolution of issues associated with the design-basis flood level in Departure STP DEP 5.0-1 related to this section is acceptable.

B. Site-Specific Maximum Rainfall and Snow Loads

Per COL FSAR Subsection 2.3S.1.3.4, the extreme liquid water precipitation [probable maximum winter precipitation (PMWP)] event at the STP site was determined to be 86.36 cm (34.0 in.). In the response to **RAI 02.03.01-6,** the applicant states in part that the roofs of those ABWR standard plant safety-related buildings designed with parapets are furnished with scuppers to supplement the roof drains, so that excessive ponding of water cannot occur. However, the applicant did not provide details of the design of the roof scuppers and drains demonstrating that an antecedent ice storm or an antecedent snow pack from the normal winter precipitation event will not clog both the roof scuppers and drains and therefore, they will prevent no more than 2.394 kPa (50 lbf/ft^2) of water accumulation on the roof. **RAI 03.08.04-14** asked the applicant to provide details of the design of the roof scuppers and drains to demonstrate that an antecedent ice storm or an antecedent snow pack from the normal winter precipitation event will not clog both the roof scuppers and drains. In the September 16, 2009 response to **RAI 03.08.04-14** (ML062610374), the applicant states that the roofs of these buildings, which are required to resist tornado-generated missiles, are reinforced concrete slabs with a minimum thickness of 35.56 cm (14 in.). The maximum height of the parapet on top of the ABWR standard plant safety-related buildings is less than 22.86 cm (9 in.). Therefore, the maximum accumulated water height on top of these roofs cannot exceed 22.86 cm (9 in.). The response also states that the deflection of these roofs from the load corresponding to 22.86 cm (9 in.) of water (i.e. 2.25 kPa [47 lb/ft^2]) would be insignificant, and thus ponding is not a concern. Based on the above details, the applicant concludes that even if the roof scuppers and roof drains clog, the roof loading could not exceed 2.394 kPa (50 lb/ft^2), which is the design live load for roofs. Since neither the DCD nor the FSAR included any information about the height of parapets on standard plant structures, **RAI 03.08.04-24** asked the applicant to include this information in the FSAR and to also explain why any potential incidental live load need not be considered concurrent with an extreme winter precipitation event. In the February 4, 2010, response to **RAI 03.08.04-24** (ML100480204)**,** the applicant states that per Subsections 3.8.4.3.1.1 and 3.8.4.3.2 of DCD Tier 2, snow load and roof live load are considered non-concurrent loads. Therefore, for the ABWR standard plant seismic Category I structures, there was no need to combine any roof live load with the extreme winter precipitation load. The applicant also revises COL FSAR Table 2.0-2, which was submitted in the response to **RAI 03.08.04-21** (ML100480204)**,** to limit the parapet height of the ABWR standard plant structures to 22.86 cm (9 in.). The staff found that the applicant's response adequately addresses the issue, because limiting the maximum height of the parapet of the ABWR standard plant structures to 22.86 cm (9 in.) would limit the maximum height water accumulation on roofs to no more than 22.86 cm (9 in.), even when both the roof scuppers and drains are clogged due to an antecedent ice storm or an antecedent snow pack from normal winter precipitation. The applicant subsequently updated the FSAR to include the revised Table 2.0-2 provided with the response to **RAI 03.08.04-21**. Therefore, this issue is resolved.

As discussed above, the applicant's response adequately addresses the effect of site-specific precipitation on the roof design of standard plant structures, including consideration of an antecedent ice storm or an antecedent snow pack from normal winter precipitation. Therefore, **RAI 03.08.04-14** and follow-up **RAI 03.08.04-24** are closed. The resolution of issues associated with the maximum design precipitation in Departure STP DEP 5.0-1 is acceptable.

C. Change in Shear Wave Velocity

Under Departure STP DEP T1 5.0-1, SER Subsection 3.7.1.4 provides a complete description and evaluation of shear wave velocity and its impact on the seismic design of the standard plant structures. The particular effect on the lateral soil pressures is discussed below.

Lateral Soil Pressure on Standard Plant Structures

The embedded portions of the standard plant structures are subjected to lateral static and seismic earth pressures. These earth pressures depend on factors such as local soil parameters (density, Poisson's ratio, and shear wave velocity); local soil conditions and seismicity; embedment depth; surcharges; effect of nearby structures; groundwater elevation;

and flood elevation. The lateral earth pressures act as out-of-plane forces on the embedded portion of the RB and CB walls and, therefore, must be enveloped by the design loads used in the ABWR DCD. In **RAI 03.08.04-1** the staff asked the applicant to describe how the lateral soil pressures were calculated for STP Units 3 and 4, and how the calculated lateral pressures compare with those used in the ABWR standard plant design. In the September 15, 2009, response to **RAI 03.08.04-1** (ML092610377)**,** the applicant provides a markup of a revision to COL FSAR Section 3H.2.6, "Site Specific Structural Evaluation," and states that:

- The at-rest seismic lateral earth pressures on the CB exterior walls are determined using the method described in Subsection 2.5S.4.10.5.2. In this method, the at-rest seismic lateral earth pressure computation will utilize the site-specific shear wave velocity. The impact of site-specific shear wave velocity on the design of the exterior walls is expected to be insignificant, because their designs are controlled by the combination of requirements for in-plane and out-of-plane loads.

- The at-rest seismic lateral earth pressure only affects the out-of-plane loads. The at-rest pressure includes the effect of the hydrostatic load and the surcharge load, in addition to the dynamic pressure caused by the earthquake.

- As noted in Subsection 2.5S.4.10.5.4, actual surcharge loads; structural fill properties; and final configurations of structures are not known at this time. Final earth pressure calculations are prepared at the project detailed design stage based on the actual design conditions at each structure, on a case-by-case basis.

- The applicant commits (COM 2.5S-3) to include the final earth pressure calculations including actual surcharge loads, structural fill properties, and the final configuration of structures following the completion of the project's detailed design in an update to the FSAR, in accordance with 10CFR 50.71(e).

The meaning of the term "at-rest seismic lateral soil pressure" as used in the response was not clear to the staff because the term "at-rest" is normally used in connection with static lateral earth pressure. It was also not clear to the staff how the applicant determined the adequacy of the standard plant structures for the STP site without determining the site-specific lateral soil pressures on these structures. Therefore, **RAI 03.08.04-17** asked the applicant to explain the meaning of at-rest seismic lateral pressure; to provide the lateral soil pressures for the RB and CB; and to compare the calculated pressures with those used in the ABWR standard plant design. The staff also asked the applicant to confirm whether the effects of the adjacent structures were considered when computing the lateral soil pressures and if not, to justify not doing so. In the February 10, 2010, response to **RAI 03.08.04-17** (ML100550613), the applicant explains that the term "at-rest seismic lateral earth pressure" is used to mean dynamic soil pressure for an at-rest condition (i.e., when the structure is neither moving away nor moving toward the soil). In this RAI response, the applicant also compares the site-specific lateral pressures acting on the walls of the RB and CB, with the corresponding values from the ABWR DCD, whereby the site specific lateral soil pressures are determined in accordance with COL FSAR Subsection 2.5S.4.10.5 considering the site specific SSE, and assuming the same DCD surcharge loads that include an additional surcharge from adjacent structures. The comparison shows that the total static earth pressure (H) and the design soil pressure, including SSE increment (H') for the DCD design, envelope the corresponding STP Units 3 and 4 design values. However, this comparison did not include the structure-to-structure interaction effects. The subject was discussed with the applicant during an audit in February, 2011. In the March 7, 2011, Supplement 1 response to **RAI 03.08.04-17** (ML110730067)**,** the applicant refers to a markup included with the Revision 1 response to **RAI 03.07.01-26** (ML110730067), which provides dynamic lateral soil pressure diagrams that include the SSSI (structure-soil-structure interaction) effects between the RB north wall, the CB south wall, and the TB. The diagrams compare the site-specific pressures with the DCD seismic design pressures. The comparison shows that the DCD design is conservative and envelopes the site specific conditions for the RB (Figure 3A-301) and the CB (Figure 3A-302), except for a small portion of the CB south wall between 7.93 m and 9.14 m (26 ft and 30 ft), which is not considered significant. In the March 15, 2011, Supplement 1 response to **RAI 03.08.04-30** (ML110770440)**,** the applicant provides a markup for Section 3H.1, which includes several lateral seismic soil pressure diagrams showing the SSSI effect of nearby buildings (i.e., RSW piping tunnels, RWB, UHS, DGFOSV, DGFOT, and the Crane wall) on the RB walls (Figures 3H.1-1 through 3H.1-6). During an audit held in February 2012, the applicant was asked to include in FSAR Section 3H.1.6 a discussion of the results presented in these figures and an explanation of the exceedances in the site-specific lateral seismic soil pressure in Figure 3H.1-2. In the April 10, 2012, Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant describes FSAR Figures 3H.1-1 through 3H.1-6, and evaluates the small exceedance in Figure 3H.1-2, demonstrating that they will not change the wall designs performed under DCD criteria. The staff noted that the procedures, data, and comparisons described in the applicant's responses adequately address the issues raised regarding the lateral pressures on the RB and CB, and the induced out-of-plane shear and moment due to the localized exceedance on the RB west wall and the CB south wall are enveloped by the corresponding shear and moment due to DCD soil pressures. The staff also noted that the RB and CB walls are designed for the envelope of DCD seismic lateral pressures (a) with SSSI effects according to the values in DCD Table 3A-18 ("Effect of Adjacent Buildings Enveloping Seismic Soil Pressures"); and (b) without SSSI effects according to DCD Figure 3H.1-11 ("Design Lateral Soil Pressures for RB Outer Walls") and DCD Figure 3H.1-14 ("Design Lateral Soil Pressures for CB Outer Walls") representing the design values without the SSSI effects. Therefore, the staff concluded that the applicant has adequately addressed the effect of site-specific shear wave velocity on the RB and CB wall designs, and the issue is resolved. The staff reviewed the proposed markup of FSAR Sections 3H.1.6 and 3H.2.6 provided with the response and determined that all of the information pertaining to this issue has been adequately included in the FSAR markup. Therefore, **RAI 03.08.04-1** and **RAI 03.08.04-17** (the portions related to the RB and CB) are resolved. The applicant incorporated the markups into the FSAR Revision 6.

Based on the discussion in Subparts A through C of this subsection, the resolution of issues associated with Tier 1 Departure STP DEP 5.0-1 related to this section is acceptable.

<u>*Tier 2* Departure Requiring Prior NRC Approval*</u>

- STD DEP 1.8-1 — Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

This departure identifies Tier 2* codes, standards, and regulations that are being updated to more current revisions or editions. The changes are in COL FSAR Table 1.8-20, "NRC Regulatory Guides Applicable to ABWR," and Table1.8-21, "Industrial Codes and Standards Applicable to ABWR." As code versions referenced in the STP FSAR differ from the codes used in the ABWR DCD or recommended in SRP Section 3.8.4 and RG 1.206, **RAIs 03.08.01-1, 03.08.04-33,** and **03.08.04-36** asked the applicant to provide a detailed comparison of the differences between the code editions and to justify any differences in order for the staff to evaluate the impact of using the newer codes. The staff also asked the applicant to evaluate any potential adverse impact from the provisions of the newer codes that are more restrictive or that result in a more robust design on the ABWR DCD structural design information. As a result of this departure, the following codes and standards were updated to newer revisions and/or editions:

(A) *RG 1.136* to Revision 3, in place of Revision 2. RG 1.136, Revision 3 is the current version that endorses the 2001 Edition of the ASME Code with the 2003 Addenda with certain exceptions. Therefore, the effect of this change is addressed along with a review of the newer version of the ASME Code below.

(B) *RG1.142* to Revision 2, in place of Revision 1. RG 1.142, Revision 2 is the current version that refers to ACI 349-97 as generally acceptable with exceptions and regulatory positions. Therefore, the effect of this change is addressed below along with a review of using ACI 349-97.

(C) *2006 International Building Code* in place of the 1991 Uniform Building Code. In **RAI 03.08.01-1**, the staff asked the applicant to provide a detailed comparison of the differences between these two codes as they apply to the ABWR standard design, and to justify any differences in order for the staff to evaluate the use of the 2006 IBC. In the response dated September 15, 2009, and the Supplement 1 response dated October 28, 2009, to **RAI 03.08.01-1** (ML092610377 and ML093070285, respectively), the applicant states that a detailed comparison of the two codes determined that the requirements of the 2006 IBC taken as a whole provides a margin of safety that is substantially similar to and in many cases greater than that provided by the earlier 1991 UBC. During the October 2010 audit (ML110110104), the staff reviewed the validation package responding to **RAI 03.08.01-1,** “Use of newer codes,” that presented a detailed comparison between the current 2006 IBC code and the 1991 UBC code. The package included reproduced text passages from the versions of the two codes, a comparison of each specific topic, and conclusions for application to STP Units 3 and 4. The staff reviewed the detailed code comparison for all applicable loadings and load combinations included in the package. The staff’s review concluded that the comparison demonstrated that the provisions of the 2006 IBC for various loadings and load combinations are either equivalent to or more thorough and comprehensive than the corresponding 1991 UBC provisions. For non-category I buildings required to withstand the SSE, the seismic input is based on the SSE ground acceleration instead of the provisions of the 2006 IBC, which the staff found acceptable. Therefore, the issue is resolved.

(D) *ACI 349–1997 Edition* in place of the *ACI 349–1980 Edition* building code for concrete structures. In the ABWR design certification (NUREG–1503, page 3·53), the staff evaluated the use of the 1980 Edition of ACI 349. **RAI 03.08.04-33** asked the applicant to provide a detailed comparison of the differences between these two editions of the code as they apply to the ABWR standard design, and to justify any differences in order for the staff to evaluate the acceptability of the 1997 Edition of ACI 349. The applicant’s response to this RAI dated September 15, 2010 (ML102630145), states that:

(a) The use of ACI 349–97 in lieu of ACI 349–80 is consistent with RG 1.142, Revision 2, which endorses AC1 349–97 with some additional or alternate requirements stated in the regulatory positions that STP Units 3 and 4 is committed to follow.

(b) A detailed review was performed of the differences between ACI 349–80 and ACI 349-97, as they apply to the design and analysis of the safety-related ABWR standard design.

(c) Generally, revisions (a) provide expanded explanations of the code requirements to eliminate possible misinterpretations or to identity specific instances where the code section applies or does not apply, (b) incorporate provisions based on more current research or experience, or (c) expand provisions to address new types or methods of construction that were not clearly allowed or disallowed in earlier revisions.

(d) The most significant changes—although not all of them—are applicable to the ABWR standard design and are found in the following chapters:

- Chapter 9, "Strength and Serviceability Requirements," of ACI 349–97 contains changes in Section 9.5 pertaining to the calculation of long-term deflections. These changes simplify calculations of deflection magnification factors and allow for the determination of the deflection at different time periods. Because the design is not expected to be governed by deflection controls, these changes will not affect the ABWR standard design.

- Chapter 10, "Flexure and Axial Loads," of ACI 349 includes changes in Section 10.6 that are based on more recent experience. These changes are therefore improvements to replace provisions that were determined to be inadequate. Changes in the other sections address more recent construction practices and experience and will result in no change in or more conservative design margins.

- Chapter 11, "Shear and Torsion," includes a large number of changes that are mostly additional provisions or are based on more recent research results and experience. None of these changes will reduce the design margins for the ABWR standard design.

- Chapter 12, "Development and Splices of Reinforcement," includes a large number of changes. Most of the changes are to provisions that address epoxy coated rebar (the ABWR does not use epoxy coated rebar) and are also revised provisions for reinforcement development length. These changes are based on more recent extensive research results and experience and generally increase the lengths for development.

- Chapter 21, "Special Provisions for Seismic Design," was added to ACI 349–97. This chapter provides requirements for analyzing and designing seismic loading. They are intended to improve the toughness of the structure and to assure that the integrity of the structure is retained even under inelastic deformations due to earthquake events. These provisions are based on more current research and experience and represent the state of the art at the time of the code revision. They will therefore result in more robust structures.

- Appendix B, "Steel Embedments," includes changes based on later research that are discussed below.

The staff's review noted that the applicant has provided a summary of the differences between ACI 349–80 and ACI 349–97 that highlights the changes in the provisions. In some cases, the newer code provides more conservative designs. In other cases, the newer code may give slightly smaller size rebars by increasing the shear strength (i.e., in Subsection. 11.7.4.2) but does not change the intended design margin. As an example, new special provisions for brackets and corbels (paragraphs 11.9.1, 11.9.3.2.1, and 11.9.4) result in corbel designs that are at least as safe or more conservative. In a few cases, the provisions in the newer code do not apply to ABWR designs (e.g., pre-stressed slabs). Changes in the requirements for two way slab systems will not have any impact, because the finite element computer analysis will be used for the major structural elements of the ABWR design. The staff examined the applicant's detailed comparisons of the two versions of the code during the audit in October 2010. The staff found the applicant's comparative analysis to be complete and accurate, but it lacked information on how the more restrictive provisions will affect the DCD design. The staff addressed this issue in **RAI 03.08.04-36,** which is discussed later in this section. RG 1.142, Revision 2 endorses ACI 349–97. The staff reviewed the comparison of the two versions of the code and found the use of the proposed edition acceptable. However, RG 1.142, Revision 2, does not endorse Appendix B, "Anchorage to Concrete," of ACI 349–97. Therefore, it is not clear from the applicant's response which provisions were used for steel embedments for both the standard plant structures and the site-specific structures. In the July 27, 2011, Supplement 1 response to **RAI 03.08.04-33** (ML11213A094), the applicant clarifies this issue by stating that COL FSAR Table 1.9S-1 already commits to RG 1.199, "Anchoring Components and Structural Supports in Concrete," for the design of site-specific SSCs. The applicant also commits to use RG 1.199 for the design of standard plant SSCs and provides a markup for FSAR Section 1.8, Table 1.8-20 and COL application Part 7, Departure STD DEP 1.8-1 to include references to RG 1.199. The applicant also deleted DCD Table 3.8-10, "Staff Position on Steel Embedments," so it will not be incorporated into the FSAR. The staff's review noted that RG 1.199 is endorsed by SRP Section 3.8.4, SRP Acceptance Criterion 4 (A) as applicable to the anchor types discussed in Appendix B of ACI 349–01. Also, the commitment to use RG 1.199 for anchoring components and structural supports in concrete overrides the provisions in DCD Table 3.8-10, which were intended to supplement an older version of ACI 349. For these reasons, the staff found the applicant's response acceptable and adequate. In addition, the FSAR was subsequently revised to incorporate the proposed changes. Therefore, this issue is resolved.

(E*) ANSI/AISC N690–1994 Edition* in place of the *N690–1984* Edition building code for steel structures. FSAR Subsection 3H.6.4.1 references ANSI/AISC N690 specifications for the design, fabrication, and erection of site-specific seismic Category I steel structures. This subsection does not specify which version of the specifications the applicant is using for structures. ABWR DCD Table 1.8-21, which the applicant incorporates by reference, specifies the 1984 edition of the standard; but it is not clear if the applicant is also using this same version of the standard for site-specific structures. According to the guidance in RG 1.206, Regulatory Position C.I.1.9.2, the applicant should use the current SRP for structures outside of the scope of the ABWR DCD or provide a justification for not doing so. SRP Section 3.8.4, SRP Acceptance Criterion 5 refers to ANSI/AISC N690–1994—including Supplement 2 (Edition 2004)—as accepted by the staff for the design, fabrication, and erection of safety-related steel structures. **RAI 03.08.04-33** asked the applicant to provide a detailed comparison of the differences between the 1984 edition and the 1994 edition of the specifications as they apply to the site-specific seismic Category I structures at the STP site, and to justify any differences in order for the staff to evaluate the acceptability of the 1984 edition of the specifications. In the response to **RAI 3.08.04-33** (ML102630145), the applicant agrees to comply with the guidance in RG 1.206, Regulatory Position C.I.1.9.2 and to use SRP Section 3.8.4, Revision 2, for structures outside the scope of the ABWR DCD, which accepts ANSI/AISC N690–1994 with Supplement 2 (2004). The response also includes a markup for COL FSAR Table 1.8-21a, "Codes and Standards for Site-Specific Systems," and adds ANSI/AISC N690–1994 to the table as one of the codes to be used for site-specific structures. The staff's evaluation noted that the proposed edition of ANSI/AISC N690 is applicable to site-specific SSCs only and as such, it does not affect the design of any standard structures. Since the proposed edition of the steel code meets the guidance in SRP Section 3.8.4, Revision 2, which is the current version of the

SRP for structures outside the scope of the ABWR DCD, the staff found the use of the proposed edition acceptable. The FSAR was subsequently updated to incorporate the proposed changes to COL FSAR Table 1.8-21a. Therefore, this issue is resolved.

(F) *ASME Code: 2001 Edition with the 2003 Addenda of Section III, Division 2,* in place of the 1989 Edition of the same code. The staff observed that Table 1.8-21 in FSAR Section 1.8 references ASME Code Section III, Division 2, Edition 2001 with the 2003 addenda and identifies certain limitations. The ABWR DCD specifies the use of ASME Code Version 1989. In NUREG–1503 page 3-49, the NRC accepted the 1989 Edition of the ASME Code, Section III Division 2. **RAI 03.08.04-33** asked the applicant to provide a detailed comparison of the differences between these two editions of the code, as they apply to the design and analysis of safety-related ABWR standard plant structures, and to justify any differences in order for the staff to evaluate the acceptability of ASME Code Section III, Division 2, Edition 2001 with the 2003 addenda. The applicant's response to **RAI 03.08.04-33** (ML102630145) states that Revision 3 of RG 1.136 endorses ASME Code Section III, Division 2, 2001 Edition with the 2003 Addenda. The applicant also states that additional requirements in the DCD regarding the containment design (e.g., Table 3.8-2) will be implemented in the design and are not affected by this code edition change. Furthermore, the response also notes that the applicant has performed a detailed review of the differences between the 1989 Edition and the 2001 Edition with the 2003 addenda of ASME Code Section III, Division 2, as they apply to the design and analysis of the safety-related ABWR containment structure. The applicant also includes a list of code enhancements, corrections, or changes to Sections CC-3400, CC-3500, CC-3700, and CC-3800. The applicant justifies why these modifications do not impact the ABWR designs. In addition, the applicant notes that there no identifiable changes to the load categories and load combinations in Sections CC-3220 and CC-3230.

During the audit in October 2010 (ML110110104), the staff examined the applicant's detailed comparison between the changes in the provisions of these two versions of the ASME Code. The staff concluded that the applicant has adequately identified the differences between the two versions of the ASME Code and their impact on the design. The staff noted that the applicant identifies several areas in the newer version of ASME Section III, Division 2, where provisions of the newer code are either more restrictive or may result in requiring a more robust design. This uncertainty may potentially result in a design that if an earlier version of the ASME Code is used, the design would not meet the provisions of the newer version of the Code. Any departure must be evaluated against other information in the FSAR—including the DCD sections incorporated by reference—to determine whether the departure is acceptable and consistent with the rest of the FSAR, and if applicable, to make any appropriate changes. However, the applicant does not demonstrate how the design information included in the ABWR DCD that is incorporated by reference is affected by the provisions of the newer code that are more restrictive or that result in a more robust design. As stated earlier, the staff identified the same issue with the code comparison of the ACI 349–97. In **RAI 03.08.04-36,** the staff asked the applicant to evaluate any such potential adverse impact from the provisions of the newer codes on the ABWR DCD structural design information. In the April 5, 2011, response to **RAI 03.08.04-36** (ML110980401)**,** the applicant presents an evaluation table that lists the potentially affected DCD sections and the impact of code changes on the design results included in each section. The applicant concludes that there is no adverse effect on any of the design results documented in the ABWR DCD. The staff's review noted that the provisions regarding the loading parameters, load combinations, load and intensity factors, strength reduction factors, allowable stresses and strains for rebars and concrete, and minimum or maximum reinforcing limits have either undergone only minor modifications between the code editions or the changes do not affect the governing design conditions. Consequently, the provisions do not affect the structural design information in the ABWR DCD. The newer code significantly changed the determination of the development length of reinforcing bars. However, that detailed information is not included in the DCD and will be considered in the detailed design phase when provisions of the newer version of the code are used. Based on the above information, the staff concluded that the applicant has adequately addressed the impact of changes in the versions of the ACI and the ASME Codes on the ABWR DCD design.

RAI 03.08.04-33 also asked the applicant to explain how the use of the Edition of the ASME Code that the applicant proposes to use meets the provisions of NCA-1140, "Use of Code Editions, Addenda and Cases." In the response to **RAI 03.08.04-33** (ML102630145), the applicant states that the proposed change meets the provisions of Paragraphs NCA-1140 (a) (1) and (a) (2), which reads as follows:

> (a) (1) Under the rules of this Section, the Owner or his designee shall establish the Code Edition and Addenda to be included in the Design Specifications. All items of a nuclear power plant may be constructed to a single Code Edition and Addenda, or each item may be constructed to individually specified Code Editions and Addenda.
>
> (a) (2) In no case shall the Code Edition and Addenda dates established in the Design Specifications be earlier than:
>
>> (a) 3 years prior to the date that the nuclear power plant construction permit application is docketed; or
>>
>> (b) The latest edition and addenda endorsed by the regulatory authority having jurisdiction at the plant site at the time the construction permit application is docketed."

The staff's review noted that in order to meet articles (a) and (b) of NCA-1140, the ASME Code Edition and Addenda should not be dated older than either 3 years from the docketing date of the COL application or the latest edition and addenda endorsed by NRC on the COL application docketing date. Since the 2001 edition with the 2003 addenda of ASME Code Section III, Division 2 is the latest endorsed version in RG 1.136, Revision 3, the proposed version is in compliance with NCA-1140 Article NCA-1140(a)(2)(b) and is thus acceptable.

As discussed above, the applicant has provided an adequate response to all of the staff's questions in **RAI 03.08.04-33** and **RAI 03.08.04-36** pertaining to Tier 2* Departure STD DEP 1.8-1. In addition, the applicant has incorporated the pertinent changes into the FSAR. Therefore, **RAI 03.08.04-33** and **RAI 03.08.04-36** are closed.

Based on the above, the resolution of issues associated with Tier 2* Departure STD DEP 1.8-1 related to this section is acceptable.

<u>*Tier 2 Departures Not Requiring Prior NRC Approval*</u>

- STD DEP 12.3-3 Steam Tunnel Blowout Panels

This departure corrects the description of the steam tunnel blowout panels in DCD Section 3.8.4 and Subsections12.3.1.4.4 and 12.3.2.3. The inconsistency in the DCD involves a description of blowout panels between the steam tunnel and the RHR pump and heat exchanger room. Section 3.8.4 of the DCD erroneously describes a high-energy line break postulated in the RHR compartment, while the ABWR sub-compartment pressurization analysis summarized in Tables 6.2-3 and 6.2-4a states that there is no high-energy line present in the RHR pump and heat exchanger room. Thus, there is no need for the blowout panels. This room is a vent path in the ABWR sub-compartment pressurization analysis model that Tables 6.2-3 and 6.2-4a identify as Node SA2. The applicant has removed the incorrect or ambiguous description of any blowout panels connecting the RHR room with the steam tunnel. Therefore, the staff found the changes in Section 3.8.4 reasonable, and the departure does not require prior NRC approval. The assessment of this departure on the changes in Sections 12.3.1 and 12.3.2 is documented in Chapter 12 of this SER.

The applicant's evaluation determined that this departure does not require prior NRC approval in accordance with 10 CFR Part 52, Appendix A, Section VIII.B.5. Within the scope of this review, the staff found it reasonable that the departure does not require prior NRC approval. The applicant's process for evaluating departures and other changes to the DCD is subject to NRC inspections.

- STP DEP 3.5-2 Hurricane Generated Missile Protection

Departure STP DEP 3.5-2 modifies DCD Tier 2 Subsection 3.5.1.4 to incorporate the guidance in RG 1.221. Some of the hurricane parameters exceed the tornado-based parameters used in the design of the standard plants. FSAR Section 3H.11 discusses the effects on the RB and CB. Subpart 3 in Subsection 3.8.4.4.5 evaluates this departure below.

COL License Information Item

- COL License Information Item 3.26 Identification of Seismic Category I Structures

In FSAR Subsection 3.8.6.4, the applicant references DCD Table 3.2-1 for a complete list of seismic Category I SSCs that includes the following site-specific seismic Category I structures (except for the DGFOT):

- UHS/RSW pump house
- RSW piping tunnel
- DGFOSV

A detailed description of these structures is in FSAR Section 3H.6 for the UHS pump house and RSW piping tunnel and in Section 3H.6.7 for the DGFOSV. In addition, applicant provides details of the DGFOT in FSAR Section 3H.7.

During a review of these sections, the staff noted several issues affecting all site-specific structures, in addition to some issues belonging specifically to each structure mentioned above. The following subsection first discusses topics related to all Category I structures, followed by topics related specifically to each one of the site-specific structures.

3.8.4.4.2 Evaluation of Seismic Category I Structures

A. Evaluation of Design Issues Related to All Category I Structures

FSAR Subsection 3H.6.6.1, “Structural Analysis and Design Summary,” states that the applicant will perform a structural analysis of the UHS/RSW pump house building and the RSW tunnels. At the time of this review, the applicant had not yet performed these analyses. Consequently, there were no final design details or results for these structures in the application. **RAI 03.08.04-13** and follow-up **RAI 03.08.04-23** and **RAI 03.08.04-30** asked the applicant to include in the FSAR the structural analysis and design information for all site-specific seismic Category I structures using the guidance in SRP Section 3.8.4, in addition to other applicable SRP sections and guidance documents. Several other RAIs included questions that are also applicable to all seismic Category I structures. The issues pertaining to all Category I structures are discussed below.

A.1 *Soil Spring Values*

RAI 03.08.04-13 asked the applicant to include in the FSAR the structural analysis and design information for all site-specific seismic Category I structures using the guidance in SRP Section 3.8.4, in addition to other applicable SRP sections and guidance documents. In the August 20, 2009, response to this RAI (ML092370556), the applicant refers to a markup of FSAR Section 3H.6 that is enclosed in the response to **RAI 03.07.01-13** (ML092360772). The staff noted that the level of detail included in FSAR Subsection 3H.6.6.3 regarding the structural design of the various elements of site-specific structures was not sufficiently descriptive and was not similar to comparable information included in the ABWR DCD. Therefore, follow-up **RAI 03.08.04-23** requested the applicant to include in FSAR Subsection 3H.6.6.3 a description of the various steel and concrete elements of the site-specific structures, including how these elements are designed.

In the February 10, 2010, response to **RAI 03.08.04-23** (ML100550613), the applicant refers to additional information in the Supplement 1 and 2 responses to **RAI 03.07.01-13** (ML093270047 and ML100050225, respectively), which included the description of the finite element models used in the design and the design results. The response to **RAI 03.08.04-23** also describes how the forces in the structure caused by differential settlements resulting from the flexibility of the basin and the pump house supporting soil are accounted for through the use of foundation soil springs in the FEA model, including the methodology for computing the soil springs for static and seismic loading. The unit static soil springs (called the modulus of subgrade reaction k) are calculated as the ratio of the applied foundation stress (contact pressure under the foundation) and the calculated settlement of the structure given in FSAR Table 2.5S4-42. These ratios are computed for the nine locations given in Table 2.5S4-42 within the foundation footprint. For design purposes, a uniform value corresponding to the average of the nine locations is assigned to each foundation. These springs are used for all static load cases. The seismic loading condition used algebraic formulas to compute the soil spring constants based on the half-space theory, where the foundations are assumed to be prismatic and rigid and embedded into the homogenous soil. The algebraic equations used to calculate the spring constant of a foundation require a single value of soil modulus (and Poisson's ratio) as input. Soil under STP Units 3 and 4 is not uniform and homogeneous but consists of multiple layers, each with a shear modulus specific to the layer. Therefore, an equivalent shear modulus representing the layered subsoil was determined as a weighted average in which the weights represent the accumulated strain energy in each layer. The average Poisson's ratio is computed as a layer-weighted value. For horizontal springs, the same values are used for static and dynamic loadings. Seismic soil springs are calculated for three sets of values representing lower, mean, and upper bound soil properties. The lower-bound values are used in the structural analyses.

The staff's evaluation determined that the applicant's methods for establishing the static and seismic spring constants are technically acceptable, because they follow engineering practices accepted by the industry and documented in text books and in published technical papers. However, the staff needed further clarification regarding the adequacy of using average uniform spring constants across the entire foundation and not considering frequency dependence of the dynamic springs used for the seismic analysis. Therefore, **RAI 03.08.04-30** asked the applicant to provide the nine subgrade modulus values and to explain why it is considered appropriate to use the average value. In addition, the staff asked the applicant to explain how the foundation subgrade modulus was used to calculate the nodal springs for the FEA model, and how the effect from the coupling of the soil springs was considered in the analysis (item 2 in RAI 03.08.04-30). The staff also asked the applicant to justify not considering the effects of pressure distribution and system frequency in developing the foundation dynamic springs (item 3 in RAI 03.08.04-30), including a description of the impact on the calculated results.

In the Supplement 1 response to **RAI 03.08.04-30** (ML110770440) dated March 15, 2011, the applicant refers to a detailed response in the Supplement 1 response to **RAI 03.08.05-4** (ML103230128)**.** In the response to **RAI 03.08.05-4,** the applicant states that it is customary to use average values of the subgrade modulus in the design whenever the foundation subgrade values vary within the foundation perimeter. The applicant also states that ground stiffness is commonly modeled by uniform, uncoupled Winkler springs. To more realistically simulate the observed soil stress distribution under a foundation, a non-uniform spring distribution (pseudo-coupled) was also used in the design in which different subgrade modulus values are assigned to different areas (zones) of the mat foundation. In regard to the influence of soil layers with varying properties with depth, the applicant mentions that the settlement of the foundation at any location is determined by the combined compression of many soil layers below that location. Variations in the properties or thickness of individual layers do not cause significant differences in the computed settlement and modulus of a subgrade reaction between corresponding points on the foundation. Therefore, variations in the modulus of a subgrade reaction due to variations in the soil layers from point to point beneath the large foundations of the STP Units 3 and 4 structures are insignificant, in terms of their influence on the design of the basemats for these structures. Regarding the effects of pressure distribution and frequency dependence on soil spring values, the applicant states in the Supplement 1 response to **RAI 03.08.04-30** (ML110770440) that the seismic loadings are calculated using an equivalent static method that considers the maximum accelerations from the SSI analysis; the pseudo-coupled soil springs case thus accounts for the pressure distribution effects.

The staff's evaluation found that the applicant has adequately addressed and clarified the concerns regarding the determination of soil springs, and the subgrade moduli computed from settlement data already include the effects of deep layers. The staff concluded that the pseudo-coupled approach is a practical and adequate approximation for considering coupling between soil springs, and this approach is recommended in ACI 336.2R, "Suggested Analysis and Design Procedures for Combined Footings and Mats," which is a widely accepted industry standard. The staff also agreed that the frequency dependence of springs need not be considered in the equivalent static method using accelerations obtained from the SSI analysis. Therefore, this issue is considered resolved.

A.2 Combination of Static and Dynamic Spring Models

Section A.1 above describes how static and seismic soil springs are determined and applied to the finite element model of the UHS/RSW pump house building. Because static and seismic soil springs differ in value, **RAI 03.08.04-30** under Item 4 asked the applicant to describe how the static and seismic soil springs are inputted into the FEA model and how the results from static and dynamic models are combined for stress evaluations.

Regarding item 4 in the Supplement 1 response to **RAI 03.08.04-30** (ML110770440)**,** the applicant states that two SAP2000 3-D FEA models are used to calculate the element design forces: one model for short-term loading (seismic) and one model for long-term loading (non-seismic). The only difference between the two FEA models is the application of the loading and soil springs in the global Z (i.e., vertical) direction. The stiffness of the soil springs for both the short-term loading and long-term loading models is determined by multiplying the corresponding foundation subgrade modulus for the short-term and long-term loading by the tributary area of mat elements for each spring. The resulting element forces from the short-term loading model for X, Y, and Z seismic loads are combined using the SRSS method. These SRSS element forces constitute the E' term in the third and fifth load combinations in COL FSAR Subsection 3H.6.4.3.4.3. The element forces that comprise the E' term are added to and subtracted from the other applicable resulting element forces from the long-term loading model in the load combinations defined in COL FSAR Subsection 3H.6.4.3.4.3, in a database outside of the FEA model to determine the final element design forces for each load combination. Because both the accidental torsional moment and the soil loads (H') are directional in nature, they are added algebraically to the seismic load combinations.

The staff found that the procedure used by the applicant is an appropriate method for the determination and superposition of the static and dynamic loadings. Therefore, this issue is resolved.

A.3 Minimum Coefficient of Friction

RAI 03.08.04-30 under Item 9 asked the applicant to clearly specify whether the minimum coefficient of friction is different at various locations of the site, and if the site coefficients differ, to explain how these values were determined. The applicant's Revision 1 response to **RAI 03.08.04-30** (ML110770440) dated March 15, 2011, regarding friction coefficients under "Sliding Stability Evaluations"; and the Revision 1 response to **RAI 03.08.04-28** Items 3 and 4 (ML110730067) dated March 7, 2011, defines the following friction coefficients at the interface between concrete and soil, as derived from the soil friction angles given in FSAR Section 2.5S.4 whereby the dynamic coefficients correspond to two-thirds of the static coefficient (see Item 9 under COL License Information Item 3.23 in Subsection 3.8.5.4 of this SER for a discussion of dynamic friction coefficients under "Friction between Gravel and Soil"):

- UHS/RSW pump house: available static coefficient of friction = 0.70
- DGFOSV: available static coefficient of friction = 0.58; dynamic = 0.39
- CB: available static coefficient of friction = 0.70; dynamic = 0.47
- RB: available static coefficient of friction = 0.70; dynamic = 0.47
- For the waterproofing membrane, the minimum available static coefficient is specified as 0.60 (later 0.75; see below)

During the audit in March 2011 (ML111320094), the staff brought to the attention of the applicant that sliding may occur first at the interfaces with the lowest coefficient of friction. The static coefficient of friction at the waterproof membrane is 0.60, which is less than 0.70 at the mud-mat soil interface and may initiate sliding before sliding occurs at the mud-mat-soil interface. The same is also true for the interface between hardened concrete and freshly poured concrete that may have a coefficient of friction less than 0.70 depending on surface roughness of the hardened concrete. In the April 25, 2011, Supplement 1 response to **RAI 03.08.04-19** (ML11119A077), the applicant addresses the issue by specifying a static coefficient of friction of 0.75 minimum for both the waterproof membrane and the concrete-to-concrete interface. The site-specific ITAAC for the waterproof membrane (Table 3.0-13) proposed in the Revision 1 response to this RAI (ML101250162) was revised accordingly in the FSAR markup submitted with the Supplement 1 response. However, the applicant did not include in the FSAR markup the requirement for intentional roughening of the hardened concrete surface needed to achieve the minimum 0.75 coefficient of friction. The applicant agreed to include the requirement in the FSAR and subsequently provided an FSAR markup in the Supplement 2 response to **RAI 03.08.04-19** (ML11168A168) that was acceptable to the staff. The FSAR was subsequently updated to incorporate the proposed changes provided with the response. Thus, this issue is resolved.

A.4 *Loads and Load Combinations*

a) *List of Loads for Site-Specific Structures*

In FSAR Subsection 3H.6.4.3, “Design Loads and Load Combinations,” the applicant did not include any descriptions of the thermal loads, loads due to the probable maximum flood, hydrostatic loads, and calculated lateral soil pressures used for the design of site-specific Category I structures. **RAI 03.08.04-12** and follow-up **RAI 03.08.04-22** and **RAI 03.08.04-29** asked the applicant (a) to include the above information in FSAR Subsection 3H.6.4.3; (b) to quantify these loads by including their actual numerical value; (c) to describe and evaluate the different thermal conditions; and (d) to use the dynamic lateral earth pressure as well as the full live loads in load combinations that include seismic loads. The applicant provides explanations and FSAR markups that include the requested corrections, clarifications, and quantifications regarding the different loadings; load combinations; load cases; load parameters and amplitudes; soil pressure diagrams; durability and load factors; and stability safety factors that were considered in the design in the following RAI responses: the response to **RAI 03.08.04-12** dated August 20, 2009 (ML092370556); Revision 1 response to **RAI 03.08.04-22** dated September 15, 2010 (ML102630145); response to **RAI 03.08.04-29** dated September 15, 2010 (ML102630145); and Supplement 1 response to **RAI 03.08.04-29** dated March 7, 2011 (ML110730067), which also refers to **RAI 03.04.02-11** dated September 15, 2010 (ML102630145), and **RAI 03.07.01-13** (ML092360772) dated August 20, 2009. As a result, a number of COL FSAR sections were revised as described below:

Subsection 3H.6.4.3: The following complementing information was added regarding dead and live, snow, static and dynamic lateral soil pressures, thermal, hydrostatic, wind and tornado, flood, SSE, extreme flood and extreme snow loads, as well as structural steel and reinforced concrete load combinations.

Subsection 3H.6.4.3.1.4: Lateral static soil pressure diagrams for the design of the UHS and the RSW tunnels and for stability analyses of the UHS were added.

Subsection 3H.6.4.3.1.5: Temperatures and six thermal cases to represent summer and winter operating and accident conditions for the UHS design were added.

Subsection 3H.6.4.3.1.6: For the UHS design, hydrostatic load parameters to consider the effects of the basin water were added.

Subsection 3H.6.4.3.3.3: Lateral dynamic soil pressure diagrams for the design of the UHS and the RSW tunnels and for UHS stability analyses were added.

Subsection 3H.6.4.3.3.4: Design parameters representing the extreme environmental flood were added.

Subsection 3H.6.4.3.4: A clarification was added to state that no high-energy line breaks are associated with the site-specific Category I concrete structures, and the corresponding load components therefore do not need to be considered.

Subsection 3H.6.4.3.4.3: Load factors in reinforced concrete load combinations were adjusted to match the load combinations in ACI 349. Furthermore, load combinations were corrected to consider full live load and dynamic lateral soil pressures in seismic load cases.

Subsection 3H.6.4.3.4.4: This section was added to the FSAR to explicitly consider the environmental requirements of ACI 350 load factors and load combinations.

The staff's evaluation concluded that the applicant has satisfactorily responded to and clarified the raised issues by complying with the code requirements in ACI 349, ACI 350, and ANSI/AISC N690 and otherwise explaining, complementing, and/or correcting the design procedures per the staff's requests. Therefore, **RAI 03.08.04-12, RAI 03.08.04-22,** and **RAI 03.08.04-29** are closed. The applicant has subsequently updated the FSAR to incorporate the proposed changes provided with these responses.

b) *Importance Factor used for Wind Loading*

During the audit reviews in October 2010 the staff noted that the Importance Factor (I) used to determine wind pressures on buildings was set to I = 1 instead of I = 1.15, as specified in the FSAR. However, I = 1.15 was used for tornado wind loads. The use of I = 1.15 in addition to a 100-year return period wind is recommended in SRP Section 3.3.1. Thus, the staff asked the applicant to justify the value used for the Importance Factor. In the Supplement 1 and Supplement 3 responses to **RAI 03.08.04-30** dated March 15, 2011, and June 16, 2011 (ML110770440 and ML11168A168, respectively)**,** the applicant states that the Importance Factor is used in ASCE 7–05 to adjust the mean recurrence interval (MRI) of maximum wind speeds expected at the site, and the Importance Factor I = 1.15 converts the 50-year MRI to the 100-year MRI winds, which is in accordance with SRP Sections 2.3.1 and 3.3.1 that require the wind design to be based on 100-year winds. The velocity pressure equation in ASCE 7–05, which is also recommended in SRP Section 3.3.1, yields the same results using the combination of I = 1.0 and 100-year winds; or using I = 1.15 and 50-year winds. These responses also include proposed FSAR markups for Subsection 3H.6.4.3.2. The staff's evaluation noted that SRP Section 2.3.1 recommends that the design be based on 100-year MRI winds, while SRP Section 3.3.1 recommends using a factor of I = 1.15 to obtain the velocity pressures. Using the Importance Factor of I = 1.15 in combination with 100-year winds would result in an overly conservative design based on wind speeds with a MRI of about 200 years (per ASCE 7–05, Table C6-7). Because using 100-year winds as the design basis meets the guidance in SRP Section 2.3.1, the staff found the applicant's procedure satisfactory and acceptable. Therefore, this issue is resolved. FSAR Subsection 3H.6.4.3.2 was subsequently updated to incorporate the proposed changes provided with the response.

c) *Consideration of 25 Percent Live Loads*

In FSAR Subsection 3H.6.4.3.1.2, the applicant states that "for computation of global seismic loads and the definition of load combinations that include seismic loads, the live load is limited to the expected live load present during normal plant operation, Lo. This load has been defined as 25 percent of the operating floor and roof live loads." SRP Section 3.7.2, SRP Acceptance Criterion 3.D recognizes the use of 25 percent of the floor design live load in the dynamic model for computing global seismic loads only. **RAI 03.08.04-9** and follow-up **RAI 03.08.04-20** asked the applicant to provide a detailed justification explaining why seismic load combinations for the design of seismic Category I structures need to consider only the normal plant operating condition, when only 25 percent of the design live load is assumed to be present. The RAI also, requested the applicant to describe the basis for the assumption that only 25 percent of the design live load is present during normal plant operation and to demonstrate that the assumption meets industry standards. In the responses to **RAI 03.08.04-9** dated September 3, 2009 (ML092510038) and **RAI 03.08.04-20** dated February 4, 2010 (ML100480204), the applicant states that live loads are applied only at the operating floors and roof of the pump house and the floors of the RSW piping tunnels. Furthermore, that 25 percent of the live load is added to the dead loads for computing global seismic inertial loads (E'), and the full live load (L) is used on all load combinations (LC) whether the term E' is included in that particular LC or not. Corresponding markups to change FSAR Subsections 3H.6.4.3.4.2 and 3H.6.4.3.4.3 are included in the response. The staff's evaluation found the applicant's response adequate, because the proposed procedure meets the recommendations of SRP Section 3.7.2 regarding the application of live loads for determining global seismic loads and the provisions of ACI 349-97 for considering live loads that include seismic load cases for the design. Therefore, this issue is resolved and **RAI 03.08.04-9** and **RAI 03.08.04-20** are closed. In addition, the proposed changes to FSAR Subsections 3H.6.4.3.4.2 and 3H.6.4.3.4.3 were incorporated into a subsequent FSAR revision.

d) *Combination of Forces and Moments*

In **RAI 03.08.04-30, Item 5** asked the applicant to describe the method of combining the resultant section forces and moments for the concrete design, and to demonstrate that the method will yield the worst combination of forces and moments. In the Supplement 1 response to **RAI 03.08.04-30** dated March 15, 2011 (ML110770440), the applicant describes the procedure for combining forces and moments to determine the reinforcement for walls, slabs, columns, and beams of the UHS/RSW pump house building. The applicant states that each element of the finite element model within a given reinforcing zone is verified for every load combination, including all possible load permutations, for all simultaneously occurring section forces and moments. Thus, although the design is performed for all possible load combinations, the tables in the FSAR show only a few selected load combinations. In FSAR Subsection 3H.6.4.3.1.5, the applicant states that the thermal gradient and the thermal axial loads are applied to the UHS/RSW pump house FEA model for six separate thermal conditions. Thermal gradients are applied to the FEA model to evaluate mechanical shear effects on the structural elements. Separate thermal load combinations are created that include the thermal axial loads. These combinations are considered in the required reinforcement analysis of out-of-plane moment and axial force couples. Therefore, the axial force and moment interaction analysis is run for mechanical loads only, without considering thermal gradient effects. The in-plane shear and out-of-plane shear include thermal gradient loads. After the final reinforcement is determined for the axial force and moment pairs combined with the maximum in-plane shear, a confirmatory cracked section analysis is performed with the computer program TEMCO considering thermal gradients and non-thermal axial and flexural loads. By allowing the concrete to crack and the reinforcement to yield, this analysis relieves the thermal moment. However, concrete and rebar strains are verified to stay within allowable limits. The staff found the described design procedure adequate because all possible load combinations, including the effects of thermal expansion and thermal gradient, are considered in the design. This ensures that the design is conducted considering the worst possible load combination. Therefore, this issue is resolved.

As a result of the February 2012 audit (ML120660018), **RAI 03.07.02-13** asked the applicant to evaluate the impact of the varying ambient temperatures on the final design of other site-specific structures. In the Supplement 5 response to **RAI 03.07.02-13** dated April 10, 2012 (ML12103A369), the applicant states that the explicit consideration of thermal loads in the design is not warranted unless the structure is subject to severe temperatures—such as those resulting from an accident—or the structure requires special crack control provisions, and none of the site-specific structures other than the UHS is subjected to accident temperatures or is required to meet any special crack control requirements. The applicant refers to ACI 349.1R-07, "Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures," and states that there is no need for an explicit thermal analysis. The applicant cites the following reasons from the above standard:

- "Stresses resulting from thermal effects are generally self-relieving, that is thermal forces and moments are greatly reduced or completely relieved once concrete cracks or reinforcement yields; as a result, thermal effects do not reduce the strength of a section for mechanical loads."

- "It is known that ambient thermal effects, in all but very unusual situations (for example major fire events), will have little effect on the ultimate strength of a concrete structure."

- "A uniform temperature change (Tm-Tb) of 50^0 F (28^0 C) or less need not be analyzed. Such a temperature change may cause up to about 0.0003 in/in (0.0003 mm/mm) strain, which is only 10% of the maximum design concrete strain of 0.003."

- "Thermal gradients should be considered in the design of reinforcement for normal conditions to control cracking. Thermal gradients less than approximately 100^0 F (56^0 C) need not be analyzed because such gradients will not cause significant stress in the reinforcement or strength deterioration."

- "In nuclear power structures, the controlling load combinations are generally those that include Eo or Ess. These load cases provide sufficient reinforcement to control cracking."

  where,

  Tm = mean temperature
  Tb = base (stress-free) temperature
  Eo = load effects of OBE
  Ess = load effects of SSE

The applicant adds that considering a base temperature of 15.6 degrees C (60 degrees F) under ambient conditions, the uniform temperature changes and the thermal gradients for these structures are less than 28 and 56 degrees C (50 and 100 degrees F) respectively. The response also includes FSAR markups for Subsections 3H.3.4.3, 3H.6.4.3.1.5, 3H.6.7.1, and 3H.7.4.3. The staff found the applicant's justification for not considering thermal effects on other site-specific structures to be technically acceptable because it follows the recommendations of

ACI 349.1R-07, which is an appropriate industry standard for considering thermal loads for nuclear power plant structures. Therefore, this issue is resolved. The applicant incorporates the proposed changes into the FSAR Revision 8.

e) Lateral Soil Pressures

In **RAI 03.08.04-12** and **RAI 03.08.04-22** and in a closed meeting with the applicant on February 2 and 3, 2011 (ML12352A207), the staff raised the question regarding the lateral earth pressure design for the site-specific Category I structures. In the Supplement 2 response to **RAI 03.07.01-13** dated December 30, 2009 (ML100050225), and Supplement 1 and 2 responses to **RAI 03.08.04-17** dated March 7, 2011, and April 25, 2011, (ML110730067 and ML11119A077, respectively) the applicant provides markups to COL FSAR Sections 3H.6, 3H.6.7, and 3H.7 for the UHS and RSW pump house; the RSW piping tunnels; and the DGFOSV and the DGFOT that include the numerical soil parameters used in the design; and the at-rest, dynamic at-rest, active, and passive soil pressure profiles without SSSI effects. In addition, the SSSI incremental seismic soil pressure diagrams and the effects of the nearby heavy structures on smaller structures such as the RSW piping tunnels, the DGFOSV, and the DGFOT are included in the following responses:

- the UHS/RSW pump house, in the response to **RAI 03.07.02-22** (ML110770440)
- the DGFOSV, in the Revision 1, Supplement 1 response to **RAI 03.07.01-27** (ML110730067)
- the DGFOT, in the Revision 1 response to **RAI 03.08.04-30** (ML110770440) and the Revision 1, Supplement 1 response to **RAI 03.07.02-24 (**ML102070067)
- the RSW piping tunnels, in the Supplement 6 response to **RAI 03.08.04-30** (ML11259A056)

A discussion of the design lateral soil pressures provided in the markups for each building is in Part B, "Evaluation of Design Issues Related to Specific Structures," later in this subsection of the SER.

The FSAR was subsequently revised to incorporate the proposed changes in Sections 3H.6, 3H.6.7, and 3H.7. Therefore, portions of **RAI 03.08.04-17** that are related to Category I structures are closed.

f) Including Extreme Snow Load as an Extreme Environmental Load in Subsection 3H.6.4.3.3

In FSAR Subsection 3H.6.4.3.3, "Extreme Environmental Load," the applicant did not include the snow load resulting from the extreme winter precipitation event. **RAI 03.08.04-10** and follow up **RAI 03.08.04-21** asked the applicant (1) to include in FSAR Subsection 3H.6.4.3.3 the snow load that resulted from the extreme winter precipitation event; (2) to use the guidance in DC/COL-ISG-7; and (3) to elaborate how the extreme snow load used in the load combination for the roof design was determined. In the August 20, 2009, response to **RAI 03.08.04-10** (ML092370556), the applicant states that the extreme snow load information was provided in Subsection 3H.6.4.3.3.5 as an attachment to the response to **RAI 03.07.01-13** (ML092360772). Subsequently, in the February 4, 2010, response to **RAI 03.08.04-21** (ML100480204), the applicant states that per COL FSAR Subsection 2.3S.1.3.4, the ground snow load for both a normal winter precipitation event and an extreme frozen winter precipitation is 0.263 kPa

(5.5 psf). The applicant also states that DC/COL-ISG-7 provides guidance for converting the ground snow load to the roof snow load using the methodology in ASCE 7–05, which utilizes exposure factor Ce, thermal factor Ct, and Importance Factor I as multipliers in Equation 7-1 of Section 7.3. Using the DC/COL-ISG-7 recommended values for these three coefficients (Ce = 1.1; Ct = 1.0; I = 1.2) and the limitation for a minimum value provided in Section 7.3 of ASCE 7-05, the roof snow load was determined to be 0.316 kPa (6.6 psf). Per DC/COL-ISG-7, the extreme winter precipitation shall be the larger of the following two cases:

Case 1: Normal winter precipitation + Extreme frozen winter precipitation
Case 2: Normal winter precipitation + Extreme liquid winter precipitation

Per COL FSAR Subsection 2.3S.1.3.4, the extreme liquid winter precipitation is 86.36 cm (34 in.). However, since the roofs of site-specific Category I structures are designed without parapets (see COL FSAR Subsection 3H.6.4.2.5), the extreme winter precipitation cannot exceed the Case 1 loading of 0.632 kPa (13.2 psf [6.6 psf + 6.6 psf]) assuming that both the roof drains and scuppers are clogged. The applicant's markups of COL FSAR Subsections 3H.6.4.2.4, 3H.6.4.3.1.3, and 3H.6.4.3.3.5 and Table 2.0-2 include the above information in the FSAR. The staff found that the applicant's response has adequately addressed the issue because the response follows the guidance in DC/COL-ISG-7 for determining the normal and extreme winter precipitation loads and includes them in appropriate sections of the FSAR. The FSAR was subsequently revised to incorporate the proposed changes. Therefore, the issue is resolved; and **RAI 03.08.04-10** and **RAI 03.08.04-21** are closed.

A.5 Design and Analysis Procedures

A.5.1 Concrete Design for Out-of-Plane Shear

During the October 2010 audit (ML110110104), the applicant presented the procedures for design of concrete sections of the UHS/RSW pump house structural members resulting from the code-required load combinations. The internal forces (i.e., shear, moment, axial force, and torsion, etc.) that are used to determine the required strength of the structural members (i.e., walls, slabs, beam, and columns, etc.) of the UHS/RSW pump house building are generated by the applicant with the help of the SAP2000 models simulating the building's static and dynamic behavior. These element forces are subsequently processed with a number of programs developed in-house for designing concrete sections. The applicant stated that the concrete slabs and walls were designed for out-of-plane shear by averaging the element shear forces across cut lines that extended along the entire width of the walls and slabs. The staff noted that the averaging of the out-of-plane shear along the entire cut line of a slab or wall could lead to a non-conservative estimate of shear stress in the slabs. ACI 349–97, Section 13.3.1 states that a slab system may be designed by any procedure satisfying the conditions of equilibrium and geometric compatibility, if it is shown that the design strength at every section is at least equal to the required strength. **RAI 03.08.04-34** asked the applicant to demonstrate that the use of average shear forces across the entire width of the slab—instead of the shear force demand at every section obtained from analysis—is acceptable by any or more of the following:

- Obtain clarification from the ACI regarding the validity of using Section 11.12 of ACI 349–97 for the situations that used the provisions of the code.

- Provide examples of any precedence with a similar methodology that the staff has accepted,

- Provide a detailed justification using industry-accepted standards, technical references, and experimental results for redistributing the shear forces obtained from the FEA.

The staff also asked the applicant to update the FSAR as necessary. In the original and Supplement 1 and 2 responses to **RAI 03.08.04-34** dated April 5, 2011; September 12, 2011; and November 14, 2011 (ML110980401, ML11259A056, and ML11322A106, respectively); the applicant states that the FEA was used to design the following buildings:

- UHS/ RSW pump house
- DGFOT
- DGFOSV
- RWB (not a Category I structure)

Because the ACI 349–97 code does not provide clear guidance for out-of-plane shear design when loads are obtained from the FEA, the design of the above structures was revised so that it is conservatively based on the out-of-plane shear obtained for each element from the FEA, without averaging the shear over several elements. According to the Supplement 1 response to **RAI 03.08.04-34**, the only exceptions are the four locations corresponding to the junction of the basemat and the buttress of the UHS basin, where the element shear was averaged over a length corresponding to twice the thickness of the slab. These exceptions are deemed adequate because the finite element mesh is very refined at those locations. They simulate unrealistically high stress concentrations that are smoothed by the averaging process. The response also contains a markup of Appendix 3H, with the modified tables and figures showing the reinforcement for the different structural elements of the buildings mentioned above. The staff concluded that the response was adequate and satisfactory because the shear force demand at every section is now based on the corresponding element forces from the finite element analysis, instead of being based on the average of several elements of a cut line. Averaging of the element shear forces at the junction of the basemat and the buttress of the UHS basin over a length corresponding to twice the thickness of the slab is considered acceptable since it corresponds to an accepted practice of 45-degree dispersion of load concentration across the slab thickness. The section forces of the RSW tunnel were not obtained from the finite element analysis, and they are not subject to the clarification from this issue. FSAR Appendix 3H was subsequently updated to incorporate the proposed changes provided with the response. This issue is therefore resolved, and **RAI 03.08.04-34** is closed.

A.5.2 Stability Evaluations

The foundations of the standard plant structures, site-specific seismic Category I structures, and non-Category I structures with the potential for interacting with Category I structures must be designed so that the buildings are stable and have adequate factors of safety. In FSAR Section 3H.6, “Site-Specific Seismic Category I Structures,” the applicant does not include sufficient information about the design and stability evaluation of foundations in order for the staff to determine whether the foundations for these structures meet the SRP Section 3.8.5 acceptance criteria. **RAI 03.08.05-1** asked the applicant to include in FSAR Section 3H.6 information about the method used to evaluate the stability of the foundations of site-specific seismic Category I structures, including consideration of buoyant forces, coefficients of friction used to evaluate the foundation sliding and its basis, consideration of active and passive pressures on foundation walls to evaluate the stability, and the results of the stability evaluation.

In the September 22, 2009, response to **RAI 03.08.05-1** (ML092660655), the applicant states that stability evaluations are performed in accordance with the loads and load combinations of SRP Section 3.8.5 acceptance criteria; and the evaluations meet the minimum required factors of safety specified in the SRP. The applicant also refers to the response to **RAI 03.07.02-10** (ML092610377) for detailed information regarding how the sliding and overturning evaluations were performed. The applicant states that the factor of safety against floatation was determined considering the maximum buoyant force under flooded conditions. In the response to **RAI 03.07.02-10**, the applicant states that the UHS/RSW pump house and the RSW piping tunnel are the only site-specific seismic Category I structures. The applicant adds that the RSW piping tunnel (with the exception of the access shaft) is a buried structure. To check the sliding and overturning of the UHS/RSW pump house, the applicant used the following criteria:

1. Loads and load combinations in COL FSAR Subsection 3H.6.4.5, with the loads defined in Subsection 3H.6.4.3.4.1. The applicant states that these loads and load combinations are in accordance with SRP Section 3.8.5 acceptance criteria.

2. As described in COL FSAR Subsection 3H.6.5.1.1.2, seismic loads from three seismic excitations (i.e., x, y horizontal, and z vertical) are applied considering the 100-40-40 combination rule, as shown below:

   ±100% x -excitation ± 40% y-excitation + 40% z-excitation
   ±40% x -excitation ± 100% y-excitation + 40% z-excitation

   (Note: Positive z-excitation is upward. Also, ±40% x-excitation ±40% y-excitation ± 100% z-excitation is not critical.)

3. Only 90 percent of the dead load is considered in determining the resisting forces and moments.

4. A friction coefficient of 0.30 is assumed. Additional information regarding the waterproofing membrane is in the response to **RAI 03.08.04-5** dated September 15, 2009 (ML092610377).

5. Passive pressure is not utilized; only at-rest soil pressure is considered for calculating the resisting sliding forces and overturning moments. This statement was subsequently revised. Table 3H.6-5 shows the factors of safety and the corresponding fractions of passive pressure required to resist sliding and overturning.

6. Calculated factors of safety against overturning, sliding, and flotation will be provided later in a supplemental response to **RAI 03.07.01-13**.

7. Figure 1 in the response to **RAI 03.07.02-10** shows how stability safety factors are calculated.

The staff evaluated Items 2 and 5 of the above response in Subsection 3.7.2.4.14 of this SER.

Regarding Item 1 of the above response, load combinations used for the stability evaluation are in accordance with the criteria in SRP Section 3.8.5, which are acceptable.

Regarding Item 3 of the response, the use of only 90 percent of the dead load to determine the resisting forces and moments is an accepted engineering practice for a lower bound estimate of the dead load to account for uncertainties and is considered acceptable.

Regarding Item 4 of the response, the applicant states that a friction coefficient of 0.3 was assumed for the stability evaluation of the UHS/RSW pump house. However, the applicant did not provide any basis for the value of the coefficient of friction that was used. See an additional discussion below under the topic, "Coefficient of Friction and Cohesion."

Regarding Item 6 above, the applicant subsequently provides the factors of safety for the UHS basin and the RSW pump house in FSAR Table 3H.6-5, which the applicant submitted with the Supplement 2 response to **RAI 03.07.01-13** dated November 19, 2009 (ML093270047). These factors meet the minimum required factors of safety per SRP Section 3.8.5, and they are therefore acceptable.

Regarding Item 7 in the response, Item 4 of follow-up **RAI 03.07.02-18** asked the applicant to clarify the nature of the driving static soil pressures in Figure 1 of the response and how the total at-rest soil pressure is calculated (including the algebraic expression). The staff also asked the applicant to include this information in the COL FSAR. In addition, the staff asked the applicant to include in the FSAR the Figure 1 submitted with the response. In the February 10, 2010, response to **RAI 03.07.02-18** (ML100550613), the applicant refers to Figures 3H.6-45 through 3H.6-50 submitted with the Supplement 2 response to **RAI 03.07.01-13** (ML100050225). These figures depict the driving and resisting lateral earth pressures on the UHS basin and the RSW pump house. The applicant also states that the lateral earth pressures, including the algebraic equations, are in FSAR Subsection 2.5S.4.10.5. The staff reviewed the referenced FSAR section and found that the active seismic lateral pressure was used as the driving lateral earth pressure, and the static earth pressure at rest was used as the resisting pressure using the formulas reviewed under this section. See an additional discussion below under the topic, "Calculation of SSE Loads." The applicant also includes Figure 1 of the response in the FSAR as Figure 3H.6-137. The methodology shown in Figure 3H.6-137 for calculating safety factors against sliding and overturning is in accordance with standard engineering practices and is therefore acceptable.

Regarding the stability evaluation of non-Category I structures, the applicant's response to **RAI 03.07.02-1** (ML092610377) indicates that the design of non-Category I structures with the potential to interact with Category I structures has not yet progressed to a point where sliding and overturning potential as a result of the SSE can be evaluated. However, in order for the staff to review the seismic design of these non-Category I structures, **RAI 03.07.02-13** asked the applicant to provide in the FSAR factors of safety against sliding and overturning. The staff asked the applicant to include the basis of the coefficient of friction used in the analysis during the SSE of the TB, RWB, SB, CBA, and the plant stack. Furthermore, during the NRC audit in May 2011 (ML12346A389), the staff reviewed the applicant's stability evaluation for the various non-Category I structures. The staff asked the applicant to provide several clarifications, including the following items:

- Clearly describe in the FSAR how the applicant determined the seismic demand for non-seismic II/I structures in the stability evaluation.

- Revise the CBA stability evaluation using ASCE 7–05 instead of ASCE 7–88 and check for two cases: (1) with the live load for both the stabilizing force and the driving force, and (2) with no live load for either the stabilizing force or the driving force.

In the response to **RAI 03.07.02-13** dated February 10, 2010 (ML100550613), the applicant provides a markup of FSAR Subsection 3.7.2.8 to identify all non-Category I structures that may potentially interact with seismic Category I structures. The applicant also describes the methodology used for the calculation of safety factors against the sliding and overturning for these structures. Subsequently, in the Supplement 1, Supplement 2, Revision 1 to Supplement 3, and Supplement 4 responses to **RAI 03.07.02-13** dated April 29, 2010; April 25, 2011; and November 28, 2011, (ML101250162, ML11119A077, ML11335A232, and ML11335A232, respectively); the applicant provides results of the stability evaluation for non-Category I structures with additional clarifications and FSAR markups. FSAR Figure 3H.3-52 was provided for the stability evaluation of non-seismic Category I structures. In these responses, the applicant uses the full dead load for calculating the restoring forces instead of the 90 percent of dead load used for Category I structures. The staff found this approach acceptable because it meets the guidance in SRP Section 3.8.5. Also, the methodology described in Figure 3H.3-52 for calculating the safety factors against the sliding and overturning is in accordance with standard engineering practices and is therefore acceptable.

In the Revision 1 to Supplement 3 response to this RAI, the applicant also provides markups of FSAR Subsections 3.7.2.8 and 3.7.3.16. These subsections describe the seismic inputs; the calculation of the seismic demands; and the method used to combine the three orthogonal seismic demands for the stability evaluation of the TB, RWB, SB, and the CBA. The seismic input motions used for the stability evaluation of the non-Category I structures are evaluated in Subsection 3.7.2.4.8 under the topic, "Development of Amplified Seismic Input Motions for II/I Stability Evaluations," of this SER. The seismic demand was calculated using the response spectrum method of a fixed-base stick model with corrections for the base excitation of the TB, RWB, and the SB. The manual calculation of an idealized single degree of freedom lumped mass model was used for the CBA. The staff found these clarifications to be reasonable engineering methods for such evaluations and therefore acceptable. The three orthogonal seismic demands were combined using the 100-40-40 percent combination rule per RG 1.192, which is considered acceptable. The coefficients of friction used for the stability evaluation were based on shear resistance of the supporting foundation soil, which is considered acceptable. In the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232) dated November 28, 2011, the applicant explains that the dynamic loads from the soil included seismic loads from soil and hydrodynamic pressures from the groundwater, which are computed in accordance with Subsection 2.5S.4.10.5. The staff reviewed the referenced section and found that active lateral seismic pressure was used as the driving lateral earth pressure. See an additional discussion below under the topic, "Calculation of SSE Loads." The staff also requested the applicant to reassess the seismic demand for the stability evaluation of any applicable seismic Category I and II/I structures, in the context of the DNFSB issue and to confirm the acceptability of the factors of safety against the stability during the SSE. This issue is evaluated below in Subsection 3.8.4.4.4, "DNFSB Issue: Resolution of Issues with Subtraction Method of Analysis."

The various elements of the stability evaluation that are applicable to all structures include:

- Method of stability evaluation
- Calculation of SSE loads (seismic demand)
- Coefficient of friction and cohesion (static and dynamic)
- Stability factors of safety for site-specific seismic Category I structures

A description of how each issue is addressed in the STP Units 3 and 4 FSAR is given below:

a. *Method of Stability Evaluation*

The stability evaluation for each structure is performed using the loads and load combinations specific to the structure, as described in FSAR Appendix 3H. These loads and load combinations are in accordance with SRP Sections 3.8.4 and 3.8.5. FSAR Figures 3H.3-52 and 3H.6-137 provide pictorial representations of the method used for the stability evaluation of non-seismic Category I structures and seismic Category I structures, respectively. These figures show the various forces considered to be acting upon the structure for the stability evaluation. These figures also show the equations for calculating the factor of safety against the sliding and overturning. The factor of safety against sliding is calculated as the ratio of the total resisting force, which is the sum of the total resisting lateral soil pressure (P_{at_rest} or $P_{passive}$) and the friction force (F) to the total driving force, which is the sum of the static and dynamic soil pressure including the seismic loads from soil and hydrodynamic pressures from the groundwater (E_s) and the self-weight excitation in the horizontal direction (E'). The factor of safety against overturning is calculated as the ratio of the restoring moment resulting from the passive earth pressure and the dead weight of the structure taking into account buoyant force to the total seismic overturning moment taking into account static and dynamic earth pressure, including hydrodynamic pressure from the groundwater and the seismic inertia force of the structure, including the vertical seismic excitation. The applicant also states that the safety factor against flotation is determined considering the maximum buoyant force under flooded conditions (i.e., the design flood level at an elevation of 12.2 m [40 ft]). For the stability evaluation of seismic Category I structures, 90 percent of the dead load is considered in the determination of the resisting forces and moments. For non-seismic Category II/I structures, the full dead load is considered in the calculation of resisting forces and moments.

FSAR Table 3H.9-1 shows the extreme environmental design parameters used for the seismic analysis; structural design; stability evaluation; and the seismic Category II/I design for the UHS/RSW pump house, RSW piping tunnel, DGFOSV, DGFOT, RWB, CBA, TB, and SB.

The staff found that the method the applicant used for the stability evaluation is a conservative approach for considering forces acting on a structure and is based on standard engineering practices for such evaluations. This method is therefore acceptable.

b. *Calculation of SSE Loads*

Figures 3H.3-52 and 3H.6-137 represent a free-body diagram of the structure, which shows the participating forces in the stability analysis. As described in the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232), the seismic effects are represented by E' and Es, whereby

- E' represents the inertia of the structure and it is either determined from an equivalent static method or a RSA

- Es represents the static and dynamic loads from the soil that includes seismic loads from soil and hydrodynamic pressure from the groundwater. These loads are computed in accordance with COL FSAR Subsection 2.5S.4.10.5.

The inertial force E' is obtained as mass times absolute acceleration and can be determined directly from the 3-D SSI analysis of the structure, or by using the accelerations obtained from the 3-D SSI analysis as input to an equivalent static or RSA of the structure. Where applicable, the input ground motion to the 3-D SSI model corresponds to an amplified ground motion, which represents the modified soil response at the location of the building resulting from the presence of an adjacent heavier structure (considering one heavy structure at a time). For a detailed description of the approach used to determine amplified motions, see discussions in Subsection 3.7.2.4.4 of this SER.

The dynamic component of Es represents the incremental lateral soil and hydrodynamic pressures resulting from the seismic event. Es can be for active and at-rest soil conditions as follows—depending on whether the wall moves or remains at rest.

- For an active condition, the dynamic increment is obtained by applying the Mononobe-Okabe method described in FSAR Subsection 2.5S.4.10.5.2.

- For an at-rest condition, the dynamic at-rest increment is obtained either from the 3-D SSI analyses; or alternatively from following the ASCE 4 provisions (used for shallow embedment); or it is based on Ostadan's method (used for deep embedment) as described in Subsection 2.5S.4.10.5.2. Where applicable, the at-rest increment is also obtained from one or more SSSI analyses that reflect the effect of nearby heavy structures on the structure under consideration.

- For hydrodynamic pressures, the hydrodynamic increment of the ground water is obtained as described in FSAR Subsection 2.5S.4.10.5.2.

Both seismic load components E' and Es are calculated for the ground input motions specified in Table 3H.9-1. The horizontal component of the inertial and incremental soil pressure induced by the vertical earthquake component is considered and combined with the 100-40-40 rule. The sliding forces are counteracted by friction and lateral soil pressure (at-rest or passive). The overturning of the structure is counteracted by the stabilizing moments from the dead weight of the structure and the lateral soil pressure (at-rest or passive). The amount of passive resistance required to achieve the minimum safety factors is determined and deemed adequate if the required coefficient Kp is smaller than or equal to 3. Since the driving seismic soil pressures used in stability design calculations were based on the Mononobe-Okabe method, which assumes active condition, the staff asked the applicant to demonstrate the total seismic demands used for stability evaluations. In the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232) Item A, the applicant computes the total seismic sliding force and the total seismic overturning moment by integrating the reactions from the SSI and/or the SSSI analysis around the boundary of the structure. The applicant then compares them against the sliding force and overturning moment design values used for the stability evaluation. These comparisons show that the driving seismic sliding and overturning moments used in the stability evaluations exceed those computed from the SSI and/or SSSI analyses. The staff considered this approach acceptable for determining the total seismic demand on a structure for a stability evaluation. The staff, however, questioned whether integrating the reactions from the SSI or SSSI along the embedded boundary of the structure will correctly estimate the resulting lateral seismic forces, especially when a heavy adjacent structure is present. This is of particular concern for the DGFOT located adjacent to the RB. For a discussion of this issue, see Item d below for the scope and Subsection 3.8.5.4 of this SER for evaluations that were performed for each structure.

c. *Coefficient of Friction and Cohesion*

The sliding resistance of the foundation is a function of the coefficient of friction and/or the cohesion between the foundation and the soil beneath it. The values for static and dynamic conditions are obtained from the geotechnical test reports by MACTEC and are shown in FSAR

Tables 3H.6-5, -12, -14, -16, and 3H.7-2. The coefficient of friction and the cohesion values for the gravel and soil interface are based on the properties of the soil under the buildings and mobilize the full soil shear strengths. Sixty-seven percent of the sand friction coefficient and 80 percent of the cohesive soil strength are used to account for the dynamic effects and provide a sufficient margin on the lateral earth pressure that is required to meet the safety factor against sliding. The staff reviewed the basis for the friction coefficient and cohesion values used in the sliding analysis of foundations in the applicant's Revision 1 response and Supplement 1 response to **RAI 03.08.04-28** dated March 7, 2011, and April 25, 2011 (ML110730067 and ML11119A077, respectively), and the qualification program for the waterproofing material to achieve calculated safety factors against sliding. The staff found these coefficients technically acceptable. Also see the discussion above in Subsection 3.8.4.4.2 under Subpart A.3, "Minimum Coefficient of Friction"; and Item 1 under COL License Information Item 3.23 in Subsection 3.8.5.4 of this SER for the discussion on "Foundation Water Proofing."

d. Stability Factors of Safety for Site-Specific Seismic Category I Structures

Stability evaluations were conducted for the following site-specific seismic Category I structures:

- UHS basin and RSW pump house
- RSW tunnel
- DGFOSVs
- DGFOT

Detailed descriptions of the stability analyses and the results for each of the above structures are in Subpart A.4 and Subparts B.1 through B.4 under Supplemental Information in Subsection 3.8.5.4, of this SER.

A.5.3 Seismic Response Analysis

RAI 03.08.04-30 Item 1 asked the applicant to provide details of how the results of the seismic response analysis from the dynamic SSI analysis are transferred to the static FEA model, including how the effects of accidental torsion are included in the analysis. The applicant's Supplement 1 response to **RAI 03.08.04-30** (ML110770440) describes the procedure applied in the design of the UHS/RSW pump house. The applicant states that the nodal absolute accelerations (i.e., zero period acceleration (ZPA)) from the SSI analysis are mapped into nine groups and a total of 208 panels of the SAP2000 structural model. The accelerations within a panel are averaged to a constant value within that panel. These panel accelerations, in conjunction with the building, equipment and live and water masses, are used to obtain the seismic inertial forces applied to the structure. Seismic forces are applied in one excitation direction at a time, and the combined effects subsequently use the square-root of the sum-of-the squares (SRSS) method at the element level. In response to what effect a mesh refinement of the SSI model would have upon the accelerations, the applicant performs a comparison of the results with the "coarse" mesh and provides tables with adjustment factors that correct the design accelerations, if any occurred, to the higher values obtained with the finer model. For a complete discussion of the mesh refinement effects on the dynamic response, refer to SER Subsection 3.7.2.4.3 under the topic, "Resolution of Items Observed in Various Sensitivity Studies." Accidental torsional effects per SRP Section 3.7.2 are simulated by applying to the SAP2000 model additional torsional moments that are equivalent to an increment of the eccentricity (between the center of rigidity and the center of mass) of 5 percent of the maximum building dimension in each direction and at each floor elevation. The resultant torsional moments are decomposed into in-plane shear element forces applied to the perimeter walls at the following elevations: the top and base of the cooling tower, the top of the basin, the top of the water level, the pump house roof, and the operating floor. All of the torsional moments acting in the clockwise direction due to both x and y directional excitations were applied simultaneously. Similarly, all of the counterclockwise torsional moments due to x and y directional excitations were applied at the same time. This method assumes that ground excitations in both directions are exciting the structure simultaneously, and they therefore represent a conservative approach. Since the accidental torsional moments are directional in nature, the resulting forces were always added to the other forces in the seismic load combination. The staff found this response acceptable because it follows a logical two-step procedure of evaluating the dynamic behavior of the structure, including the SSI effects: the first step involves the dynamic analysis of the SSI system; the second step applies the equivalent static method of analysis to the FEA model used for the final and detailed design. Therefore, this issue is resolved.

A.5.4   Bearing Pressures

In **RAI 03.08.04-30 Item 7,** the staff asked the applicant to include the maximum static and dynamic bearing pressures under the foundations and to compare these values with the maximum allowable static and dynamic bearing pressures. In the Supplement 1 response to **RAI 03.08.04-30** regarding soil bearing pressures, the applicant states that the static and dynamic ultimate bearing pressures are given in Tables 2.5S.4-41B and 2.5S.4-41C, respectively, and that actual soil pressures can be obtained by dividing the ultimate values by the calculated safety factors given in those same tables. During the October 2010 audit (ML110110104), the applicant presented the procedures used to determine the dynamic soil pressures beneath the UHS/RSW pump house foundation mat, which resulted from the SSE loadings. Using this method, the applicant calculates the equivalent eccentricity of a vertical load considering the moments in two perpendicular directions acting on the foundation. An equivalent foundation contact area is then calculated that is symmetric about the eccentric location of the vertical load. The dynamic bearing pressure below the foundation is calculated as a uniform pressure on the reduced foundation contact area. The staff noted that the applicant’s methodology for calculating soil bearing pressures based on an equivalent foundation and uniformly distributed soil pressures under the foundations was not consistent with the analysis and design of the structures, including the basemat, and such a procedure may significantly underestimate the expected foundation toe pressures for loading combinations with large overturning moments. Therefore, in **RAI 03.08.04-35** the staff asked the applicant to provide additional information describing how the procedure used for verifying soil bearing pressures reconciles with the analysis and design of the structures and based on simulating the soil with elastic soil spring elements in SAP2000. In the June 28, 2011, Revision 1 response to this RAI (ML11181A002), the applicant states that the methodology used for the foundation soil bearing capacity evaluation and the determination of corresponding safety factors is in accordance with that described in COL FSAR Subsection 2.5S.4.10.3. The methodology for the evaluation of eccentrically loaded foundations was developed by Professor J. Brinch Hansen and Professor G.G. Meyerhof, (Hansen, J.B., 1970, A Revised and Extended Formula for Bearing Capacity. Bulletin of the Danish Geotechnical Institute, No. 28, pp. 5-11) and is considered well-established in geotechnical manuals and textbooks. In this methodology, the coupled moment and the vertical load acting simultaneously at the center of the foundation are transformed into an equivalent foundation loading system, with the same vertical load solely acting at a point offset from the center of the actual foundation. The factor of safety of the foundation is expressed as the ratio of the ultimate load to the applied vertical load (see COL FSAR Section 2.5s.4, Equation 2.5S.4-22). The factors of safety documented in COL FSAR Table 2.5S.4-41C are based on design load combinations acting on the foundations, including those where foundation uplift may be present. In the referenced paper above, Hansen states that the actual bearing capacity of an eccentrically loaded foundation will be very nearly equal to the bearing capacity of the centrally loaded effective foundation area. This concept of using the effective foundation area with a centrally applied vertical load to represent the actual foundation area subject to the same vertical load plus moments is widely recommended in geotechnical manuals and textbooks. It implicitly accounts for the non-uniform pressure distribution under the actual foundation, including the heel and toe pressures. Since the foundation soil is represented by soil springs in the FEAs, the pressure distribution at the bottom of the basemat under the eccentric loading will vary, and thus the design of the structure—including its basemat—will account for higher heel and toe pressures. The staff's evaluation noted that the above method of computation was performed according to the recommendations in industry recognized text books and publications such as Joseph E. Bowles, J. B. Hansen, and G. G. Meyerhof, and it provides a consistent method of comparing the actual bearing pressure and the calculated ultimate bearing capacity under a foundation subjected to vertical load and moment. The design of the basemat is comprised of finite elements and the use of foundation soil springs that take into account the effect of a variation of pressure distribution at the foundation-soil interface under vertical load and moment. The applicant's response justifies the design procedures used to determine the safety factors against soil failure as well as the structural design based on soil springs, thereby satisfying the staff's request for clarification. The staff agreed with the explanation and **RAI 03.08.04-35** is therefore closed.

B. Evaluation of Design Issues Related to Specific Structures

In **RAI 03.08.04-13, RAI 03.08.05-1**, and follow-up **RAI 03.08.04-23** and **RAI 03.08.04-30**, the staff asked the applicant to include the structural analysis and design information, as well as the stability evaluations for all site-specific seismic Category I structures in the FSAR, using the guidance in SRP Section 3.8.4 and other applicable SRP sections and guidance documents. The following discussions apply to the design of individual site-specific structures listed below.

B.1 Design Information for the UHS/RSW Pump House and RSW Piping Tunnels

The detailed analysis and design information for the UHS/RSW pump house and the RSW piping tunnels is documented in FSAR Section 3H.6, "Site-Specific Seismic Category I Structures." This section includes a description of the structures; analysis; design methods; finite element models; materials; SSI analysis; seismic wave propagation effects; stability evaluation; applicable codes and loadings; lateral earth pressure diagrams; description of the wall, slab, and foundation design; soil springs; uplift analysis; tables and figures reporting section forces; governing load combinations; and resulting concrete bending and shear reinforcement and layouts.

The following seven subsections address the specific review criteria in the SRP:

B.1.1 Description of the Structures

The UHS/RSW pump house structure consists of the main basin building with the cooling towers located at roof level and the attached pump house enclosing the RSW pump systems. The building is constructed on a mat foundation and is framed with shear walls. The roof and operating floor are designed as a composite concrete slab with steel beams. In FSAR Subsections 3H.6.3.1 through 3H.6.3.4, the applicant provides a general description of the UHS/RSW pump house and the RSW piping tunnels that pertains to each proposed unit at the STP site. Figure 3H.6-221 shows a partial site plan with the locations of the UHS/RSW pump house and the RSW piping tunnels relative to other structures. The staff noted that the FSAR did not include plan and section views for the UHS/RSW pump house or the RSW piping tunnels with basic structural information that meets the guidance in SRP Section 3.8.4. The applicant was therefore asked to provide this additional information during several audit discussions. In the Supplement 5 response to Item 10 and the Supplement 6 response to Item 1 of **RAI 03.07.02-13** dated April 10, 2012, and May 29, 2012, (ML12103A369 and ML12153A101, respectively); the applicant provides markups of FSAR Subsections 3H.6.3.1 through 3H.6.3.4 and Figures 3H.6-258 through Figure 3H.6-262, which in turn provide plan and section views of the UHS/RSW pump house and the RSW piping tunnels. These figures show the basic information and dimensions of the buildings and are considered adequate per the guidance in SRP Section 3.8.4. Therefore, this item is considered resolved. The applicant incorporated the changes into the FSAR Revision 8.

B.1.2 Applicable Codes, Standards and Specifications

In FSAR Subsection 3H.6.4.1 and in FSAR Section 1.8, the applicant provides information regarding the applicable design codes (i.e., ACI 349–97 and RG 1.142) for concrete structures; AISC N690–1994, including Supplement 2 (2004), for steel structures; ACI 350.1-10, “Specification for Tightness Testing of Environmental Engineering Concrete Containment Structures”; and several other codes considered in the design. The staff finds the referenced codes and standards to be appropriate for design and construction of the UHS/RSW pump house and the RSW piping tunnels. During review of the UHS/RSW pump house design calculations performed in the May 2011 audit, the staff noted that an older version of the AISC N690 (N 690–84) code was used for the design of the steel members. Because the used version was older than the 1994 version endorsed by the SRP and specified in FSAR Table 1.8-21a for site-specific structures, the staff asked the applicant to revise the UHS/RSW pump house calculation and to use version AISC N690–94, including Supplement 2. In the Supplement 5 response to **RAI 03.07.02-13** Item 12 dated April 10, 2012 (ML12103A369)**,** the applicant states that the calculations are revised to meet the AISC N690–94 code requirements, including Supplement 2. The staff found this response acceptable, because the calculations are revised to meet the version of the code endorsed by SRP 3.8.4. Therefore, this issue is considered resolved.

B.1.3 Loads and Load Combinations

As described in more detail under Subpart A.4 (Item a) in Subsection 3.8.4.4.2 above, **RAI 03.08.04-12** and follow-up **RAI 03.08.04-22** and **RAI 03.08.04-29** asked the applicant to clearly describe the various loads and load combinations, especially in regard to the design of the UHS/RSW pump house and RSW piping tunnels. FSAR Subsection 3H.6.4.3 provides the design loads and load combinations used for the UHS basin, the RSW pump house, and the RSW piping tunnels. These loads include normal loads, severe environmental loads, and extreme environmental loads. This section also includes load combinations with the acceptance criteria that are used to design steel and concrete structures. Subsection 3H.6.4.2 provides site design parameters that are considered when establishing the design loads. The staff found the loads and load combinations described in Subsection 3H.6.4.3 acceptable because they were determined to be adequate based on considerations of the applicable site design parameters; their adequate descriptions of all of the components of the loads with their numerical values, as needed; and verification that the load combinations that were used comply with the appropriate code requirements and the guidance in SRP Section 3.8.4.

B.1.4 Design and Analysis Procedures

In FSAR Subsections 3H.6.6.1 (UHS/RSW pump house) and 3H.6.6.2.2 (RSW piping tunnels) the applicant provides information regarding the structural design and analysis of the UHS/RSW pump house and the RSW piping tunnels. During the audit in May 2011 (ML12346A389), the staff reviewed the structural design calculations for the UHS/RSW pump house, including the analysis and design assumptions; applied loadings and load combinations; the thermal analysis; the hydrodynamic analysis; the finite element models and results of the analyses; the beam, column, slab, and wall designs; the lateral earth pressure parameters and loads; the design methodologies; and the equivalent static seismic procedures. The following is a summary of the staff's observations.

The analysis is performed with the help of SAP2000 finite element models using spring, frame, and thick-shell finite elements. A total of eight SAP models are used as follows: four models with uniform soil springs and four models for non-uniform soil springs. The four models account for static loads, dynamic loads, static without vertical loads, and seismic loads without vertical loads. Different soil springs are used under the UHS and the RSW pump house. All results of the analysis are enveloped, and seismic forces are typically combined with the SRSS rule for member design. Inertial seismic forces are applied to the SAP model as equivalent static forces. The maximum accelerations from the SSI analyses are grouped into nine vertical groups and 208 panels representing different locations in the structure. Accelerations are averaged within each panel section. Full and empty basin cases are considered, and the dynamic soil spring values are used in the SAP models. Applied inertial forces are computed as the product of the acceleration times the corresponding mass times an adjustment factor (used to compensate for different mesh sizes in SAP and SSI models). The x-, y-, and z-loads are subsequently combined by the SRSS superposition rule. The x-, y- and z- accelerations vary between 0.12g (slab) and 0.60g (tower). It was noted that the maximum element accelerations could be as high as three times the calculated average acceleration for a panel. In order to verify the use of the average panel accelerations instead of the maximum element accelerations is still a conservative approach: section forces (element axial and shear forces and bending moments) taken from the SASSI model were compared with the corresponding SAP values along horizontal and vertical cut lines. The force comparisons show that the SAP results are consistently higher than the corresponding SASSI values. Specifically, the comparison for one of the interior N-S walls of the cooling tower (Section Cut #55) showed ratios greater than 1.60 for all six internal forces of the shell element. Based on this example, the staff considered the overall approach followed for the seismic design to be conservative compared with the SSI results. The design of the concrete and steel elements is performed for the loading combinations in Subsection 3H.6.4.3. Torsional effects from the wind on interior columns, buttresses, and walls are considered. Hurricane winds (296 km/h [184mph]) are used for all LCs including wind loads (W). No uplift was identified for any LC. The software program TEMCO (see also separate discussion under Subpart A.4 (Item d) "Combination of Forces and Moments" in Subsection 3.8.4.4.2 above) is used to design sections subjected to thermal gradients and non-thermal axial forces and moments. Thermal gradients are applied to all concrete components such as walls, slabs, roofs, foundation mats, beams, and columns. Simulations of hydrodynamic effects of the basin water and lateral soil pressures applied to the buried walls are discussed in more detail below.

The staff found the design and analysis procedure for the UHS/RSW pump house structures to be acceptable and in accordance with the standard engineering practices for equivalent static methods of analysis and the guidance in SRP Section 3.8.4 and the applicable codes and standards. The staff, however, identified the following two issues that are evaluated separately in this SER section: (a) as the original SASSI analysis did not include the adhered water masses on the submerged part of the columns, new response accelerations had to be determined from a modified SSI model to correctly design the columns (see detailed discussion below); and (b) the calculation referenced standard ANSI/AISC N690-84 for the steel design instead of ANS/AISC N 690–94, which is adopted for STP Units 3 and 4 site-specific structures as stated in FSAR Section 1.8 (see resolution of this issue in Subpart B.1.2 above).

The procedures for the design and analysis of the RSW tunnels are described in Subsection 3H.6.6.2.2. Manual calculations are used for the analysis and design of the RSW tunnels. The individual components of the tunnel (roof slab, intermediate slab, basemat, and walls) have out-of-plane frequencies in excess of 33 Hz; and their out-of-plane seismic loads are determined using a conservative acceleration of 0.21g that exceeds the ZPA of the response spectra in Figures 3H.6-138 and 3H.6-139. Tunnel walls and slabs are designed as one-way slabs for the loads and load combinations in Subsection 3H.6.4.3. Lateral soil pressures used to design the RSW piping tunnels are discussed separately below. The tunnel is also designed for the effects of seismic wave propagation, which is discussed separately below. The staff found the simplified analysis and design method used for the tunnel to be conservative and acceptable.

Furthermore, the staff's review specifically addressed the following six issues:

a. *Modeling of Hydrodynamic Loads for the UHS Basin*

In FSAR Subsection 3H.6.6.2.1, the applicant describes the loadings used for the design of the UHS basin, the UHS CT enclosure, and the RSW pump house. **RAI 03.08.04-16** and **RAI 03.08.04-26** asked the applicant to include in the FSAR details of how the hydrodynamic loads were calculated and applied to the finite element model. The staff also asked the applicant to discuss whether the applicant's response meets SRP Section 3.7.3 Acceptance Criterion 14, and if not, to justify not doing so. In the September 16, 2009, response to **RAI 03.08.04-16** (ML092610374), the applicant describes the procedures of the analysis used to evaluate the dynamic response of the water masses in the basin. Furthermore, in the February 4, 2010, response to **RAI 03.08.04-26** (ML100480204), the applicant includes a markup of Subsection 3H.6.6.2.1, which now contains a more detailed description of the procedures of the analysis. During the May 2011 audit, the staff reviewed the calculations and noted that:

- Convective and impulsive masses were determined following Housner's method described in TID 7024, "Nuclear Reactors and Earthquakes," dated August 1963.

- Impulsive masses were added to the walls of the SSI model.

- Convective water masses were not added to the SSI model due to the very low frequency of 0.135 Hz (N-S) and 0.078 Hz (E-W), but equivalent horizontal loads resulting from applying the response spectrum accelerations at 0.50 percent damping were used in the design.

- The water mass in the basin is applied vertically as area load to the basemat.

- Horizontal pressures due to vertical acceleration are computed as the hydrostatic pressure times the maximum vertical spectral acceleration of 0.475g.

- The horizontal pressures due to horizontal acceleration are computed following the methodology in ASCE4-98 and TID 7024 for determining impulsive (rigid) and convective (sloshing) components and accelerations.

- Horizontal convective loads are applied to walls, buttresses, and columns between elevations of 41.75 ft and 71.00 ft.

- Because the SSI model accounted for the impulsive water masses, all relevant modes of combined fluid-tank vibration were included in the analysis.

As described above, the analysis meets all of the criteria in SRP Section 3.7.3 for above-ground tanks. A description of the analysis is included in the FSAR. Therefore, the staff found the response to be adequate. The applicant subsequently updated FSAR Subsection 3H.6.6.2.1 to incorporate the proposed changes in the response. **RAI 03.08.04-16** and **RAI 03.08.04-26** are therefore closed.

b. *Design of Submerged Columns due to Hydrodynamic Effects*

The column design for the submerged columns in the basin is based on the maximum accelerations obtained from the SASSI model along the column axis. However, no attached water masses on the submerged part of the columns, in addition to the self weight, were originally considered in the SSI model. Thus, in **RAI 03.07.02-28** the applicant was asked to consider attached water masses in the SSI model, because water masses would affect the column modes, frequency, and thus the accelerations along the column axis. This issue was also discussed with the applicant during the May 2011 audit. In the Supplement 4 response to **RAI 03.08.04-30** dated June 28, 2011 (ML11181A002), the applicant provides the modified acceleration values together with a design procedure whereby new scale factors were determined to amplify the horizontal acceleration acting on the column mass. For that purpose, eight SAP column models (four with additional masses and four without additional water masses) were developed covering all possible top and bottom boundary conditions. These eight models were subjected to the spectral accelerations obtained from all full basin SSI case analyses at the top and bottom of the columns. Since the modal response analysis performed for the column models uses only one input spectrum, the envelope of the top and bottom spectra was used in the dynamic analysis. Scale factors were obtained as the ratio between the resulting nodal accelerations along the column axis (with and without additional water masses). The column design was revised by applying these scale factors to modify the original accelerations that were used. The staff found this procedure acceptable.

c. *Effect of Seismic Wave Propagation on the RSW Tunnel*

During the audits of the design calculations for the RSW piping tunnels in March and May 2011 (audit report in ML111320094), the staff noted that seismic wave propagation effects were considered using the ASCE 4–98 methodology. This is an accepted industry practice, and its application in designing the RSW piping tunnels is considered adequate. However, it was noted that Rayleigh waves with an apparent wave velocity of 2,012 m/s (6,600 ft/s) were considered in the design, although 914.4 m/s (3,000 ft/s) for the same was specified in the FSAR. Also, it was noted that additional lateral pressure on the tunnel walls at the bend due to seismic wave propagation was not considered. The applicant agreed to revise the calculations for the tunnels and to address the issue as follows:

- Use an apparent wave velocity of 914.4 m/s (3,000 ft/s) per Subsection C3.5.2.1 of ASCE 4–98.

- Consider the maximum ground velocity based on the site-specific SSE maximum ground acceleration of 0.13g.

- Consider a triangular pressure distribution on the transverse leg of the tunnel near the bend, which is limited by the maximum passive pressure represented by a passive pressure coefficient of Kp = 3, if applicable.

- Revise the COL application to reflect these changes.

In the Supplement 5 response to **RAI 03.08.04-30** dated July 12, 2011 (ML11196A040), the applicant confirms that the calculations of the axial tensile strains; forces and moments at the tunnel bends; and soil pressures on the transverse leg of the tunnels near the bends due to seismic wave propagation for the RSW piping tunnels were revised using the above parameters with a maximum ground velocity of 15.85 cm/s (6.24 in./s) (which is based on 121.9 cm/s/g [48 in/sec/g] and the site-specific SSE maximum ground acceleration of 0.13g). The corresponding markups to revise FSAR Subsection 3H.6.6.2.2, Table 3H.6-6, and Figure 3H.6-247 resulting from the resolution of this item are also in this response. The staff's evaluation noted that the analysis meets the criteria in ASCE4–98, considers the passive pressures, and incorporates the changes in the FSAR. Therefore, the response is adequate and the issue is resolved. FSAR, Section 3H.6 was subsequently updated to incorporate the proposed changes in the response.

d. <u>Steel and Concrete Elements</u>

In **RAI 03.08.04-13** and **RAI 03.08.04-23,** the staff asked the applicant to include in FSAR Subsection 3H.6.6.3 a description of the various steel and concrete elements of the site-specific structures, including how these elements are designed and the design results. In the August 20, 2009, response to **RAI 03.08.04-13** (ML092370556), the applicant refers to the response to **RAI 03.07.01-13** (ML092360772) for the design information related to Category I structures. Later in the response to **RAI 03.08.04-23** dated February 10, 2010 (ML100550613), the applicant proposes changes to FSAR Subsection 3H.6.6.3 regarding the design of the UHS/RSW pump house building. This subsection includes codes and material specifications used in the design; references to tables and figures reporting section forces, governing load combinations, and bending and shear reinforcement and layout of slabs and walls; static and dynamic spring constants and their derivations based on the subgrade modulus; references to tables reporting section forces, governing load combinations, and resulting concrete bending and shear reinforcement and layouts for beams and columns. The referenced tables and figures were already provided in the Supplement 2 response to **RAI 03.07.01-13** (ML100050225) and are included in the design information for the UHS/RSW pump house as well as the RSW piping tunnels. Concrete design information for the RSW piping tunnels is also included in the February 10, 2010, response to **RAI 03.07.02-15** (ML100550613) containing the FSAR write-up for Subsection 3H.6.6.2.2. In the Supplement 4 response to **RAI 03.08.04-30** (ML11181A002), the applicant describes the effects of a mesh sensitivity study performed on the SSI analysis of the UHS/RSW pump house, which resulted in higher accelerations especially at the pump house roof slab. The modification factor of 1.41 was applied to the vertical accelerations and was considered in the design of the slab. The staff's evaluation of the applicant's response noted that:

- Table 3H.6-6, “Results of RSW Piping Tunnel Design,” showed very small differences between the required versus the provided reinforcement (0.7 versus 0.79 or 0.97 versus 1.00) or was non-existent (1.56 for both).

- In Table 3H.6-6 Note 3, it was not clear whether the additional reinforcement resulting from the SSE wave propagation had to be added to the reinforcement shown in the table.

- A sketch defining the reinforcing zones in the RSW piping tunnels was not included.

- It is not clear whether the passive pressure shown in Figure 3H.6-247 is applicable to the entire RSW piping tunnels design.

The staff asked the applicant to clarify these issues and to also provide the SSI soil pressures for the RSW piping tunnels. In the Supplement 6 response to **RAI 03.08.04-**30 dated September 12, 2011 (ML11259A056), the applicant clarifies these questions and states that in all cases, the provided reinforcement exceeded the required reinforcement thus meeting the code requirements. The applicant also states that an additional margin of about 11 percent was included in determining the required reinforcements for the RSW piping tunnel by dividing the calculated moments and axial forces by 0.9 (i.e., increased by 1/0.9 = 1.11). Note 3 in Table 3H.6-6 was revised to clearly state that additional reinforcements were needed to account for the seismic wave propagation effects. FSAR Figure 3H.6-348 was added to clearly identify the various reinforcement zones referred to in Table 3H.6-6. Regarding the design lateral soil pressures, the applicant states that the passive pressure shown in Figure 3H.6-247 is applicable to the entire RSW piping tunnel design. The applicant adds that the seismic soil pressures used in the design envelope the SSI and SSSI soil pressures, with the exception of a small portion along the wall height where the SSI soil pressure from the separated soil case is higher. But the tunnel wall design was controlled by passive pressure. The staff found this response technically acceptable and adequate since it clarified the issues identified by the staff. The FSAR markups included in the response were subsequently incorporated into the FSAR. Therefore, this issue is resolved.

e. *<u>Concrete Reinforcement Design Tables</u>*

Required concrete reinforcement for walls and slabs of the UHS/RSW pump house resulting from the design process are in FSAR Tables 3H.6-7 and 3H.6-8. These tables describe the location, direction, and face of the structural element and refer to the reinforcement layout drawing numbers and reinforcement zone numbers. The staff noted that it was difficult to determine where in the structure the reinforcement zone belonged. Therefore, the applicant was asked to provide additional descriptive plans or figures showing the actual location of each reinforcing zone within the structure. In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369) Item 2, the applicant provides the FSAR markup with a reference to the new Figures 3H.6-40a, b, and c, which clarify the wall and slab labeling conventions for the UHS/RSW pump house building. The staff considered the additional information adequate. This issue is therefore resolved. The applicant incorporated the proposed changes into the FSAR Revision 8.

f. *<u>Lateral Soil Pressures</u>*

RAI 03.08.04-12 asked the applicant to include in the FSAR the calculated lateral soil pressures used for the design of site-specific structures. In the August 20, 2009, response to **RAI 03.08.04-12** (ML092370556), the applicant refers to the FSAR markup of Subsections 3H.6.4.3.1.4 and 3H.6.4.3.3.3 provided with the response to **RAI 03.07.01-13** (ML092360772). The staff found that the information included in the response provided only a definition of the lateral soil pressures. Therefore, **RAI 03.08.04-22** asked the applicant to include in the FSAR the values of the lateral soil pressures used in the analysis. In the revised response to **RAI 03.08.04-22** dated September 15, 2010 (ML102630145), the applicant provides revised markups of FSAR Subsections 3H.6.4.3.1.4 and 3H.6.4.3.3.3 that refer to lateral soil pressure diagrams 3H.6-41 through 3H.6-50. These FSAR markups were submitted in the Supplement 2 response to **RAI 03.07.01-13** (ML100050225) for the UHS basin, the RSW pump house, and the RSW tunnel. From the soil pressure diagrams included in the FSAR, it was not clear how the soil pressures were used in the design and stability evaluation of site-specific structures. Therefore, in the Supplement 1 response to **RAI 03.08.04-17** dated March 7, 2011 (ML110730067), the applicant provides markups of FSAR Section 3H.6 for the UHS/RSW pump house and the RSW piping tunnels that include soil pressure diagrams 3H.6-41 through 3H.6-44 (revised); 3H.6-232 through 3H.6-240; and 3H.6-245 through 3H.6-247 providing the at-rest, dynamic at-rest, active, and passive soil pressure profiles for the UHS basin, the RSW pump house, and the RSW tunnel. None of the soil pressure diagrams provided with the response included SSSI effects, however. The SSSI incremental seismic soil pressure diagrams, which include the effects of the UHS, RB, and RWB on the RSW piping tunnels, are in the Revision 1, Supplement 1 response to **RAI 03.07.02-24** dated March 7, 2011 (ML110730067) and the Supplement 6 response to **RAI 03.08.04-30** (ML11259A056). The SSSI incremental soil pressures for the UHS basin and the RSW pump house were in the March 15, 2011 response to **RAI 03.07.02-22** (ML110770440). Subsequently, the applicant revised the soil pressure diagrams considering the SSI and SSSI effects in the Revision 1, Supplement 1 response to **RAI 03.07.01-29** dated November 28, 2011 (ML113360516), for the UHS basin, the RSW pump house, and the RSW tunnel to address various issues discussed with the applicant during the audits in May 23, 2011 and July 25, 2011. Detailed discussions of these issues are in Subsection 3.7.2.4 of this SER. Soil pressure diagrams in the above responses are incorporated into Revision 7 of the FSAR. During the audit in February 2012, it was noted that lateral seismic soil pressures used for the design were not clearly marked in the soil pressure diagrams for the RSW tunnel. Also, the soil pressures used for the stability evaluation of the RSW tunnel were not included in the FSAR. In Supplement 5 response to **RAI 03.07.02-13** dated April 10, 2012 (ML12103A369), the applicant provides the lateral soil pressure diagrams used for stability evaluation of the RSW tunnel and revises the seismic soil pressure diagrams for the RSW tunnel to include the design pressure diagrams. These changes have been incorporated into the FSAR Revision 8. A discussion of the lateral soil pressures used for design of the UHS basin, the RSW pump house, and the RSW tunnel is presented below.

UHS Basin

The basin walls are designed for static and dynamic lateral soil pressures. The at-rest lateral soil pressures shown in Figure 3H.6-238 are used to calculate the static lateral soil pressure on the UHS basin walls. Hydrostatic pressure and surcharge pressure of 14.36 kPa (300 psf) are included in the at-rest lateral soil pressure. Dynamic soil pressure used for the design of the UHS basin walls is shown in Figure 3H.6-220. The designed dynamic soil pressure envelops the seismic soil pressures calculated considering the SSI, SSSI, and the provisions of ASCE 4-98. Both empty and full basin conditions are considered for calculating the seismic soil pressures. Details of the SSI and SSSI analyses for calculating the seismic soil pressures are discussed in Subsection 3.7.2.4.4 of this SER. Driving and resisting lateral soil pressures used for the stability evaluation are shown in Figures 3H.6-46 and 3H.6-49, respectively. In addition, walls are also designed for the passive lateral soil pressure required to be developed for the stability evaluation. The staff's evaluation found that the applicant has appropriately considered the static lateral soil pressure for designing the UHS basin walls by using at-rest soil pressure that are based on earth pressure on non-yielding walls and are conservative. The staff also considered computing the dynamic lateral soil pressure as the envelope of the SSI, SSSI, and ASCE 4–98 dynamic lateral earth pressures that provides a conservative estimate of the expected lateral dynamic soil pressure and meets the intent of the guidance in SRP Section 3.8.4. The method used by the applicant to determine the driving and resisting earth pressures for the stability evaluation is discussed along with the discussion of the stability evaluations. Based on the above information, the staff concluded that the lateral soil pressures used by the applicant for the design of the UHS basin are acceptable.

RSW Pump House

The different soil pressure conditions for the RSW pump house walls are shown in the lateral pressure diagrams: (a) dynamic at-rest pressures (Figures 3H.6-41 through -43); (b) static active, passive, and at-rest pressures (Figures 3H.6-233, -234, -236, -237, -239, and -240); (c) seismic pressures obtained from the SSI, SSSI and ASCE 4-98 analyses (Figures 3H.6-218, and -219); and (d) driving and resisting static and dynamic lateral pressures used in the stability analyses (Figures 3H.6-45, -47, -48, and -50). At-rest lateral soil pressures shown in Figures 3H.6-239 and 3H.6-240 are used to calculate the static lateral soil pressure on the RSW pump house walls. Hydrostatic pressure and surcharge pressure of 14.36 kPa (300 psf) are included in the at-rest lateral soil pressure. The lateral soil pressure diagrams in Figures 3H.6-218 and 3H.6-219 show that the dynamic pressure used to design the walls envelopes the results obtained from the seismic analyses performed to cover different soil cases, full and empty basin conditions, loading effects from nearby structures, and seismic soil pressure based on the provisions of ASCE 4–98. The lateral soil pressures on the north wall (Figure 3H.6-219) resulting from the SSSI analysis with other buildings are in several RAI responses: **RAI 03.07.02-22** (ML110770440); Supplements 3 and 4 to **RAI 03.07.01-27** (ML11143A054 and ML11192A043); and Revision 1 of Supplement 1 to **RAI 03.07.01-29** (ML113360516)). The diagrams show a pronounced peak between a 3.05-m (10-ft) and 3.96-m (13-ft) depth and a depth of about 13.72-m (45-ft), which exceeds the seismic design pressure. In the Supplement 3 response to **RAI 03.07.01-29** dated April 10, 2012 (ML12103A369), the applicant explains that the induced out-of-plane shear and moment in each wall panel from the design soil pressures are greater than those from the SSSI soil pressures. In addition, this response provides the markup of FSAR Subsection 3H.6.4.3.3.3 to document the basis for accepting localized higher lateral soil pressures on the RSW pump house north wall. The applicant further states that the RSW pump house north wall, in addition to the SSI and SSSI soil pressures, is designed for the seismic soil pressure per ASCE 4–98 and the passive soil pressure (Kp=1.2) without taking any credit for a lack of soil pressure at its juncture with the RSW piping tunnel. The staff's evaluation concluded that the applicant has appropriately considered the static lateral soil pressures for the design of the RSW pump house walls by using at-rest soil pressures that are based on the earth pressure on non-yielding walls and is conservative. In addition, the walls are designed for passive soil pressure required for the stability evaluation (Kp=1.2). The staff also considered the computing dynamic lateral soil pressure as the envelope of the SSI, SSSI, and ASCE 4–98 dynamic lateral earth pressures that provides a conservative estimate of the expected lateral dynamic soil pressure and meets the intent of the guidance in SRP Section 3.8.4. The localized exceedances of the seismic soil pressures on the north wall of the RSW pump house is acceptable because the out-of-plane moments and shears based on the envelope design of the lateral soil pressures are greater than the out-of-plane moments and shears using the actual seismic lateral soil pressures. The method used by the applicant to determine the driving and resisting earth pressures for the stability evaluation is discussed along with the discussion of the stability evaluations. Based on the above information, the staff concluded that the lateral soil pressures used by the applicant to design the RSW pump house are acceptable.

RSW Tunnel

The different soil pressure conditions for the RSW tunnel are shown in the lateral pressure diagrams: (a) dynamic at-rest pressures (Figure 3H.6-44); (b) static active, at-rest and passive pressures (Figures 3H.6-245 through -247); (c) seismic pressures obtained from the SSI, SSSI, and ASCE 4-98 analyses (Figures 3H.6-212 through -215). In addition, Figures 3H.6-216 and 3H.6-217 provide lateral seismic soil pressures on the RSW tunnel east and west walls, respectively, for the UB in situ soil cases including the effects of vertical excitation. As no driving and resisting lateral pressures used in the stability analyses were reported, the applicant was requested during the audit in February 2012 (ML120660018) to include the missing information in the FSAR. During the same audit, the applicant was also requested to include the missing envelope design pressures in Figures 3H.6-214 and -215. In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant provides revised Figures 3H.6-214 and -215, which include the envelope design lateral seismic soil pressures on the RSW tunnel north and south walls near the RSW pump house. The response also includes Figures 3H.6-253 and -254 showing the driving and resisting lateral soil pressures used for the stability evaluation of the tunnel. The applicant also proposes markups to FSAR Figures 3H.6-214, -215, -253, and -254. In FSAR Subsection 3H.6.6.2.2, the applicant describes the lateral soil pressures used for the design of the RSW tunnel. Static lateral soil pressures used in the design are based on the at-rest lateral soil pressure, including the effects of the hydrostatic pressure and a surcharge pressure of 23.94 kPa (500 psf). The staff considered this information acceptable, because the at-rest lateral pressure provides a conservative estimate of the static lateral pressure on structures. The dynamic lateral soil pressure due to the SSE used for the design is the envelope of soil pressure calculated using the methodology defined in Subsection 3.5.3.2.2 of ASCE 4–98, in the SSI analysis, and in the SSSI analysis. Details of the SSI and SSSI analyses and their staff's evaluations are in Subsection 3.7.2.4.4 of this SER. Furthermore, in the response to RAI 03.08.04-30 S6, the applicant states that the entire RSW tunnel was designed using the passive pressure shown in Figure 3H.6-247. The staff noted that the SSI pressures for the soil separated case reported in Figures 3H.6-212 and -213 locally exceeds the envelope design pressure between 6.71 m (22 ft) and 7.32 (24 ft) depth below grade. In the Supplement 6 response to **RAI 03.08.04-30** (ML11259A056), the applicant compares the out-of-plane moments and shears due to the SSI soil separated case with the out-of-plane moment and shear used in design. The out-of-plane moment and shear used in the design were larger and controlled the design.

The staff conducted a review of the structural design report for the RSW piping tunnels during the February 2012 audit. During this review, the staff noted that FSAR Figures 3H.6-216 and -217 showing the lateral seismic soil pressures for the UB in situ soil case, including the effect of the vertical excitation on the RSW piping tunnel east wall and west wall, respectively, did not include the envelope soil pressure used in the design, even though the demand on the east and west walls appear to exceed the design envelope lateral pressure for the east and west walls from 1.52 m (5 ft) below grade to grade level. This shortcoming is not considered to have any significant impact on the design because the bottom elevation of the tunnel roof slab is 1.52-m (5-ft) below grade, and the higher pressures shown up to a depth of 1.52-m (5-ft) will be directly transmitted to the tunnel roof slab. Furthermore, these two figures were only intended to show the negligible effect of the vertical accelerations on the horizontal pressures. The staff's evaluation concluded that the methodology used by the applicant for determining the dynamic lateral soil pressure on the RSW tunnel is acceptable, because the envelope of lateral soil pressures from the SSI and SSSI analysis, lateral soil pressure calculated using the ASCE 4–98 methodology, and the lateral passive pressures used for the design all meet the intent of the guidance in SRP Section 3.8.4. The method used by the applicant to determine the driving and resisting earth pressures for the stability evaluation is discussed along with the discussion of the stability evaluations.

B.1.5 Structural Acceptance Criteria

In FSAR Subsection 3H.6.4.1, the applicant has provided information regarding the applicable design codes. Because the original list of codes was incomplete, **RAI 03.08.04-8** asked the applicant to provide a more complete list of standards and RGs. The applicant provides this information in the response to **RAI 03.07.01-13** dated August 20, 2009 (ML092360772). For each loading combination delineated in FSAR Subsection 3H.6.4.3.4, the structural acceptance criteria appear in the load combinations that correspond to the respective code (i.e., ACI 349 and RG 1.142 for concrete structures; ANSI/AISC N690–1994 including Supplement 2 (2004) for steel structures; and ACI 350.1 for environmental requirements.) The staff found the acceptance criteria in the loading combinations acceptable, because they meet the provisions of the codes that were evaluated and found to be acceptable. Also, by referencing these codes—including the edition, revision, and date of issuance—the applicant has provided the design criteria related to stresses, strains, gross deformations, factors of safety, and other parameters that quantitatively identify the margins of safety. Therefore, the structural acceptance criteria included in the FSAR are acceptable.

B.1.6 Materials, Quality Control, and Special Construction Techniques

In FSAR Subsection 3H.6.4.4, the applicant provides information regarding the structural materials used in the design of the site-specific Category I structures, including the UHS/RSW pump house and the RSW piping tunnels. The structural materials that are listed conform to the steel and concrete codes used for the design. No special construction techniques are proposed or described in the FSAR, and no explicit mention of quality control programs is included in the FSAR. SRP Section 3.8.4 provides specific guidance for applicants to provide information about:

(a) the concrete ingredients and reinforcing bar splices;

(b) the nondestructive examination of the materials to determine physical properties, the placement of concrete, and erection tolerances;

(c) the extent to which the materials and quality control programs comply with ACI 349, with additional criteria provided by RG 1.142 for concrete and ANSI/AISC N690-1994 including Supplement 2 (2004) for steel, as applicable; and

(d) welding of reinforcing bars if proposed, to comply with ASME Boiler and Pressure Vessel Code (Code) Section III, Division 2.

RG 1.206 further specifies that for quality control in general, verifying the extent of compliance with the applicable provisions of SRP Sections 3.8 and 17.5 and recommendations of RG 1.55, “Concrete Placement in Category I Structures.” The staff asked the applicant to incorporate the above information into the FSAR. In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant includes the markup of FSAR Subsection 3H.6.4.4.7, “Materials and Quality Control,” that provides the requested information and states that concrete ingredients, reinforcing bar splices, nondestructive examination of the materials to determine physical properties, placement of concrete, and erection tolerances will meet the requirements of ACI 349 supplemented by the RGs, codes, and standards referenced in the DCD and the FSAR. The response further states that materials and quality control programs comply with ACI 349, with additional criteria provided by RG 1.142 for concrete and ANSI/AISC N690–1994, including Supplement 2 (2004), for steel. The staff found the applicant’s response and FSAR update acceptable, because the proposed material and quality control provisions are according to the codes and the standards are accepted for use. The applicant includes all requested information in the FSAR markup. Therefore, this issue is resolved. Based on the above discussions, the staff concluded that the information from the applicant pertaining to materials and quality control meets the guidance in SRP Section 3.8.4 and is therefore acceptable. The applicant incorporated the proposed changes into the FSAR Revision 8.

B.1.7 Testing and In-Service Requirements

According to SRP Section 3.8.4 and RG 1.206, the applicant should specify any testing and ISI requirements applicable to critical areas of the UHS/RSW pump house and the RSW piping tunnels to meet the requirements of 10 CFR 50.65 and the guidance in RG 1.160. However, it was not clear how the ISI of normally inaccessible, below-grade concrete walls and foundations is performed. Therefore, the staff asked the applicant to confirm how the ISI requirements are accomplished for below-grade concrete walls and foundations to (a) examine for signs of degradation the exterior, exposed portions of below-grade concrete, when excavated for any reason; and (b) conduct periodic site monitoring of ground water chemistry to confirm that the ground water remains nonaggressive. The applicant states in the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369) that site-specific seismic Category I structures are included in the scope of the Design Reliability Assurance Program (DRAP). Because FSAR Section 17.6S1.1b includes all SSCs identified as risk-significant via the DRAP in the initial maintenance rule scope, these site-specific seismic Category I structures are included in the Maintenance Rule Program. This program comprises monitoring and maintaining the requirements for the structural materials used in the design of the site-specific seismic Category I structures that will be implemented in accordance with 10 CFR 50.65 and RG 1.160, as described in FSAR Section 17.6S and Table 13.4S-1. The applicant also states that the periodic monitoring of the ground water chemistry is described in FSAR Subsection 2.4S.12.4. The applicant also provides a markup of the new FSAR Subsection 3H.6.4.4.6, “Testing and ISI Requirements,” with the above information. The staff found that the applicant has appropriately addressed the issue of testing and ISI of seismic Category I structures, including the below-grade concrete walls and foundation, by including them in the Maintenance Rule Program described above. The applicant’s response also includes FSAR markups. The applicant incorporates these markups into the FSAR Revision 8.

As described above, the applicant’s response regarding the design information adequately addresses the staff’s requests for additional information and clarification. Therefore, **RAI 03.08.04-13, RAI 03.08.04-23,** and **RAI 03.08.04-30** (only portions evaluated here) are considered closed. The staff concluded that the design information included in FSAR Section 3H.6 for the UHS basin, the RSW pump house, and the RSW tunnel, including the proposed markups with the responses mentioned above, are adequate and meet the guidance in SRP Section 3.8.4.

B.2 Design Information for the DGFOSV

In the response to **RAI 03.07.01-19** dated February 4, 2010 (ML100480204), the applicant provides analysis and design information for the DGFOSV that was not previously in the FSAR. The response included a markup of the new FSAR Section 3H.6.7, which provides a general description of three DGFOSV including their sizes and locations. However, the information included in the response did not describe how structural analysis and design of the structure was performed. Therefore, in **RAI 03.08.04-30, Item 8,** the staff asked the applicant to provide complete structural analysis and design information for the DGFOSV to ensure that they meet Acceptance Criteria 1 through 7 of SRP Sections 3.8.4 and 3.8.5. The applicant provided additional analysis and design information in multiple responses to address various requests from the staff for additional information and clarification. The following RAI responses included additional information and clarification for the DGFOSV, and any FSAR markups provided with the responses were subsequently incorporated into FSAR Revision 7: Revision 2 to **RAI 03.07.01-19** (ML101620284), **RAI 03.07.01-27** (ML102630145), Revision 1, Supplement 1 to **RAI 03.07.01-27** (ML110730067), Supplements 3 and 4 to **RAI 03.07.01-27** (ML11143A054 and ML11192A043), **RAI 03.07.01-29** (ML11168A168), Revision 1 of Supplement 1 to **RAI 03.07.01-29** (ML113360516), **RAI 03.07.01-30** (ML11335A232), Revision 1 of Supplement 3 to **RAI 03.07.02-13** (ML11335A232), Supplement 4 to **RAI 03.07.02-13** (ML11335A232), Revision 1 to **RAI 03.07.02-32** (ML11364A098), Supplement 1 to **RAI 03.08.04-17** (ML110730067), Revision 1 to **RAI 03.08.04-25** (ML101090143), Revision 1 to **RAI 03.08.04-30** (ML110770440), Supplements 2, 6, and 7 to **RAI 03.08.04-30** (ML11143A054, ML11259A056, and ML11322A106), Revision 1, Supplement 1 to **RAI 03.08.04-31** (ML110330168), Supplement 1 to **RAI 03.08.04-34** (ML11259A056), Revision 1 to **RAI 03.08.04-35** (ML11181A002), Revision 2 to **RAI 03.08.05-2** (ML101340651), and Supplement 1 to **RAI 03.08.05-4** (ML103230128). Subsequently, the applicant included additional information and clarifications in the following responses: Supplements 1 and 2 to **RAI 02.03.01-24,** Supplement 3 to **RAI 03.07.01-29,** and Supplements 5 and 6 to **RAI 03.07.02-13.** The applicant incorporated these changes into the FSAR Revision 8.

The staff's review of the analysis and the design information for the DGFOSV against the SRP Section 3.8.4 acceptance criteria is discussed below.

In the Revision 1 response to **RAI 03.08.04-30** (ML110770440), the applicant adds Section 3H.6.7, "Diesel Generator Fuel Oil Storage Vaults (DGFOSV)," to the FSAR that includes a description of (i) the structure; (ii) stability evaluation; (iii) applicable codes and loadings; (iv) a description of the wall, slab, and foundation structural design, soil springs, uplift analysis, tables and figures reporting section forces, and governing load combinations; (v) resulting concrete bending; and (vi) shear reinforcement and layouts.

B.2.1 Description of the Structure

In FSAR Section 3H.6.7, the applicant provides a general description of the three DGFOSV pertaining to each proposed unit at the STP site. Figure 3H.6-221 shows a partial site plan with the locations of the DGFOSV relative to other structures. The DGFOSV are described as a reinforced concrete structure located below grade with an access room above grade. The DGFOSV are buried in the structural backfill. The embedment depth to the bottom of the 0.61 m (2 ft) thick mudmat is approximately 13.72 m (45 ft), the maximum height from the bottom of the mudmat is approximately 18.60 m (61 ft), and the basemat dimensions are approximately 24.84 by 14.6 m (81.5 by 48 ft). The staff noted that the description did not include plans and sections of the DGFOSV with sufficient information to define the primary structural aspects and elements as specified in SRP Section 3.8.4. The applicant was therefore asked to include in the FSAR plans and sections with sufficient information about the primary structural layout and elements. The applicant in the Supplement 5 response to **RAI 03.07.02-13** dated July 12, 2011(ML12103A369), includes FSAR Figures 3H.6-250, -251, and -252 that provide the requested information for the DGFOSV with dimensions of the major structural elements. Design results for the various structural elements are provided in Table 3H.6-11. Designation of the various reinforcing zones is shown in Figure 3H.6-141, and the corresponding reinforcement zone figures are provided in Figures 3H.6-142 through 3H.6-208. Based on a review of the above information, the staff concluded that the description included in the FSAR for the DGFOSV structure meets the guidance in SRP Section 3.8.4, and is acceptable. Therefore, this issue is resolved. The applicant incorporated these changes into the FSAR Revision 8.

B.2.2 Applicable Codes, Standards and Specifications

In FSAR Subsection 3H.6.7.1, the applicant states that the applicable codes, standards, and specifications from FSAR Section 3H.6.4 are used for the analysis and design of the DGFOSV. Use of these codes, standards, and specifications for seismic Category I structures were already evaluated above under Subpart B.1.2 for UHS/RSW pump house and RSW piping tunnels. Therefore, the staff concluded that referencing these codes, standards, and specifications for the analysis and design of the DGFOSV structure is acceptable.

B.2.3 Loads and Load Combinations

In FSAR Subsection 3H.6.7.1, and by reference in Section 3H.6.4, the applicant provides information regarding the applicable loads and load combinations used for the DGFOSV design. These loads and load combinations were evaluated under Subpart A.4 in Subsection 3.8.4.4.2 above and are acceptable for the design of seismic Category I structures that meet the provisions of the concrete and steel codes and other applicable standards described elsewhere in this FSAR. The underground vault walls are subject to static and dynamic lateral earth pressures and additional dynamic soil pressures from the nearby heavy structures, and are discussed below under Subpart B.4 located in this subsection below. Therefore, the staff concluded that the loads and load combinations used for the DGFOSV design are acceptable.

B.2.4 Design and Analysis Procedures

In FSAR Subsection 3H.6.7.2, the applicant provides information regarding the structural design and analysis of the DGFOSV. During the March 2011 audit, the staff reviewed the analysis and design of the DGFOSV calculation entitled, “Basic Structural Design of Diesel Generator Fuel Oil Storage Vault.” The staff’s review is summarized below.

The analysis is performed with the help of a SAP2000 finite element model using spring, beam, and thick shell finite elements. Regarding the finite element discretization of the DGFOSV finite element model, the staff received an excerpt of the SAP manual about joint connectivity that recommends shell element aspect ratios near unity but not greater than four for the best results. A mesh sensitivity attachment was presented containing a mesh study performed for a UHS wall, which compares hand calculated (theoretical solution) values with the SAP results for different mesh sizes. Moments and deflections are shown to differ by less than 1 percent if the wall span is discretized with at least eight elements between supports. This modeling approach was followed for the DGFOSV analysis and is deemed technically acceptable for design purposes. The seismic design is performed with equivalent static forces, whereby the inertial forces are computed as products of masses and accelerations. The accelerations are taken from the SSI analysis of the vault. Static co-directional earthquake response components are combined by using the SRSS rule, and the resulting base shear and total axial base force from the SAP are compared to the corresponding values of the SSI (and/or SASSI) model. It was noted that the amplification had to be applied to the vertical acceleration of the SAP model in order to match the total vertical force with that from the SSI analysis. A more detailed discussion of this issue is presented below.

To represent the soil flexibility, two separate sets of soil springs are used. One set corresponds to uniform, uncoupled Winkler springs. The second set, so-called pseudo-coupled springs, is intended to simulate the effect of coupled springs and consists of uncoupled springs with a stiffness that varies with the distance from the center of the foundation. Two separate finite element models are established with identical superstructure but with the different set of springs (uniform and non-uniform) attached to the base. Two sets of springs are also required to represent static and dynamic soil behavior. While the horizontal springs are the same, the dynamic stiffness of the vertical springs is higher and needs to be accounted for. This is accomplished by defining a static and a dynamic finite element model, both with identical superstructure but with different springs (static and dynamic) attached to the base. The structural design is performed enveloping the results from all of the models, whereby the dynamic model is used only for dynamic load cases (i.e., seismic). The responses are combined later with the results from the static model. If uplift is detected (soil springs in tension), a separate SAP model, with compression-only springs is developed to evaluate the effects of the basemat uplift on the structural response. For the most critical load combination showing the largest uplift effects and response quantities (forces, displacements, etc.) from the linear and “nonlinear” (modeled with compression-only soil springs) analysis are compared, and a correction factor is determined which, if greater than one, is used to amplify the linear responses for all load combinations. The applicant also applies this approach to the design of other buildings that the staff has already reviewed. The staff considers this method to be an approximate but acceptable approach for handling uplift effects.

The fuel oil tank located within the vault also uses the SAP model. Physically, the tank is supported on three saddles and is also attached to the basemat with guy wires. The tank is modeled with two nodes representing the C.G. of the tank. These two nodes are rigidly attached to the base slab elements. The rigid assumption is justified by stating that the vendor specification will ask for a rigid construction that will also prevent fluid sloshing effects. The inertial forces corresponding to the fuel oil tanks are applied at these two nodes. Loads representing the effects from accidental torsion follow the procedure in ASCE4–98 and in SRP Section 3.7.2, SRP Acceptance Criterion 11 and are applied to model nodes to simulate the additional torsional demand. The staff found this procedure acceptable because it follows the guidance in SRP Section 3.7.2.

The below grade roof is designed as a composite section consisting of steel beams acting in conjunction with the concrete slab. The verification of the composite steel-concrete beams for vertical loads is based on hand calculations, whereas the verification of the slab as a diaphragm for in-plane shear is based on a FEA using a SAP model in which vertical loads are suppressed. The walls were designed for static and dynamic lateral earth pressures, for lateral pressures resulting from surcharges applied at ground level, for hydrostatic and hydrodynamic pressures from ground water, and for the increased static lateral pressures resulting from adjacent nearby buildings. This issue is discussed later under the topic “Lateral Soil Pressures.” Bearing pressures are not evaluated in the same report. A separate MACTEC (Geotechnical) report evaluates the bearing safety factors by comparing the total demand with the ultimate soil bearing capacity. The procedure for verifying the bearing capacity and the demand (Hansen’s method) follows the methodology described in FSAR Subsection 2.5S.4.10. A more detailed discussion is presented under Subpart A.5.4 in Subsection 3.8.4.4.2 above. Wind loads were determined according to ASCE 7–88 using the following parameters:

Exposure D, wind velocity V = 110 mph, and Importance Factor I = 1.11.

This older code (ASCE 7–88) was used because the DGFOSV was treated as a standard structure and was thus designed according to DCD standards. The staff found this approach acceptable because the calculated velocity pressure using the above parameters and the provisions of ASCE 7–88 is greater than the velocity pressure based on site-specific wind and the provisions of ASCE 7–05.

The tornado design is based on a wind velocity of 482.7 km/h (300 mph), which corresponds to DCD design criteria. Basic missile velocity was taken as 370 km/h (230 mph), which is considered adequate as the site-specific tornado velocity corresponding to Region II is 321.8 km/h (200 mph). The verification of the tornado missile impact on the above grade portions of the structure is performed according to the guidance in RG 1.76. Calculations show that the sphere impact is not relevant, whereas car impact loads are the design that governs the loads. In order to provide an enveloping design, different impact locations on the access shaft and hood are considered. The staff's review determined that the design procedures are adequate and acceptable. Furthermore, the staff's review specifically addressed the following two issues.

a. *Seismic Adjustment Factor*

As already described in the previous section, the structural analysis and design of the DGFOSV and other Category I structures is performed in a two-step procedure. In the first step, a dynamic SSI analysis is performed using the computer program SASSI. From this analysis, maximum absolute accelerations are obtained that are subsequently used as the seismic excitation in the second step. In the second step, a finite element model is developed with the program SAP which is subject to equivalent static forces determined by converting the maximum accelerations obtained in the first step into static equivalent loads applied at the corresponding nodal points. As a verification of this approach and since both models are expected to produce similar results, the forces acting on the structure in the horizontal and vertical direction obtained in both models, SASSI and SAP, are compared for consistency. In general, forces obtained from the equivalent static model (SAP) are larger than those from the SSI model (SASSI) because of inherent conservatism in the equivalent static model.

During the March 2011 audit review of the DGFOSV design calculations, the staff noted that since the resulting vertical force from the SAP model was less than the corresponding value from the SASSI model, the equivalent vertical accelerations used in the SAP analysis had to be amplified by a factor of 1.27 in order to match the base axial force obtained from the SSI analysis. No such adjustment was needed in the horizontal direction, as both base shears were similar. Because both the SASSI and the SAP models are based on the same geometry, layout, and material, and the absolute accelerations on the SAP model are expected to yield conservative results, it was not apparent why an additional, relatively large amplification was needed to obtain comparable total base seismic loads in the vertical direction. The staff asked the applicant to provide a justification regarding the different behavior of the two structural models.

In the Supplement 2 response to **RAI 03.08.04-30** dated May 16, 2011 (ML11143A054), the applicant states that even though the structural models in SASSI and SAP are based on the same geometry and have similar weight properties, the SASSI analysis accounts for the interaction effect of the structure with the soil around and below the structure. On the other hand, in the equivalent static analysis with the SAP model, the SSI effect is not accounted for, since the soil is not modeled. Therefore, the seismic design loads obtained from the SSI analysis are more accurate. Furthermore, the response states that for the structural design, the equivalent static method is used to facilitate the design for all applicable loads and load combinations, and the equivalent horizontal and vertical accelerations to be used in the equivalent static model are calculated using the accelerations from the SSI analysis. The applicant also states that depending on the structural dynamic characteristics and structure's interaction with the soil, the behavior of the horizontal and vertical responses may be different. Therefore, amplification factors for accelerations in horizontal and vertical load calculations may not necessarily be the same. As a conclusion, the response states that since the design has ensured that the equivalent static loads bound the loads from the SSI analysis, there is no impact on the design. The staff's evaluation noted that it is difficult to single out the reasons for the differing model response in both directions. However, the staff agreed with the applicant that there is sufficient conservatism built into the procedures, so as not to influence the final design. Therefore, this issue is resolved.

b. *Lateral Soil Pressures*

In the Supplement 1 response to **RAI 03.08.04-17** (ML110730067), the applicant provides a markup for FSAR Section 3H.6.7 stating that the lateral soil pressures used in the DGFOSV design are shown in Figures 3H.6-241 through 3H.6-244. The referenced figures incorporate the at-rest, dynamic at-rest, active, and passive soil pressure profiles without the SSSI effects. The SSSI incremental seismic soil pressure diagrams, which include the effects of the nearby structures (DGFOT, crane wall, UHS, RSW piping tunnels, and RB) on the DGFOSV are depicted in Figures 3H.6-226 through 3H.6-231, which are included in the Revision 1 of Supplement 1 response to **RAI 03.07.01-27** dated March 7, 2011 (ML110730064). Because no driving and resisting lateral pressures used in the stability analyses were reported, the applicant was requested during the audit in February 2012 to include the missing information in the FSAR.

In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant provides Figures 3H.6-255 through -257 showing the driving and resisting lateral soil pressures used for the stability evaluation of the vault. The storage vault walls are analyzed for load combinations that include static and dynamic loads and involve three different soil pressures—active, at-rest and passive. The resulting load components are shown in several lateral pressure diagrams as follows: (a) dynamic at-rest pressures (Figure 3H.6-242); (b) static active, at-rest, and passive pressures (Figures 3H.6-241, -243, and -244); and (c) seismic pressures obtained from SSI, SSSI, and ASCE 4-98 analyses (Figures 3H.6-226 through -231). A surcharge of 23.94 kPa (500 psf) as well as hydrostatic and hydrodynamic loads are also included in the corresponding load combinations. Included in Figures 3H.6-226 through 3H.6-231 is the dynamic soil pressure used for the design, which envelopes the dynamic pressures obtained from the SSI, SSSI, and ASCE 4–98 analyses. The staff noted that the SSI pressures reported in Figures 3H.6-228 through -231 locally exceed the envelope design pressure between 10.67 m and 11.28 m (35 ft and 37 ft) depth below grade. In Revision 1 of the Supplement 1 response to item D.2.2 of **RAI 03.07.01-29** dated December 28, 2011 (ML113360516), the applicant compares the out-of-plane moments and shears resulting from the SSI soil pressures exceeding the design pressures with the out-of-plane moment and shear used in the design of walls No. 1 through 4. The comparison was carried out with the help of SAP models generated for each of the four walls under consideration. The comparison showed that the out-of-plane moment and shear used in the design were larger and the wall capacity still had a margin of about 90 percent (out-of-plane shear, Wall #2) with respect to the applied lateral soil pressures. The staff's evaluation concluded that the applicant has appropriately considered the static lateral soil pressures for the design of the DGFOSV walls by using the earth pressures resulting from the two possible conditions: the yielding and non-yielding wall. The staff also considered computing the dynamic lateral soil pressure as the envelope of SSI, SSSI, and ASCE 4–98 dynamic lateral earth pressures provides a conservative estimate of expected lateral dynamic soil pressure and meets the intent of the guidance in SRP Section 3.8.4. In addition, the walls are designed for the passive soil pressure that is required for the stability evaluation. The method used by the applicant to determine the driving and resisting earth pressures for the stability evaluation is discussed with the discussion of the stability evaluations. Based on the above information, the staff concluded that the lateral soil pressures used by the applicant for the DGFOSV design are acceptable.

In the review performed during the May 2011 audit, the staff asked the applicant to consider in the design of the DGFOSV the surcharge loads originating from the large equipment access building (LEAB), which is located directly west of the DGFOSV. In the Supplement 4 response to **RAI 03.07.01-27** dated July 6, 2011 (ML11192A043), the applicant provides a markup to FSAR Section 3H.6.7 and states that the LEAB foundation will be designed so that the surcharge load on the walls of the adjacent DGFOSV is insignificant. This change is included in the current FSAR revision. Therefore, this issue is resolved.

B.2.5 Structural Acceptance Criteria

In FSAR Subsection 3H.6.7.1 and with a reference to Section 3H.6.4, the applicant provides information regarding the applicable design codes and load combinations for the DGFOSV. For each loading combination in Section 3H.6.4, the structural acceptance criteria appear in the corresponding code. That is to say ACI 349 and RG 1.142 for concrete structures; and ANSI/AISC N690–1994, including Supplement 2 (2004), for steel structures. The staff considers that by referencing these codes including the edition, revision, and date of issuance, the applicant has specified the structural acceptance criteria related to stresses; strains; gross deformations; factors of safety; and other parameters that quantitatively identify the margins of safety. Therefore, the staff concluded that the structural acceptance criteria the applicant establishes by referencing the applicable codes, standards, and load combinations are acceptable.

B.2.6 Materials, Quality Control, and Special Construction Techniques

In FSAR Subsection 3H.6.7.1 and with a reference to Subsection 3H.6.4.4, the applicant provides information regarding the structural materials used in the design of the DGFOSV. No special construction techniques are proposed or described in the FSAR, and there is no explicit mention of quality control programs in the FSAR. SRP Section 3.8.4 provides specific guidance for applicants to provide information about:

(a) the concrete ingredients and reinforcing bar splices;

(b) the nondestructive examination of the materials to determine physical properties, the placement of concrete, and erection tolerances;

(c) the extent to which the materials and quality control programs comply with ACI 349, with additional criteria provided by RG 1.142 for concrete and ANSI/AISC N690-1994 including Supplement 2 (2004) for steel, as applicable; and

(d) welding of reinforcing bars if proposed, to comply with ASME Boiler and Pressure Vessel Code (Code) Section III, Division 2.

RG 1.206 further specifies for quality control in general, verifying the extent of compliance with applicable provisions of SRP Sections 3.8 and 17.5 and recommendations of RG 1.55. The staff asked the applicant to incorporate the above information into the FSAR.

In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant provides a markup of FSAR Subsection 3H.6.7.5, "Materials and Quality Control," which references FSAR Subsection 3H.6.4.4.7 that provides the requested information and states that concrete ingredients, reinforcing bar splices, nondestructive examination of the materials to determine physical properties, placement of concrete, and erection tolerances will meet the requirements of ACI 349, supplemented by the RGs, codes, and standards referenced in the DCD and the FSAR. The applicant's response also states that materials and quality control programs comply with ACI 349, with additional criteria provided by RG 1.142 for concrete and ANSI/AISC N690-1994 including Supplement 2 (2004) for steel. The staff found the applicant's response and FSAR update acceptable, because the proposed material and quality control provisions are in accordance with the codes and standards accepted for use and all requested information is included in the FSAR markup. Therefore, this issue is resolved. Based on the above discussions, the staff concluded that the applicant's information pertaining to materials and quality control meets the guidance in SRP Section 3.8.4 and is therefore acceptable. The applicant incorporated the proposed markups into the FSAR Revision 8.

B.2.7 Testing and In-Service Requirements

According to SRP Section 3.8.4 and RG 1.206, the applicant should specify any testing and ISI requirements applicable to critical areas of the DGFOSV to meet the requirements of 10 CFR 50.65 and the guidance in RG 1.160. However, it was not clear how the ISI of normally inaccessible, below-grade concrete walls and foundations are performed. Therefore, the staff asked the applicant to confirm how the ISI requirements are met for below-grade concrete walls and foundations to (a) examine for signs of degradation of the exterior, exposed portions of below-grade concrete when excavated for any reason; and (b) conduct periodic site monitoring of ground water chemistry to confirm that the ground water remains nonaggressive. The applicant in the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), provides a markup of FSAR Subsection 3H.6.7.4, "Testing and ISI Requirements," referencing FSAR Subsection 3H.6.4.4.6 which states that site-specific seismic Category I structures are included in the scope of the DRAP and per FSAR Section 17.6S1.1b, all SSCs identified as risk-significant via the DRAP are included within the initial maintenance rule scope. Therefore, these site-specific seismic Category I structures are included in the Maintenance Rule Program, which includes monitoring and maintaining the requirements for the structural materials used in the design of the site-specific seismic Category I structures that will be implemented in accordance with 10 CFR 50.65 and RG 1.160, as described in FSAR Section 17.6S and Table 13.4S-1. The applicant also states that periodic monitoring of ground water chemistry is described in FSAR Subsection 2.4S.12.4. The applicant also provides a markup of new FSAR Subsection 3H.6.4.4.6 with the above information. The staff's review found that the applicant has appropriately addressed the issue of testing and ISI of the DGFOSV, including the below-grade concrete walls and foundation, by including them in the Maintenance Rule Program described above. The applicant incorporated the proposed updates into the FSAR Revision 8.

Based on the above discussions, the staff concluded that the applicant has provided adequate design information in the FSAR for the DGFOSV to meet the acceptance criteria in SRP Section 3.8.4 and has responded to all staff requests for additional information and clarification. Therefore, the design information included in the FSAR for the DGFOSV is acceptable. Therefore, **RAI 03.08.04-30, Item 8**, is resolved. The applicant incorporated the proposed markups into the FSAR Revision 8.

B.3 Design Information for the DGFOT

During the FSAR review, the staff found references to the DGFOT in several RAI responses. **RAI 03.08.04-30,** Item 10 asked the applicant to confirm the DGFOT as a seismic Category I structure and to include the analysis and design information to show how the design of the DGFOT meets the SRP Acceptance Criteria 1 through 7 in the SRP Sections 3.8.4 and 3.8.5. In the Revision 1 response to **RAI 03.08.04-30**, Item 10 dated March 15, 2011 (ML110770440), the applicant provides analysis and design information for the DGFOT that was not previously included in the FSAR. The response included a markup of the new FSAR Section 3H.7, "Diesel Generator Fuel Oil Tunnel (DGFOT)," which provides a general description of the three DGFOT that includes their size, location, analysis and design methods, finite element models, materials, SSI analysis, seismic wave propagation effects, stability evaluation, applicable codes and loadings, and lateral earth pressure diagrams. The response also includes descriptions of the wall, slab, and foundation design; soil springs; the uplift analysis; tables and figures reporting section forces; governing load combinations; and resulting concrete bending and shear reinforcements and layouts. Thereafter, the applicant provides additional analysis and design information in multiple responses to address various requests from the staff for additional information and clarification. The following RAI responses are sorted in chronological order and include additional information and clarification for the DGFOT and any FSAR markups provided with the responses that were subsequently incorporated into FSAR Revision 7: Supplement 1 to **RAI 03.08.04-17** (ML11730067), Revision 1 to **RAI 03.08.04-30,** Supplements 2 and 4 to **RAI 03.07.01-27** (ML11143A054 and ML11192A043), Supplement 5 to **RAI 03.08.04-30** (ML11196A040), and Supplement 1 to **RAI 03.08.04-34** (ML11259A056). Subsequently, the applicant included additional information and clarifications in the following responses for which the included FSAR markups are pending the FSAR update: Supplements 1 and 2 to **RAI 02.03.01-24**, Supplements 3 and 4 to **RAI 03.07.01-29**, and Supplements 4, 5, and 6 to **RAI 03.07.02-13**. The applicant incorporated the proposed markups into the FSAR Revision 8.

The staff's review of the analysis and the design information for the DGFOT against the SRP Section 3.8.4 acceptance criteria are discussed below.

B.3.1 Description of the Structure

In FSAR Section 3H.7.3, the applicant provides a general description of the three DGFOT pertaining to each proposed unit at the STP site. Figure 3H.6-221 shows a partial site plan with the locations of the three DGFOT relative to other structures. The DGFOT are described as reinforced concrete tunnels approximately 15.24, 61, and 67 m (50, 200, and 220 ft). Each DGFOT is connected at one end to the RB and at the other end to a DGFOSV. Each DGFOT has two access regions that extend above grade. The overall above-grade dimensions of the access regions are approximately 2.29 m (7.5 ft) wide by 2.29 m (7.5 ft) long and 4.57 m (15 ft) high. The top of the DGFOT is located approximately at grade. DGFOT No. 1B is the shortest tunnel that runs approximately 15.24 m (50 ft) between the RB and DGFOSV No. 1B, and has a wall thickness of 61 cm (24 in.) on both sides. The interior below grade dimensions of this tunnel are approximately 2.13 m (7 ft) high by 1.07 m (3.5 ft) wide. The other two longer DGFOT (approximately 61 m and 67 m [200 ft and 220 ft] long) have a wall thickness of 61 cm (24 in.) on one side and 76.2 cm (30 in.) on the other side to allow for the placement of embedded conduits. The interior below grade dimensions of these tunnels are approximately 2.13 m (7 ft) high by 0.91 m (3 ft) wide. The staff noted that the description did not include plans and sections of the DGFOT with sufficient information to define the primary structural aspects and elements, as specified in SRP Section 3.8.4. The staff therefore asked the applicant to include in the FSAR plans and sections with sufficient information about the primary structural layout and elements. In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant includes FSAR Figure 3H.7-36, which provides a section of the DGFOT with dimensions of the major structural elements. Design results for the various structural elements are in Table 3H.7-1. Designations of the various reinforcing zones are shown in Figures 3H.7-9 through 19A. The staff reviewed the applicant's information included in the FSAR for the DGFOT structure and determined that the information meets the guidance in SRP Section 3.8.4 and is acceptable. Therefore, this issue is resolved. The applicant incorporated the proposed markups into the FSAR Revision 8.

B.3.2 Applicable Codes, Standards and Specifications

In FSAR Subsection 3H.7.4.1 and Section 1.8, the applicant provides information regarding the applicable design codes, standards, and specifications. The DGFOT is considered part of the standard plant structures, and it is designed using the codes and standards for standard plant structures. The use of these codes and standards was reviewed in NUREG–1503. Departures pertaining to the use of these codes, standards, and specifications for standard plant structures were already evaluated earlier in the SER. Therefore, the staff concluded that referencing these codes, standards, and specifications for the analysis and the design of the DGFOT structure is acceptable.

B.3.3 Loads and Load Combinations

In FSAR Subsections 3H.7.4.2 and 3H.7.4.3, the applicant provides information regarding the applicable loads and load combinations used for the design of the DGFOT. FSAR Table 3H.9-1 provides a summary of the extreme environmental loads used in the design. DCD design loads in combination with site-specific loads are considered in the design and as a minimum the DGFOT is designed for the site-specific loads. Load combinations used for the design are in accordance with SRP Section 3.8.4, ACI 349–97, and RG 1.142. Based on the above information, the staff concluded that the loads and load combinations used for the DGFOT design are acceptable.

B.3.4 Design and Analysis Procedures

In FSAR Subsection 3H.7.5.1, the applicant provides information regarding the structural design and analysis of the DGFOT. During the March 2011 and May 2011 audits (ML111320094 and ML12346A389), the staff reviewed the analysis and design of the DGFOT calculation entitled, "Basic Structural Design of Diesel Generator Fuel Oil Tunnels (DGFOT)." The staff's review is summarized below.

Only the DGFOT No.1B was analyzed. It is the shortest of the three tunnels and has a wall thickness of 61 cm (24 in.) on both sides. This tunnel was considered the most critical design, enveloping the other two tunnels because of the dimensions of the tunnel cross section and shorter tunnel length for resisting torsion effects. The structure is modeled using twelve 3D-SAP models (10 static and 2 dynamic models; see FSAR Figure 3H.7-1), shell elements, and soil springs. The different models are used to account for the effects of the basemat uplift and static and dynamic soil springs, as well as to consider the effect of uniformly distributed soil spring stiffness representing the Winkler uncoupled case, and non-uniformly distributed spring stiffness representing the pseudo-coupled spring case, which is used to better simulate the coupling of springs under the base slab of the tunnel. A more detailed description of the methodology regarding the determination of uplift effects, soil springs, and application of equivalent static accelerations is included in the section describing the DGFOSV. The equivalent absolute accelerations to be used as seismic excitation in the SAP model are taken from the SSI analyses of the tunnel. The accelerations are converted into inertial loads and are applied as distributed area loads and nodal point loads in x-, y-, and z- directions. Co-directional responses are subsequently combined with the SRSS rule. In order to verify the seismic results obtained from the equivalent static analysis with the SAP, a comparison is made between the resulting base shear and total vertical force from the SAP model, with the same quantities obtained from the SASSI model. If the SAP base shear values are less than the corresponding SASSI values, adjustment factors are determined to amplify the equivalent accelerations used in the SAP model, in order to match the vertical and/or horizontal base shears of both models. This case matched SSI and SAP results, and equivalent accelerations were amplified by a factor of 1.15. The final adjusted accelerations are as follows:

	Horizontal	Vertical
Below grade nodes	0.45g	0.37g
Above grade nodes	0.85g	0.40g

Accidental torsional effects are considered in accordance with ASCE 4–98, which requires an additional torsional moment to be applied to the structure. Regarding tornado missiles, the minimum required wall thickness to prevent penetration was calculated to be 38.1 cm (15 in.), which is less than the provided wall thickness of 61 cm (24 in.). The global missile impact effects resulted in added torsional reinforcement in the tunnel section. To maintain stability, a restraint is required around the access shafts to prevent movement from a horizontal missile hit. The supports will be designed during the detailed design phase. Seismic wave propagation effects, which induce additional stresses in the tunnel due to soil deformation during wave passage, are considered by applying the guidance in ASCE 4–98, Subsection 3.5.2.1. Site-specific friction coefficients are used to determine seismic wave propagation effects. The resultant total strain was computed to be about 8.5 percent of the rebar yield strain and was considered in the design. The axial force and moment and shear forces induced in bends are also considered in the design by applying these forces to the entire tunnel being treated as a beam. Maximum lateral pressure at bends was limited to full passive lateral soil pressure.

The staff’s review found the design and analysis procedure for the DGFOT to be technically acceptable, because it follows standard engineering methods for consideration of the various design loads in the analysis and the design. The staff, however, needed some additional clarification regarding the evaluation of seismic wave propagation effects and a description of lateral soil pressures, as described below.

a. Seismic Wave Propagation Effects on DGFOT

Seismic wave propagation effects on the DGFOT were considered using the ASCE 4–98 methodology. This is an accepted industry practice, and its application in designing the DGFOT is considered adequate. However, it was noted that Rayleigh waves, with an apparent wave velocity of 2,012 m/s (6600 ft/s), were considered in the design, although 914.4 m/s (3,000 ft/s) was specified in the FSAR. Also, it was noted that the effect of additional lateral pressure due to seismic wave propagation on the tunnel wall panels spanning between the top and bottom slabs of the tunnel at the bend was not considered. This issue was discussed with the applicant during the audit in May, 2011 (ML12346A389), and the applicant agreed to revise the calculations for the tunnels and to address the issue as follows:

- Use an apparent wave velocity of 914.4 m/s (3,000 ft/s) per Subsection C3.5.2.1 of ASCE 4–98.

- Consider the maximum ground velocity based on site-specific SSE maximum ground acceleration of 0.13g.

- Consider a triangular pressure distribution on the transverse leg of the tunnel near the bend, which is limited by the maximum passive pressure represented by a passive pressure coefficient of Kp = 3.

- Revise the FSAR to reflect these changes.

In the Supplement 5 response to **RAI 03.08.04-30** (ML11196A040), the applicant confirms that the calculations of the axial tensile strains, forces, and moments at the tunnel bends and soil pressures on the transverse leg of the tunnels near the bends due to seismic wave propagation for the DGFOT were revised using the parameters given above with a maximum ground velocity of 15.85 cm/s (6.24 in./s) (which is based on 121.9 cm/s/g [48 in/s/g] and the site-specific SSE maximum ground acceleration of 0.13g). The corresponding markups to revise FSAR Subsection 3H.7.5.2.4, "SSE Wave Propagation Effects," resulting from the resolution of this item were also provided with this response. The staff's evaluation noted that the site-specific SSE criteria were used in the design of the DGFOT, which is considered a standard plant structure. In the Supplement 7 response to **RAI 03.08.04-30** (ML11322A106), the applicant acknowledges that since the layout of the DGFOT is site-specific, the seismic wave propagation is also based on the site-specific SSE. Because the analysis meets the criteria in ASCE 4–98, considers the passive pressures, and incorporates the changes in FSAR, the staff found the response adequate and the issue resolved. FSAR, Section 3H.7, was subsequently updated to incorporate the proposed changes provided with the response.

b. Lateral Soil Pressures on the DGFOT

Complementing the initial information regarding soil pressures in the Revision 1 response to **RAI 03.08.04-30** Item 10 (ML110770440), the applicant later, in the Supplement 1 response to **RAI 03.08.04-17** (ML110730067), provides a markup to FSAR, Section 3H.7 for the DGFOT, which includes the numerical soil parameters used in the design; and the at-rest, dynamic at-rest, active, and passive soil pressure profiles without the SSSI effects. The SSSI incremental seismic soil pressure diagrams, which include the effects of the nearby structures (RB, crane wall, and DGFOSV No.1) on the DGFOT, were included in the Revision 1 response to **RAI 03.08.04-30** (ML110770440). The tunnel walls are analyzed for load combinations including static and dynamic loads and involve three different soil pressures—active (Ka = 0.33), at-rest (Ko = 0.50), and passive (Kp = 3.0). The resulting load components are shown in several lateral pressure diagrams as follows: (a) dynamic at-rest pressures (Figure 3H.7-2); (b) static active, at-rest, and passive pressures (Figures 3H.7-33 through -35); (c) seismic pressures obtained from the SSI, SSSI, and ASCE 4–98 analyses (Figures 3H.7-5 through -8); and (d) driving and resisting lateral pressures used for stability analyses (Figures 3H.7-3 and -4). A surcharge load of 23.94 kPa (500 psf) as well as hydrostatic and hydrodynamic loads are also included in the corresponding load combinations. Included in Figures 3H.7-5 through -8 is the dynamic soil pressure used for the design, which envelopes the dynamic pressures obtained from the SSI, SSSI, and ASCE 4–98 analyses. The staff's evaluation concluded that the applicant has appropriately considered the static lateral soil pressures for the design of the DGFOT walls by using the earth pressures resulting from the two possible conditions: the yielding and non-yielding wall. The staff also considered computing the dynamic lateral soil pressure as the envelope of the SSI, SSSI, and ASCE 4–98 dynamic lateral earth pressures that provide a conservative estimate of the expected lateral dynamic soil pressure meets the intent of the guidance in SRP Section 3.8.4. The method used by the applicant to determine the driving and resisting earth pressures for the stability evaluation is discussed along with the discussion of stability evaluations in Subsection 3.8.4.4.2 of this SER. Based on the above information, the staff concluded that the lateral soil pressures used by the applicant for the design of the DGFOT are acceptable.

B.3.5 Structural Acceptance Criteria

In FSAR Subsection 3H.7.4.1, the applicant provides information regarding the applicable design codes and standards for the DGFOT. The loading combinations in Subsection 3H.7.4.3.4.2 include the structural acceptance criteria, which correspond to the above design codes and standards. By referencing these codes and standards and including the edition, revision, and date of issuance, and using the loading combinations according to these codes and standards, the applicant has specified the structural acceptance criteria related to stresses, strains, gross deformations, factors of safety, and other parameters that quantitatively identify the margins of safety. Therefore, staff concluded that the applicant's structural acceptance criteria established by referencing the applicable codes, standards, and load combinations are acceptable.

B.3.6 Materials, Quality Control and Special Construction Techniques

In FSAR Subsection 3H.7.4.4, the applicant provides information regarding the structural materials used in the design of the DGFOT. No special construction techniques are proposed or described in the FSAR and no explicit mention of quality control programs is included in the FSAR. SRP Section 3.8.4 provides specific guidance for applicants to provide information about the major materials used and the quality control parameters. For quality control, RG 1.206 specifies, in general, verifying the extent of compliance with the applicable provisions of SRP Sections 3.8 and 17.5 and recommendations of RG 1.55, "Concrete Placement in Category I Structures." Therefore, the applicant was asked to incorporate the above information into the FSAR.

In the Supplement 5 response to **RAI 03.07.02-13** (ML12103A369), the applicant provides a markup of FSAR Subsection 3H.7.4.4.5, "Materials and Quality Control," which references FSAR Subsection 3H.6.4.4.7 that provides the requested information and states that concrete ingredients, reinforcing bar splices, nondestructive examination of the materials to determine physical properties, placement of concrete, and erection tolerances will meet the requirements of ACI 349, supplemented by the RGs, codes, and standards referenced in the DCD and the FSAR. The response also states that materials and quality control programs comply with ACI 349, with additional criteria provided by RG 1.142 for concrete; and ANSI/AISC N690–1994 including Supplement 2 (2004) for steel. The staff found the applicant's response and the FSAR update acceptable because the proposed material and quality control provisions are according to the codes and standards accepted for use, and all requested information is included in the FSAR markup. The applicant incorporated the proposed update into FSAR Revision 8. Therefore, this issue is resolved. Based on the above discussions, the staff concluded that the applicant's information pertaining to materials and quality control meets the guidance in SRP Section 3.8.4, and is therefore acceptable.

B.3.7 Testing and In-service Requirements

According to SRP Section 3.8.4 and RG 1.206, the applicant should specify any testing and ISI requirements applicable to critical areas of the DGFOT to meet the requirements of 10 CFR 50.65 and the guidance in RG 1.160. However, it was not clear how the ISI of normally inaccessible, below-grade concrete walls and foundations are performed. Therefore, the staff asked the applicant to clarify how the ISI requirements are accomplished for below-grade concrete walls and foundations to (a) examine for signs of degradation of the exterior, exposed portions of below-grade concrete when excavated for any reason; and (b) conduct periodic site monitoring of the ground water chemistry to confirm that the ground water remains nonaggressive.

The applicant's Supplement 5 response to **RAI 03.07.02-13** (ML12103A369) provides a markup of FSAR Subsection 3H.7.4.4.4, "Testing and ISI Requirements," which references FSAR Subsection 3H.6.4.4.6 that states the site-specific seismic Category I structures are included in the scope of the DRAP and per FSAR Section 17.6S1.1b, all SSCs identified as risk-significant via the DRAP are included within the initial maintenance rule scope. Therefore, these site-specific seismic Category I structures are included in the Maintenance Rule Program, which includes monitoring and maintaining requirements for the structural materials used in the design of the site-specific seismic Category I structures that will be implemented in accordance with 10 CFR 50.65 and RG 1.160, as described in FSAR Section 17.6S and Table 13.4S-1. The applicant also states that periodic monitoring of the ground water chemistry is described in FSAR Subsection 2.4S.12.4. In addition, the applicant provides a markup of the new FSAR Subsection 3H.6.4.4.6 with the above information. The staff found that the applicant has appropriately addressed the issue of testing and ISI of the DGFOT, including the below-grade concrete walls and foundation by including them in the Maintenance Rule Program described above. The applicant incorporated the proposed changes into the FSAR Revision 8.

Based on the above discussions, the staff concluded that the applicant has provided adequate design information in the FSAR for the DGFOT to meet the acceptance criteria in SRP Section 3.8.4 and has responded to all of the staff's requests for additional information and clarification. Therefore, the design information included in the FSAR for the DGFOT is acceptable. Therefore, **RAI 03.08.04-30, Item 10** is resolved.

B.4 *Design Information of the Interfaces Between the RSW Tunnels, DGFOT, and Adjoining Seismic Category I Structures*

In FSAR Subsection 3H.6.3.4, "Reactor Service Water Piping Tunnels," the applicant states that the interfaces between the tunnels and the pump house and the CB are configured to allow relative movement between the tunnels and structures. But the applicant did not provide a description of the interface configuration between the tunnels and the pump house and the CB, or a description of the analysis and the design methodology. In **RAI 03.08.04-15** and follow up

RAIs 03.08.04-25 and **03.08.04-31**, the staff asked the applicant to provide such descriptions and to include the relevant information in the FSAR, specifically to clarify the items listed below.

a. *Design Loads and Deformations*

In **RAI 03.08.04-15,** the staff asked the applicant to describe the interface configuration between the tunnel and the pump house and the CB and to include the analysis and the design methodology for the interface, loads, and load combinations that were used; the amount of relative movement considered in the design along with the technical basis, and to demonstrate that the flexible connection used at the interface is adequate for the design loads and deformations. In the October 5, 2009, response to **RAI 03.08.04-15** (ML092810321), the applicant provides a conceptual detail of the interface between the RSW piping tunnels and the RSW pump houses and the CB structures. The applicant states that the detail allows the flexibility to accommodate the relative movements between the buildings and the tunnels. The gap between the tunnels and the buildings is specified to accommodate the calculated relative movements due to seismic displacements and differential settlement. The applicant also states that the interfaces will be designed and finalized during the detailed design phase considering the applicable loads and loading combinations described in COL FSAR Subsection 3H.6.4.3.

The staff noted that the response did not include any information regarding size, dimension, and material for the interface or calculated data to support the displacement capacity requirement of the joint. Therefore, **RAI 03.08.04-25** asked the applicant to provide detailed information to demonstrate that the design joint has enough deformation capacity to accommodate the deformation demand that is obtained from analysis to confirm that the tunnel interface will maintain integrity, and confirm that loads due to interaction of the tunnel and the building are appropriately included in the design. The applicant was also requested to include in the FSAR critical design information pertaining to the design of the interface (e.g., separation gap, calculated differential displacement, material and stiffness properties of the interface material, etc.). In addition, the applicant was asked to address the potential degradation of the interface material due to groundwater, in-service inspection of the interface material, and measures against potential in-leakage of groundwater.

In the Revision 2 response to **RAI 03.08.04-25** dated May 13, 2010 (ML101340651), the applicant states that the joint is designed to accommodate the expected relative building movements without transmitting significant forces. The joint material will be polyurethane foam impregnated with a waterproof sealing compound or a similar material. The separation gaps between the RSW tunnel and the DGFOT and the adjoining pump house, CB, or the DGFOSV will be at least 50 percent larger than the absolute sum of the calculated displacements due to seismic and long-term settlement. The applicant adds that to minimize the movements due to settlement, the complete installation of the details will not occur until after the short term settlement is substantially complete. Considering the negligible strength and limited area of the sealing material compared to strength (minimum compressive strength, f'c, of 27.58 MPa [4,000 psi]) and massive size of the tunnels and abutting structures, the effect on the interaction between structures, if any, was negligible. The response also states that the material used as flexible filler will be able to be compressed to approximately one-third of its thickness (based on a 50 percent margin or a commensurate value if a margin larger than 50 percent is provided) without subjecting the building to more than a negligible force relative to the resistance capacity of the building. Furthermore, typical vendor data indicate that the material tensile strength is about 144.8 kPa (21 psi) and that vendor testing for this material in a 12.7 cm (5 in.) joint compressed to 50 percent movement has a 48.27 kPa (7 psi) compressive stress in the compressed condition. The values for the required and provided separation gaps due to seismic movements plus long-term settlement are in Table 3H.6-15 of the FSAR. The response also includes a markup of the new FSAR Section 3H.6.8, “Seismic Gaps at the Interface of Site-Specific seismic Category I Structures and the Adjoining Structures,” providing a description of the interface.

The staff reviewed the response and noted that it stated that the joint material is designed to be compressed to one-third of its thickness, and the applicant provided vendor test results where the 48.27 kPa (7 psi) compressive stress was observed when 12.7 cm (5 in.) thick joint was compressed to 50 percent of its thickness. Because this information did not provide an estimate of how much compressive stress may be developed when the material is compressed to one-third of its thickness, in **RAI 03.08.04-31** the staff asked the applicant to justify why no significant stress will be imparted to the adjoining building when the joint is compressed to a one-third thickness.

In the response to **RAI 03.08.04-31** September 15, 2010 (ML102630145), the applicant states that the actual material for seals has not been selected, nor has it been tested for the compressive stress applied when it is compressed to one-third of the original joint size. However, based on typical vendor data that show the compressive stress of the joint filler when the joint is expanded and contracted by 50 percent, the relationship between the joint size and the compressive stress appears to be approximately linear throughout the compression zone. The response includes a graph showing that the compressive stress is approximately 11 kPa (1.6 psi) when installed in a 12.7 cm (5 in.) nominal joint. This stress decreases to 3.45 kPa (0.5 psi) when the joint expands to approximately 150 percent of the original size (i.e., 18.42 cm [7.25 in.]), and increases to 44.82 kPa (6.5 psi) when the joint contracts to 50 percent of the original size (i.e., 6.35 cm [2.5 in.]). Therefore, there is sufficient confidence that the compressive stress will be less than 172.38 kPa (25 psi) when compressed to one-third of the original joint size. In FSAR Revision 6, the COL application has been updated to require the maximum compressive stress of the material to be less than 172.38 kPa (25 psi) when subjected to the maximum static and dynamic differential displacements of the joints. Based on ACI 349-97 Section 10.15, the bearing capacity of 27.58 MPa (4,000 psi) concrete is 16.41 MPa (2380 psi), which is significantly higher than the maximum pressure of 172.38 kPa (25 psi) that may be applied at the seismic joint. Therefore, local effects of this load are considered negligible. The structures experiencing the load from the seal material are either loaded in-plane (e.g., RSW piping tunnels) or out-of-plane (e.g., RSW pump house walls). For structures loaded in-plane, the axial capacity of the concrete section is more than 12.41 MPa (1,800 psi) based on ACI 349–97, Subsection 10.3.5.2. This is significantly higher than the 172.38 kPa (25 psi) load that may be exerted by the filler material at the joint locations. Embedded concrete walls loaded out-of-plane by the seal material have also been designed for a minimum 103.43 kPa (15 psi) soil pressure load during seismic events. Because the area where the filler material will be placed is very small in comparison to the area of the wall loaded by static and dynamic soil pressure, the pressure exerted by the filler material is insignificant compared to the total applied soil pressure. Therefore, global effects on the walls are considered negligible. The staff agreed that limiting the contact pressure to 172.38 kPa (25 psi) is reasonable and does not represent a significant additional load demand compared with other design loadings.

b. *Movements at the DGFOT*

The FSAR Table 3H.6-15 provided with the Revision 2 response to **RAI 03.08.04-25** (ML101340651) did not include the required and provided gaps at the end of the DGFOT and away from the DGFOSV. Therefore, in **RAI 03.08.04-31**, the staff asked the applicant to provide the anticipated movements at both end connections of the DGFOT. In the Revision 1 to the Supplement 1 response to **RAI 03.08.04-31** dated January 31, 2011 (ML110330168), the applicant states that the layout of the DGFOT is as shown in COL FSAR Figure 3H.6-221. There are three DGFOTs for each unit and each DGFOT is connected at one end to the RB and at the other end to a DGFOSV. There is a seismic gap between each of the DGFOT and the adjoining RB and DGFOSV. COL FSAR Table 3H.6-15 will be revised to include the required and provided gaps for the DGFOT. The applicant provided FSAR mark-up of revised Table 3H.6-15 with the response. This revised table also incorporates changes from the revised SSI analysis for the RSW piping tunnels and the revised SSI analysis for the DGFOSV. The staff found that the applicant's response addresses the issue because the gap is designed to accommodate the estimated differential movements and the values were incorporated into FSAR Revision 6. Therefore, this issue is resolved.

c. Potential Material Degradation

In **RAI 03.08.04-25,** the staff asked the applicant to provide information regarding the potential degradation of the interface material due to groundwater, the ISI of the interface material, and the measures against potential in-leakage of groundwater. In the Revision 2 response to **RAI 03.08.04-25** (ML101340651), the applicant states that because of the low rate with which groundwater can flow through the detail if it were to fail in any particular location, in-leakage of groundwater is a housekeeping issue and not a safety concern. Even a degraded flexible filler material acts as a sieve to slow the flow of groundwater into the building/tunnel. Constant exposure to groundwater may deteriorate the waterproofing material. However, the provided joint detail allows the waterproofing material to be inspected or to be replaced if it becomes degraded. The staff agreed with this response because the potential seepage through the gap will be limited, and periodic maintenance inspections will anticipate repair needs ahead of any significant in-leakage event. Therefore, this issue is resolved.

d. ITAAC Table with Key Parameters

In **RAI 03.08.04-31**, the staff asked the applicant to provide an ITAAC with key parameters for as-built verification of the connections. In the response to **RAI 03.08.04-31** dated September 15, 2010 (ML102630145), the applicant states that the COL application will be updated to state that maximum compressive stress of the material will be less than 172.38 kPa (25 psi) when subjected to the maximum static and dynamic differential displacements of the joints. Because an appropriate material will be selected to meet this COL application requirement, there will be no significant stress imparted to the building when the joint is compressed to a one-third thickness, an additional site-specific ITAAC is not required. The staff agreed that since FSAR Table 3H.6-15 provides the required gaps for the seismic joints, and the FSAR includes the criterion for maximum compressive load on the sealing material when compressed to a one-third thickness, sufficient control for the design information is included in the FSAR. Therefore, this issue is resolved.

As described above, the applicant's response regarding the RSW piping tunnels and the DGFOT interfaces with adjoining seismic Category I structures adequately addresses the staff's requests for additional information and clarification in **RAI 03.8.04-15** and follow-up **RAI 03.08.04-25** and **RAI 03.08.04-31**, and included necessary information about the interface in the FSAR. Therefore, **RAI 03.08.04-15, RAI 03.08.04-25,** and **RAI 03.08.04-31** are considered closed. FSAR Sections 3H.6.8 (new) and Table 3H.6-15 (new) were subsequently updated to incorporate the proposed changes provided with the response to RAI 03.08.04-31.

3.8.4.4.3 Design Information for Non-Seismic II/I Structures

ABWR DCD Subsection 3.7.2.8, which is incorporated by reference into the COL FSAR, addresses the interaction of non-seismic Category I structures with seismic Category I structures. ABWR DCD specifies three criteria, and all non-seismic Category I structures must meet one of these criteria. The staff required additional information to determine the implementation of these criteria for non-seismic Category I structures with the potential to interact with seismic Category I structures. In **RAI 03.07.02-1** and subsequently in **RAI 03.07.02-13,** the staff asked the applicant to provide additional information including the stability evaluation of these structures. The staff's assessment of these RAIs is discussed in SER Subsection 3.7.2.4.8.

The staff noted that additional information regarding the design and stability evaluation of non-Category I structures was needed to complete its evaluation. Subsequently, **RAI 03.07.02-32** asked the applicant to describe in the FSAR the following:

- Clearly describe in the FSAR the criterion used to determine that the collapse of a non-Category I structure will not cause the non-Category I structure to strike a Category I structure. Also, clarify in the FSAR that non-Category I structures that are not identified in the FSAR as structures that can interact with Category I structures meet this criterion.

- Describe in the FSAR the analysis and design of each non-Category I structure that can interact with Category I structures to demonstrate that the structure is analyzed and designed not to fail under SSE conditions in a manner that the margin of safety of the structure is equivalent to that of seismic Category I structures. Also, include site-specific ITAAC for each structure to confirm that the as-built structure is analyzed and designed as described in the FSAR.

- For each non-Category I structure, describe in the FSAR the stability evaluation procedure, including how seismic demand and restoring forces for the stability evaluation are determined.

In the Revision 1 response to **RAI 03.07.02-32** dated December 19, 2011 (ML11364A098), the applicant includes proposed changes to FSAR Subsections 3.7.2.8 and 3.7.3.16 to address the above questions from the staff. The staff's review of the applicant's response is discussed below.

A. Criterion for Determining Interaction with Category I Structure

In the response the applicant states that a specific criterion will be added in COL FSAR Subsection 3.7.2.8 that if the above-grade height of the non-Category I structure is less than the shortest horizontal distance between the non-Category I structure and the closest Category I structure, collapse of the non-Category I structure will not cause the non-Category I structure to strike a Category I structure. The COL application will also be revised to reflect that non-Category I structures that are not identified in the FSAR as structures that can interact with Category I structures will meet this criterion. The applicant also includes in FSAR Subsection 3.7.2.8 the non-Category I structures that may potentially interact with seismic

Category I structures based on this criterion. The following structures are included in this category:

a. TB
b. SB
c. RWB
d. CBA
e. The plant stack on the RB roof

The staff's review concluded that the applicant's proposed criterion for potential interaction of non-seismic Category I structures with seismic Category I structures is reasonably conservative in the absence of a more explicit analysis. The staff reviewed the proposed FSAR changes included in the response and confirmed that the applicant has appropriately included the criterion for determining the potential interaction and identify all non-seismic Category I structures which may potentially interact with seismic Category I structures. The applicant incorporated the proposed changes into FSAR Revision 8.

In the response to RAI 03.07.02-13 dated February 10, 2010 (ML100550613), the applicant states that the plant stack located on the RB roof is an integral part of the RB roof and positively anchored to the roof. The stack and its anchorage to the RB roof are designed to withstand all applicable loads including the SSE. Based on this information, the staff concluded that the design of the stack is part of the DCD, and its stability evaluation is not necessary**.**

B. Design of Non-Category I Structures

The applicant has not performed a detailed structural design of non-Category I structures. In the Revision 1 response to **RAI 03.07.02-32** dated December 19, 2011 (ML11364A098), the applicant provides a markup of FSAR Subsection 3.7.2.8, which includes design criteria for the above four non-Category I structures. This section states that for the overall design of non-seismic Category I structures, the procedures described in the IBC under seismic design criteria will be applied. However, the lateral load resisting system will be designed to remain elastic under the extreme environmental loads shown in Table 3H.9-1 using the same loads, load combinations, and design codes (i.e., ACI 349 and ANSI/AISC 690) as those for adjacent Category I structure. In the markup of FSAR Subsection 3.7.3.16 attached to this response, the applicant states that the lateral load resisting system consisting of structural elements required for the transfer of lateral loads to the foundation, in addition to meeting the IBC requirements, shall be capable of resisting the entire lateral load assuming that all components of the non-Category I structure, with the exception of siding that may come off during a tornado event, will remain intact during the extreme environmental loading. It also states that the exterior walls of the non-Category I structure that is adjacent to the Category I structure shall be capable of resisting SSE loads using the same loads, load combinations, and design codes as those for the adjacent Category I structure, including those elements of the non-Category I structure that may be located within a story height of the exterior wall, or that may come in contact with the exterior wall upon its failure under SSE loads. The design loads and load combinations specified in Table 3H.9-1, "Extreme Environmental Design Parameters for Seismic Analysis, Design, Stability Evaluation and Seismic Category II/I Design," include seismic, tornado, tornado missile, and flood loading.

The staff reviewed the loads specified in Table 3H.9-1, and found them to be the same or more conservative than the corresponding loads used for site-specific Category I structures. Based on a review of the above design criteria, which specify that the non-Category I structures will be designed using the IBC and that the lateral load resisting system consisting of structural elements required for transfer of lateral loads to the foundation shall be capable of resisting the entire lateral load due to the extreme environmental loads per Table 3H.9-1, the staff concluded that there is reasonable assurance that the design of non-Category I structures will meet the SRP Section 3.7.2, SRP Acceptance Criterion 8(C) and have a margin of safety equivalent to that of a Category I structure against a failure due to extreme environmental loads. The applicant also includes site-specific ITAAC for the non-Category I structures in Tables 3.0-21 through 3.0-24 to verify completion of design according to the criteria specified and as-built configuration of the four non-Category I structures described above. Based on the above, the staff concluded that design of non-Category I structures described in the FSAR is acceptable.

C. Stability Evaluation of Non-Category I Structures

In the responses to **RAI 03.07.02-13** dated February 10, 2010 (ML100550613)**,** Supplements 1, 2, and 3 responses to **RAI 03.07.02-13** dated April 29, 2010, April 25, and July 27, 2011, (ML101250162, ML11119A007, and ML11213A094, respectively), and Revision 1 response to **RAI 03.07.02-32** dated December 19, 2011 (ML11364A098), the applicant describes the stability evaluation method for non-Category I structures and provides a markup of FSAR Subsections 3.7.2.8 and 3.7.3.16 with a description of the method. In the response to **RAI 03.07.02-13**, the applicant includes a schematic diagram showing the various driving and resisting forces that are considered to be acting on the structure for the stability evaluation and the formulas used for determining sliding and overturning factors of safety. This figure was subsequently included as FSAR Figure 3H.3-52. The driving forces consist of seismic demand due to inertia of the structure in the horizontal direction and the static and dynamic soil pressure. The resisting forces are due to friction force at the base of the structure and passive soil pressure. Self weight excitation in the upward vertical direction and buoyancy force due to flooding are considered for stability evaluation. The seismic inertia force is determined either from an equivalent static method, or a RSA. The static and dynamic soil pressures include seismic loads from soil and hydrodynamic pressure from groundwater calculated in accordance with Subsection 2.5S.4.10.5. The stability evaluation method described by the applicant is similar to what was reviewed and accepted for seismic Category I structures. The only difference being the use of full dead weight (instead of 90 percent of the dead weight) of the structure as a stabilizing force for calculating safety factors against sliding and overturning. The applicant adds that this is acceptable per the guidance in SRP Section 3.8.5, SRP Acceptance Criterion 3. The staff discussed the subject of using the methodology described in Figure 3H.3-52 for determining the seismic demand for the stability evaluation with the applicant during the audit. The applicant subsequently demonstrated in the Supplement 4 response to **RAI 03.07.02-13** dated November 28, 2011 (ML11335A232), the acceptability of the methodology by comparing the evaluation per formulations shown in the figure with the seismic demand calculated using the SSSI analysis for the RWB. The seismic input motion used for the stability evaluation is based on the amplified site-specific SSE considering the effect of adjacent structures, except for the TB. Acceptability of the seismic input motion is discussed in Subsection 3.7.2.4.8 of this SER. Seismic demands along three orthogonal directions are combined using the 100-40-40 percent combination rule per RG 1.192, Revision 2, and is acceptable. The coefficient of friction assumed for the stability evaluation is based on shear resistance of the supporting foundation soil. Dynamic coefficient of friction (sliding) is used when passive soil resistance is needed to maintain stability. Properties of the foundation soil used for the stability evaluation are per Subsection 2.5S.4.2. The use of static and dynamic coefficient of friction is discussed in connection with the stability evaluation of seismic Category I structures and is therefore acceptable. The following discusses the stability evaluation for each non-Category I structure. Factors of safety against sliding, overturning, and floatation along with the coefficient of friction used for the sliding stability evaluation are in FSAR Table 3H.6-14.

a. Stability Evaluation of the Turbine Building

The staff reviewed the stability evaluation of the TB during the audit in May, 2011. The stability analysis is performed using the site-specific SSE of 0.13g. A coefficient of sliding friction of 0.3 was used per Table 2.5S.4-16 for stratum D soil, which was the lowest value of the sliding coefficient. Dynamic soil pressure was computed using ASCE 4–98, which provides at-rest dynamic soil pressure and is larger than the active dynamic soil pressure and is conservative. Passive pressure corresponding to Kp = 2.25 (less than 3.0 maximum) was used to calculate the resisting pressure. The floatation safety factor was calculated considering the design-basis flood elevation of 12.19 m (40 ft) MSL. A site-specific tornado wind with 321.8 km/h (200 mph) maximum velocity was used. Tornado wind pressure was reduced considering the dimension of the TB using the provisions of Topical Report BC-TOP-3-A, "Tornado and Extreme Wind Design Criteria for Nuclear Power Plants," issued August 1974. This Topical Report is approved for use by the NRC. The seismic demand is determined from the RSA of six fixed-base stick models. The mass and stiffness properties of the stick models are determined from a 3D model of the structure. For each orthogonal direction (i.e., E-W, N-S, and vertical), two fixed-base stick models are used; one representing the turbine generator pedestal and another representing the TB. The turbine generator pedestal shares a common basemat with the TB. For each orthogonal direction, the seismic demands from the two models are combined using absolute sum and are further increased to account for the base mass effect by exciting the base mass with the ZPA of the corresponding site-specific SSE input motion. The total seismic demand due to the three seismic excitations is determined by combining the seismic demand for each orthogonal direction using the 100-40-40 rule per RG 1.192, Revision 2. The calculated factors of safety against overturning, sliding, and flotation were found to exceed the minimum safety factors described in the SRP acceptance criteria. Based on the above review, the staff concluded that the stability evaluation of the TB performed by the applicant is acceptable.

b. Stability Evaluation of the Service Building

The staff reviewed the stability evaluation calculations for the SB during the audit in May, 2011. The staff needed clarifications regarding the seismic input motion and the seismic stick model used for the stability evaluation. In the Supplement 3 response to **RAI 03.07.02-13** (ML11213A094) and the Revision 1 response to **RAI 03.07.02-32** (ML11364A098), the applicant provides the clarifications and FSAR markup as discussed below. The stability evaluation of the SB is performed using the site-specific SSE amplified to account for the presence of the adjacent RB and CB shown in Figures 3.7-53 through 3.7-55. The seismic demand for the stability evaluation is determined from the RSA of two fixed-base stick models, one for each horizontal direction. The mass and stiffness properties for the stick models are determined in a manner similar to that described for the TB. For each horizontal direction, the seismic demand from the RSA is increased to account for the base mass effect. The increase is calculated by exciting the base mass using the ZPA of the corresponding amplified site-specific SSE input motion. The seismic demand for the vertical excitation is computed using the ZPA of the vertical amplified site-specific SSE input motion. The total seismic demand due to the three seismic excitations is determined by combining the seismic demand for each orthogonal direction using the 100-40-40 rule per RG 1.192, Revision 2. The FSAR markups provided with the above responses were subsequently incorporated into the FSAR. Based on the review, the staff found that the applicant has properly performed the stability evaluation of the SB for floatation, sliding, and overturning using the methodology and loads described in FSAR Subsection 3.7.2.8, 3.7.3.17 and Figure 3H.3-52. The stability factors all exceeded the minimum values in SRP 3.8.5. Based on the above details, the staff concluded that the stability evaluation for the SB performed by the applicant is acceptable.

c. *Stability Evaluation of the Radwaste Building*

The RWB stability evaluation is described in FSAR Subsection 3H.3.5.3. Seismic input motions used for the stability evaluation are the induced acceleration response spectra due to site-specific SSE that is determined from an SSI analysis that accounts for the impact of the nearby RB (Figures 3.7-44 through 3.7-46). The seismic demands for the stability evaluation are determined by the RSA of a fixed-base stick model described in Subsection 3H.3.5.1. During the audit in May, 2011, the staff reviewed the stability evaluation calculations for the RWB. During this audit, the staff confirmed that the applicant has followed the methodology described in Figure 3H.3-52 for calculating the stability factors for floatation, sliding, and overturning. Loads used for the stability evaluation conformed to those shown in FSAR Table 3H.9-1. The static coefficient of friction of 0.58 and the sliding coefficient of friction of 0.39 (two-thirds of 0.58) was used based on site-specific soil properties. Conservatively, only 90 percent of the dead load was considered for calculating the floatation safety factor. Sliding and overturning safety factors were calculated for wind, tornado, and seismic loads. The calculated factors of safety for all cases exceeded the minimum value specified in SRP Section 3.8.5. During the audit, the staff identified that the stability evaluation did not consider the latest amplified input motions obtained from the MSM SSI analysis. Subsequently, the applicant confirms in the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232) that the RWB stability evaluation was re-analyzed considering the amplified input motions obtained from the MSM SSI analysis of the RB. FSAR Table 3H.6-14 was updated with the revised safety factors and was later incorporated into the FSAR. Based on the above details, the staff concluded that the stability evaluation performed by the applicant for the RWB is acceptable.

d. *Stability Evaluation of the Control Building Annex*

For the stability evaluation of the CBA, the SSE input at the foundation level (Figures 3.7-47, 3.7-48, and 3.7-49) is the induced acceleration response spectra due to the site-specific SSE that is determined from an SSI analysis that accounts for the impact of the nearby CB. Seismic demands along each orthogonal direction for the stability evaluation of the CBA are calculated using manual calculations, where the CBA is idealized as a single degree of freedom structure. Three orthogonal seismic demands are combined using the 100-40-40 rule as outlined in RG 1.192, Revision 2. During the audit in May, 2011, the staff reviewed the calculation for the stability evaluation of the CBA. The stability evaluation of the CBA was performed using a preliminary design of the structure with dimensions 25 m (L) x 23.54 m (W) x 7.56 m (H) (82.09 ft x 77.22 ft x 24.8 ft). The structure is surface mounted with a 1.22-m (4-ft) thick basemat with the top of basemat at elevation 10.67 m (35 ft) MSL. A stability evaluation was performed for wind, tornado, seismic, and flood loading. The factors of safety against floatation, sliding, and overturning were 1.19, 1.16, and 2.03, respectively. These values are all above the minimum allowable values per SRP Section 3.8.5 and are considered acceptable. Seismic loads were evaluated considering the structure as a single degree of freedom system. Flood loads were based on the PMF level of 12.19 m (40 ft) MSL. It was noted that the wind loads were based on the provisions of ASCE 7–88 instead of the newer ASCE 7–05 referenced in SRP Section 3.3.1. Also, it was noted that live loads were included in calculating the stabilizing forces for the stability evaluation. In the Supplement 3 response to **RAI 03.07.02-13** (ML11213A094), the applicant confirms that the CBA stability evaluation calculation was revised to address the above issues, and there was no change in the reported safety factors. Based on the review, the staff concluded that the stability evaluation of the CBA performed by the applicant is acceptable.

Based on the above discussions, the staff concluded that all issues related to design and stability evaluation of the non-seismic Category I structures are resolved. The procedures described above concerning the analysis and design of non-seismic Category I structures, the applicant's proposed measures for the completion and verification of the design using the ITAAC, and the results of the stability evaluation using appropriate seismic input and methodology provide reasonable assurance that these structures will have a safety margin against failure that is equivalent to the margin of safety of seismic Category I structures. This topic is therefore resolved.

Supplemental Information

3.8.4.4.4 DNFSB Issue: Resolution of Issues with Subtraction Method of Analysis

In **RAI 03.07.01-29** and follow up **RAI 03.07.01-30**, the applicant was asked to address a technical issue identified by DNFSB/DOE, which states that results may be non-conservative when analyzing embedded structures using the SM of analysis in the SASSI computer code. As the SM had been used for several of the SSI/SSSI analyses, the applicant was asked to demonstrate the acceptability of the SM of analysis to ensure that the SSCs are designed to meet the requirements of GDC 2 and specifically to address the following issues:

(a) compare structural loads, and any other design response quantities used in the design and developed using the SM with those using the DM or the MSM and evaluate the differences;

(b) demonstrate and justify that the differences identified in item (a) either have no impact on the design of seismic Category I SSCs or revise the design to address the differences;

(c) if the MSM is used to validate the SM, provide a validation program for the MSM; and

(d) provide FSAR markups to document the actions taken, the adequacy of the obtained results, and any design modifications resulting from the differences in the dynamic response.

In the response to **RAI 03.07.01-29** dated June 16, 2011 (ML11168A168), the applicant describes the plan for addressing the issues indicating that (a) the design of the RB, CB, DGFOT and RSW piping tunnels was already based on the SSI results from the DM, and thus no action was needed; (b) the evaluation of the impact on the design would be carried out for the UHS/RSW pump house and the DGFOSV, because the design was based on the SSI results utilizing the SM; and (c) an assessment of the 2D-SSSI results, which were based on the SM, would be performed by re-analyzing one sample case with the DM. The response also includes a plan for the validation of the MSM by comparing it with the DM for the CB model. The applicant's plan for addressing the issue was subsequently discussed in detail with the applicant during audits in July and September 2011 (ML112160142 and ML113140513, respectively). It was noted that the results of the SSI analysis were used to determine the seismic accelerations and the section cut forces for the design of the structures, and the results from the SSI and SSSI analyses were used to determine seismic soil pressures. Furthermore, amplified seismic input motions for lighter structures adjacent to heavy structures were determined from the SSI analysis of the heavy structure. Therefore, it is necessary to evaluate the impact on the design and the stability evaluation of structures wherever the SM was used to determine the seismic input motion, seismic accelerations, and seismic soil pressure.

In Revision 1 to the Supplement 1 response to **RAI 03.07.01-29** dated November 28, 2011 (ML113360516), the applicant provides a detailed evaluation of the impact from using the SM for the SSI and SSSI on the structural design. A detailed assessment of the applicant's response regarding the impact from using the SM on the structural design is included in SER Subsection 3.7.2.4.20, "DNFSB SASSI Subtraction Method Issues." A few places in the discussion in this SER section reference the evaluation performed in SER Section 3.8. An evaluation of those topics is in the paragraphs that follow. Also, the impact of using the SM on the stability evaluations is in the applicant's Supplement 4 response to **RAI 03.07.02-13** (ML11335A232) and is discussed below.

a. *Evaluation of the UHS Basin Columns and Beams*

As discussed earlier in SER Subsection 3.8.4.4.2 under Subpart B.1.4, (Item b), "Design of Submerged Columns due to Hydrodynamic Effects," the effect of the hydrodynamic mass on the design of the UHS basin columns was not initially included and was later accounted for by multiplying the accelerations obtained from the SSI analysis by a scale factor, as described in the Supplement 4 response to **RAI 03.08.04-30** (ML11181A002). Since the SM of analysis was used in the above SSI analysis to determine the accelerations in the columns, it was necessary to evaluate the impact of the MSM on these accelerations. Accordingly, in the Revision 1 to the Supplement 1 response to **RAI 03.07.01-29** (ML113360516), the applicant provides a procedure for evaluating the UHS basin concrete beams and columns for the impact of the MSM. The procedure consists of repeating the analysis of the UHS basin for the UB soil case for the full and empty basin cases using first the SM and then the MSM. A new scale factor was determined by dividing the analysis forces obtained from the MSM of analysis with those obtained from the SM of analysis. All of the SM design forces obtained from the original UHS basin analysis for the envelope of all soil cases were then multiplied by the new scale factors to obtain the MSM design forces with the minimum value of the scale factor being 1. The beam and column design based on the SM design forces are then checked against the MSM design forces for adequacy. The applicant concludes that based on the results of this assessment, all UHS basin concrete beams and columns designed based on the SSI analysis using the SM will be adequate for the SSI results of the analysis using MSM. The staff found that even though the scale factors were developed based on only the UB soil case, the procedure described by the applicant provides reasonable assurance that the impact of the MSM on the design of the UHS basin beams and columns is negligible, because the design based on the SM design forces is adequate for the MSM design forces for all beams and columns.

b. *Benchmark Study*

As discussed in SER Subsection 3.7.2.4.21, "Confirmatory Analysis of SASSI2000 Section Forces," section cut forces from the SSI analysis using the SASSI2000 program were used to compare the results from the analysis of the SM and MSM to address the DNFSB issues. The purpose of the benchmark study was to show that the section cut forces from the SSI analysis using the SASSI2000 program were accurate or conservative. As noted in the Supplement 1 response to **RAI 03.07.01-29** dated November 14, 2011 (ML113250374), in order to benchmark the calculation of section cut forces from the SASSI2000, the applicant repeated the dynamic analysis performed in the SASSI2000 using the SAP2000 program with an identical input and nearly identical model. The structural mesh of the models was identical to the so-called coarse mesh (also referred to as the original mesh) model used for the SSI analysis of the UHS/RSW pump house. The SAP2000 model was run as a fixed base and the SASSI2000 model was modified by adding massless solid elements and fixing them at the base to simulate a fixed-base condition. Details of the SASSI2000 fixed-base modeling are in the Supplement 2 response to **RAI 03.07.01-29** (ML11364A098). Other details of the dynamic analyses using the SAP2000 and SASSI2000 are also in the same RAI response. It is noted that only the full basin case was considered in this benchmark study.

In the Supplement 2 response to **RAI 03.07.01-29**, the applicant compares the section cut forces for the 19 section cuts from the SASSI2000 and SAP2000 fixed-base analyses. Based on this comparison, in a significant majority of the cases (88 out of 95 force components), the SASSI2000 results are higher than the corresponding SAP2000 results. In these cases, the large force for large force components, the SASSI2000 results are within 15 percent of those from the SAP2000. For relatively small force components, in some cases the percentage difference is higher. For 7 out of 95 force components, the SAP2000 results are higher than the SASSI2000 results. In five of these seven cases, the SASSI2000 results are within 6 percent of the SAP2000 results. For the remaining two cases, the difference is between 15 percent and 24 percent for relatively small moments of 370 kN-m (273 kip-ft) and 154.56 kN-m (114 kip-ft). It is noted that although the SAP2000 and SASSI2000 models are nearly identical, there are differences in the modeling of the fixed base for the two models. While the SAP2000 model is fixed base, the SASSI2000 model has below grade soil that is simulated to behave as fixed base. Also, the SASSI2000 analysis is based on the frequency domain analysis, whereas the SAP2000 analysis is based on the linear modal time history analysis. Due to these differences, minor differences in the results from the two analyses are expected. Considering the above details and the fact that only in very few cases were relatively large differences found for force components with relatively low values, the staff concluded that the applicant has adequately demonstrated that the section cut forces from the SASSI2000 analysis are conservative and reliable. To further demonstrate that the differences observed from the benchmark study do not have any adverse impact on the design, the applicant increased the section cut forces for those panels that showed the SASSI2000 results to be lower than the SAP2000 results by the percentage difference, and compared them with the available demand margin in section cut forces resulting from the use of the equivalent static method. The applicant also includes the increase factor from the use of the SM analysis and due to mesh refinement discussed in SER Subsection 3.7.2.4.20, “DNFSB SASSI Subtraction Method Issues,” along with the increase factor due to the benchmark study.

c. *Impact on the Stability Evaluations*

In the Supplement 4 response to **RAI 03.07.02-13** (ML11335A232), the applicant addresses the impact of the SM on the stability evaluation of the seismic Category I and non-seismic Category I structures. The applicant states that the SM of analysis may affect the SSI and SSSI results as well as amplify the input motions used in the stability analysis. Each of these items is addressed below as follows:

SSI Results

The results of the SSI analysis are used in the stability evaluations of the UHS/RSW pump house, RSW piping tunnels, DGFOSV, and DGFOT. The SSI analyses of these structures and the SSI analysis of the structure used to determine the applicable amplified input motion were performed using the MSM or DM of analysis. Therefore, no further evaluation is required.

SSSI Results

The results of the SSSI analysis were used in the stability evaluations of the RSW piping tunnels, DGFOSV, DGFOT, and RWB. The SSSI analysis for typical cross section of the RSW piping tunnels and the RWB was performed using the SM, MSM, and the DM of analyses. For other structures, the SSSI analyses were performed using the SM. Based on the detailed examination of the effect of the SASSI method of solution on the SSSI soil pressures in Revision 1 to the Supplement 1 response to **RAI 03.07.01-29**, the impact is expected to be within 10 percent. The minimum margin between the seismic sliding forces and the overturning moments used in the stability evaluations and those from the SSSI analysis is about 50 percent. Since this minimum margin of 50 percent is significantly more than the expected 10 percent difference from the SASSI method of analysis, no further evaluation is required.

Amplified Input Motions

Changes in the amplified input motion of light structures located adjacent to heavy structures may impact the results of the SSI analysis that were used to address the stability of the light structures. For STP Units 3 and 4, the amplified input motions are applicable to the DGFOSV, DGFOT, RSW piping tunnels, RWB, SB, and CBA structures. A comparison of the amplified input motions for the DGFOSV, DGFOT, RSW piping tunnels, RWB, and SB obtained from the MSM and SM SSI analyses of the RB show the following:

- The impact of the SASSI MSM on amplified horizontal input motion is negligible.
- The SASSI MSM affects the amplified vertical input motion in frequencies exceeding about 8 HZ.

The amplified input motions used in the stability evaluations of the RSW piping tunnels, DGFOSV, and DGFOT were obtained from the SSI analysis using the MSM. Therefore, no further evaluation is required for these structures.

The response spectra used for the stability evaluation of the SB envelopes the SB amplified input motions considering MSM. Therefore, no further evaluation is required for the SB.

The RWB stability evaluation was re-analyzed considering the amplified input motions obtained from the MSM SSI analysis of the RB. The calculated stability safety factors were found to exceed the required minimum safety factors. Therefore, no further evaluation is required for the RWB.

Amplified input motions from the MSM SSI analysis of the CB are not available for the CBA. However, the current stability evaluation of the CBA was performed using the following conservative measures:

- The superstructure mass lumped at the roof level was conservatively excited using a vertical acceleration equal to 1.5 times the peak spectral acceleration.
- In calculating the resisting forces and moments, as stated within the calculation, about 4,136.8 kN (930 kips) of the mass at the roof level was conservatively not considered.

Referring to COL FSAR Table 3H.6-14, the stability safety factors for the CBA are as follows:

- Sliding Safety Factor = 1.16.
- Overturning Safety Factor = 2.03.

Based on the above details, the most critical safety factor is the sliding safety factor. Eliminating the second conservative measure noted above will increase this safety factor from 1.16 to 1.44, which represents a 24 percent margin in the reported sliding safety factor.

Considering this and the additional margin from using the conservative vertical acceleration noted above, the existing margin in the calculation of the stability safety factors will be more than adequate for any change in the amplified input motions due to the use of the MSM SSI analysis of the CB. Therefore, no further evaluation is required for the CBA.

The staff's evaluation noted that the applicant has addressed the three results from the SASSI analysis that may have an impact on the stability evaluation of any Category I or II/I structure. The staff agreed that either

- The SASSI output used in the evaluation was already based on the MSM or the DM of analysis, or

- The expected change in the SASSI response value used in the stability evaluation was either negligible or of limited influence so as not to affect the adequacy of the already reported safety margins.

Therefore, this issue is resolved.

d. Vertical Excitation of Basin Water.

During the audit performed in February 2012, the applicant was asked to investigate the effect of the MSM on the vertical excitation of the basin water and on the pressures exerted on the basin walls. In the Supplement 3 response to **RAI 03.07.01-29** dated April 10, 2012 (ML12103A369), the applicant states that a comparison of the results of the analysis at the basemat level obtained with the MSM showed only a negligible impact on the vertical accelerations. For more details, see SER Subsection 3.7.2.4.3, "Hydrodynamic Effects of Water in the UHS Basin." Therefore, this issue is resolved.

3.8.4.4.5 Hurricane Wind Design: Design for Site-Specific Hurricane Winds and Missiles

Nuclear power plants must be designed so that they remain in a safe condition under extreme meteorological events, including those that could result in the most extreme wind events, tornadoes, and hurricanes that could reasonably be predicted to occur at the site. Initially, the tornadoes were considered to be the bounding extreme wind events. In RG 1.76 issued in April 1974, the design-basis tornado wind speeds were chosen so that the probability that a tornado exceeding the design basis would occur was on the order of 10^{-7} per year per nuclear power plant. In March 2007, the NRC issued Revision 1 of RG 1.76 that relies on the Enhanced Fujita Scale, which was implemented by the National Weather Service in February 2007. The Enhanced Fujita Scale is a revised assessment relating tornado damage to wind speed, which resulted in a decrease in design-basis tornado wind speed criteria in Revision 1 of RG 1.76. Since design-basis tornado wind speeds were decreased as a result of the analysis performed to update RG 1.76, it was no longer clear that the revised design-basis tornado wind speeds would bound the design-basis hurricane wind speeds in all areas of the United States. This prompted an investigation into extreme wind gusts during hurricanes and their relation to design-basis hurricane wind speeds, which resulted in issuing RG 1.221 in October 2011. RG 1.221 also evaluates missile velocities associated with several types of missiles considered for different hurricane wind speeds. The hurricane missile analyses presented in RG 1.221 are based on missile aerodynamics and initial condition assumptions that are similar to those used for the analyses of tornado-borne missile velocities adopted for Revision 1 to RG 1.76. However, the assumed hurricane wind field differs from the assumed tornado wind field in that the hurricane wind field does not change spatially during the missile's flight time, but does vary with height above the ground. Because the size of the hurricane zone with the highest winds is large relative to the size of the missile trajectory, the hurricane missile is subjected to the highest wind speeds throughout its trajectory. In contrast, the tornado wind field is smaller, so the tornado missile is subject to the strongest winds only at the beginning of its flight. This results in the same missile having a higher maximum velocity in a hurricane wind field than in a tornado wind field with the same maximum (3-second gust) wind speed.

The STP COL application incorporates by reference the ABWR DCD. Subsection 3.5.1.4 of the DCD states, in part, that "tornado-generated missiles have been determined to be the limiting natural phenomena hazard in the design of all structures required for safe shutdown of the nuclear power plant. Since tornado missiles are used in the design basis, it is not necessary to consider missiles generated from other natural phenomena." However, Subsection 3.5.4.2 of the DCD states, in part, that the COL applicant "shall identify missiles generated by other site-specific natural phenomena that may be more limiting than those considered in the ABWR design and shall provide protection for the structures, systems, and components against such missiles."

Accordingly, the staff issued **RAI 02.03.01-24** asking the applicant to address the following:

a) Consistent with the requirements of 10 CFR 52.79(a)(1)(iii), 10 CFR 100.20(c)(2), 10 CFR 100.21(d), and the COL license information requirement of ABWR DCD Section 3.5.4, identify hurricane wind speed and missile spectra for the STP site. RG 1.221 describes a method that the staff considers acceptable for selecting the site-specific hurricane wind speed and hurricane-generated missiles.

b) Pursuant to the requirements of GDC 2, GDC 4, and the Combined License Information requirement of ABWR DCD Section 3.5.4, confirm that the ABWR standard plant and the STP site-specific SSCs important to safety are designed to protect against the combined effects of hurricane winds and missiles defined above.

c) Revise the appropriate FSAR sections to appropriately reflect the results of questions a) and b) above.

In the response to this RAI dated January 12, 2012 (ML12018A387), the applicant proposed a revision to the FSAR that included all necessary changes to COL FSAR Chapters 1 and 2 to incorporate RG 1.221. The applicant determined the STP site-specific, design-basis hurricane windspeed to be 338 km/h (210 mph) for a 3-second wind gust using the new data and new guidance in RG 1.221. To ensure that the STP Units 3 and 4 design reflects the guidance in RG 1.221, the applicant's proposed revisions to COL FSAR Table 2.0-2, "Comparison of ABWR Standard Plant Site Design Parameters and STP 3 & 4 Site Characteristics," included a requirement for "STP Site Hurricane Wind Speed and Missiles." The applicant's proposed revision incorporated this change as a new site-specific departure, STP DEP 3.5-2, "Hurricane Generated Missile Protection." The applicant also stated in the response that a future supplement to this RAI would include a new Section 3H.11, "Design for Site-Specific Hurricane

Winds and Missiles," and other supporting changes to COL FSAR Chapter 3. The supplemental response to this RAI would also include changes to Section 3.0 of COL application Part 7, which would describe and justify site-specific Departure STP DEP 3.5-2. This departure will incorporate the guidance in RG 1.221. Additionally, COL application Part 9, "Inspections, Tests, Analyses and Acceptance Criteria", would be revised as necessary to ensure that these additional requirements are properly implemented.

The staff's review of the above proposed changes to FSAR Chapters 1 and 2 as necessary to incorporate RG 1.221 in the STP COL application is documented in Section 2.3 of this SER.

During an audit held on February 27, 2012 (ML120660018), the applicant presented the scope of and impact from applying RG 1.221 to FSAR Chapter 3 based on the applicant's calculations. Subsequently, the applicant submitted Supplement 1 to the response to **RAI 02.03.01-24** (ML12103A368; dated April 10, 2012) and provided the FSAR markup of Section 3H.11. The response also includes (1) the FSAR markup of corresponding changes to Sections 3.3, 3.4, and 3.5; (2) the COL application Part 7 markup that adds Departure STP DEP 3.5-2; and (3) the COL application Part 9 markup that revises ITAAC Tables 3.0-1, 3.0-5, 3.0-17, and 3-0-21 through 3.0-24 by adding hurricane wind and missiles to the list of natural phenomena and Tables 3.0-25 through 3.0-27 to address hurricane winds and missiles for the standard plant structures. The staff reviewed the above response and the supporting calculations during audits held on July 24, 2012 and December 10, 2012 (ML12229A301 and ML12366A019). The applicant provided additional clarifications to FSAR Section 3H.11 in Supplements 2 through 5 of **RAI 02.03.01-24** dated May 22, 2012 (ML12150A161); August 29, 2012 (ML12249A036), October 8, 2012 (ML12289A112), and January 3, 2013 (ML13037A595), respectively; which are based on discussions held during these audits and were also reviewed by the staff. The following paragraphs describe the staff's review of information included in FSAR Section 3H.11, along with the other COL application markups stated in the above responses.

1. *Evaluation of FSAR Section 3H.11: Design for Site-Specific Hurricane Winds and Missiles*

In FSAR Section 3H.11, the applicant describes the hurricane parameters, loads and load combinations, evaluation methods, and design results for hurricane winds and hurricane missiles of all seismic Category I standard plant structures and site-specific structures; in addition to a stability evaluation of non-seismic Category I structures.

(a) *Hurricane Parameters, Loads, and Load Combinations*

Hurricane Wind Parameters

RG 1.221 describes a method that the staff considers acceptable for selecting site-specific hurricane wind speeds and hurricane-generated missiles.

In the Supplement 1 response to RAI 02.03.01-24 (ML12103A368), the applicant describes the hurricane wind parameters in proposed Section 3H.11.1. The applicant uses the maximum hurricane wind speed of about 338 km/h (210 mph) as described in FSAR Subsection 2.3S.1.3.3.2, "Site-Specific Design-Basis Hurricane," and in Table 2.0-2. This information is consistent with the guidance in RG 1.221 and is discussed in Section 2.3 of this SER. The applicant specifies a spectrum of three hurricane missiles. The missile spectrum and the associated velocities for the missiles are:

a 1810-kilogram (4,000–pound) automobile at 59.7 m/s (133.6 mph) (horizontal) and 26 m/s (58.17 mph) (vertical)

a 130-kilogram (287–pound) Schedule 40 pipe at 46.5 m/s (104 mph) (horizontal) and 26 m/s (58.17 mph) (vertical)

a 66.7-gram (0.147–pound) solid steel sphere at 41.1 m/s 92 mph) (horizontal) and 26 m/s (58.17 mph) (vertical)

The missile spectrum chosen by the applicant is consistent with the list in RG 1.221, Table 1, "Design-Basis Hurricane Missile Spectrum." The horizontal and vertical missile impact velocities are consistent with those for a hurricane windspeed of 338 km/h (210 MPH) per RG 1.221 Table 2, "Design-Basis Missile Velocities as a Function of Hurricane Windspeed." The hurricane wind missile spectrum and missile velocities used by the applicant are considered acceptable because they are consistent with the guidance in RG 1.221. This information has been incorporated into the FSAR Revision 8.

Loads and Load Combinations

In Section 3H.11.1 of the FSAR provided with the Supplement 1 response to RAI 02.03.01-24, the applicant defines the hurricane load effects as the sum of the hurricane wind pressure and hurricane missile impact load. The applicant computes the hurricane wind pressure on structures using the procedure described in Chapter 6 of ASCE 7–05, "Minimum Design Loads for Buildings and Other Structures." The staff found the applicant's calculations to be consistent with the guidance in SRP Section 3.3.1 and is therefore acceptable.

The applicant evaluates the structures for the hurricane missile impact for predicting both local damage and overall damage. Local damage in terms of the penetration, perforation, and spalling of concrete structures is evaluated using the U.S. Department of the Army formula in TM 5-855-1, "Fundamentals of Protective Design for Conventional Weapons," dated November 1986. In addition, Table 1 of SRP Section 3.5.3 lists the minimum required concrete thickness specified for Region II. This specification was previously accepted by the staff for the ABWR design and is thus acceptable. A local damage prediction performed for steel barriers used the Ballistic Research Laboratory (BRL) formula described in *Reactor Safeguards* by C. R. Russell (published by Macmillan; New York, 1962). This formula is acceptable per the guidance in SRP Section 3.5.3 for local damage predictions of steel barriers. An overall damage evaluation of concrete and steel barriers was performed in accordance with Revision 3 of SRP Section 3.5.3, by comparing the flexure and shear capacity of the barrier with the total flexure and shear demand. These evaluations determined the total flexure and shear demand by considering the total hurricane wind consisting of hurricane wind pressure and missile impact in combination with other applicable normal loads. The other normal loads that are considered in the load combinations with the hurricane load effects are dead loads, live loads, the normal operating temperature, normal operating piping and equipment reactions, and lateral soil pressure and groundwater effects under normal operating conditions. The staff found that the applicant's methodology for the overall damage prediction is acceptable, because the applicant considered all applicable loads for the hurricane impact evaluation similar to that used for evaluating tornado wind effects.

In addition, the applicant also analyzed structures for the global effects of hurricane winds and missiles. For any critical missile hit location considered, the entire structure was analyzed for the resulting equivalent static load from the hurricane missile impact in conjunction with the hurricane wind pressure. The resulting induced forces and moments from this analysis were combined with the induced forces and moments from other applicable loads within the load combination to determine the total demand for the design of the structural elements.

Based on the above, the staff found that the loads and load combinations used by the applicant to determine the effects of hurricane winds and missiles on structures are acceptable; because the hurricane load considers the total effect of the hurricane wind pressure and the missile impact in accordance with the guidance in SRP Sections 3.3.1 and 3.5.4, and these loads are used in load combinations that consider other normal loads similar to the load combination used for the tornado load, which has the same probability of occurrence.

(b) Evaluations for Hurricane Design

In Section 3H.11.2 of the FSAR provided with the Supplement 1 response to **RAI 02.03.01-24**, the applicant describes the methodology used for local and global evaluations of the effects on structures from hurricane winds and hurricane-generated missiles. Subsequently, in the Supplement 4 response to **RAI 02.03.01-24** (ML12289A112; dated October 8, 2012), the applicant provides further details of the evaluations. Discussions of these evaluations follow.

Local Evaluations

For local damage predictions of concrete structures, the applicant performed penetration, perforation, and scabbing calculations for hurricane-generated missiles using the TM 5-855-1 formula. For steel structures, the assessment was based on the BRL formula. The acceptance criteria state that the thickness of the walls and the roofs of each structure shall not be less than the minimum thickness required to prevent penetration, perforation, and scabbing. During the design audit, the staff reviewed the design calculation that provides a local assessment of the minimum thickness for hurricane missiles based on the empirical equations from TM 5-855-1. The staff noted that the applicant has used the TM 5-855-1 formula appropriately and as recommended in TM 5 855-1; has applied a 30 percent correction factor for a missile velocity of less than 305 m/s (1,000 ft/s) (as is the case for the STP site-specific missile velocity); and has applied a 15 percent correction factor to account for the accuracy of the equation. Furthermore, the minimum concrete thickness per Table 1 of SRP Section 3.5.3 for Region II was determined to be 39.1 cm (15.4 in.).

The overall damage prediction of barriers is performed using a flexure and shear capacity evaluation of the panel impacted by the hurricane missile that considers the total hurricane load, in conjunction with all other applicable loads per the load combinations discussed above. The guidance in RG 1.221 states that the automobile missile should be considered to have an impact at all altitudes of less than 9.1 m (30 ft) above all grade levels within 805 m (0.5 miles) of the plant structures. In the February 2012 audit, the applicant provides a site plan to demonstrate that the automobiles are not parked within 805 m (0.5 miles) of the plant structures at levels higher than the plant grade level. The applicant evaluated the structures for automobile missile impacts at altitudes up to 9.1 m (30 ft) above grade level.

The hurricane-generated automobile missile impact controlled the overall damage to local panels in the design of the structures. The applicant adopted the automobile missile impact forcing function recommended in a report from the ASCE Committee proceedings on impactive and impulsive loads (Second Conference on Civil Engineering and Nuclear Power, 1981). The report is based on the peak impact force of 2,046 kN (460 kips) for an automobile impacting the barrier at a velocity of 96 km/h (60 mph). The applicant scaled up the peak impact force linearly in the ratio of the peak automobile impact velocity of 215.6 km/h (134 mph) for the STP site. In the Supplement 3 response to **RAI 02.03.01-24** (ML12249A036; dated August 29, 2012), the applicant provides the calculated peak impact force of 4,555 kN (1,024 kips) and 1979.4 kN (445 kips) for the horizontal and vertical impact, respectively. The dynamic forcing function shape is defined in the shape of a triangular pulse with a total duration of 0.1 second.

The applicant's Supplement 1 response to **RAI 02.03.01-24** also justifies using the impact force from an automobile impact that uses the ASCE formulation by comparing it with the value obtained using the Bechtel Topical Report BC-TOP-9A, "Design of Structures for Missile Impact," which is accepted by the NRC for missile impact evaluations. Equation 5-1 in Section 5.1 of the Bechtel topical report provides the formulation for determining the impact force. Using Equation 5-1, for a 1810-kilogram (4,000–pound) automobile missile with a horizontal velocity of 215.6 km/h (134 mph), the maximum impact force will be about 2,179.5 kN (490 kips). For a triangular impulse, the peak dynamic load factor (DLF) is less than 1.7. Considering a peak DLF of 1.7 and a 2,179.5-kN (490–kips) impact force, the equivalent static impact force will be 3,705 kN (833 kips). Based on the above comparisons, the staff found that the forcing function the applicant used for the automobile impact is acceptable. For the overall damage assessment of panels for hurricane missile loads, the applicant calculated the dynamic load factor for the wall or roof panel on a case-by-case basis. The minimum dynamic load factor was limited to 1.0 for the triangular pulse loading to ensure that the structures are assessed for at least 4,555 kN (1,024 kips) and 1979.4 kN (445 kips) for a horizontal and vertical impact, respectively.

In Section 3H.11.2 of the FSAR, the applicant describes the methodology used to design the panels for a missile impact in the elastic and plastic range based on the available concentrated force capacity of the panel, after considering other applicable loads and the hurricane wind pressure load. The panel is considered to be in the elastic range if the product of the DLF and the impact force is less than or equal to the available concentrated force capacity of the panel. For panels in the plastic range, the ductility demand is limited to the maximum allowable ductility ratio based on ACI 349-97. The staff found that the applicant's methodology for assessing the overall damage to the panels follows standard engineering practice and is therefore acceptable.

Global Evaluations

Evaluations of global effects from hurricane winds and missiles on structures considered the following:

The structure, in its entirety, is evaluated for the total hurricane load in conjunction with all other applicable loads using the load combinations described in FSAR Section 3H.11.1. The missile loads are applied at critical locations of walls running parallel to the missile impact loads. The induced effects are combined with other applicable loads.

The sliding and overturning stability of the structure is evaluated considering the total hurricane load in conjunction with other applicable loads. The load combination in SRP Section 3.8.5 for tornado wind loads is used for stability evaluations by replacing the tornado load with hurricane load. Minimum required factor of safety for sliding and overturning is set to be 1.1.

The staff found the applicant's methodology for evaluating the global effects of hurricane winds and hurricane missiles on structures acceptable, because it will appropriately estimate the additional stresses in the entire structure and demonstrate the stability of the structure per the guidance in SRP Section 3.8.5.

(c) Structures Designed for Site-Specific Hurricanes

The following seismic Category I structures are designed for site-specific hurricane:

- RB
- CB
- RSW piping tunnels
- UHS/RSW pump house
- DGFOSV
- DGFOT

The non-seismic Category I structures with the potential for interacting with seismic Category I structures were evaluated for site-specific hurricane. Site-specific hurricane are used for stability evaluations of and designs of lateral load-resisting systems for the following non-seismic Category I structures:

- TB
- SB
- RWB
- CBA
- Stack on the RB roof

Evaluation of Site-Specific Seismic Category I Structures

The site-specific seismic Category I structures consist of the UHS/RSW pump house, the DGFOSV, and the RSW piping tunnels. In the February 2012 audit (ML120660018), the staff reviewed the design calculation that supports the evaluation summary data provided with the Supplement 1 response to RAI 02.03.01-24 in Tables 3H.11-1 and 3H.11-2, for hurricane missile impact evaluations of the UHS/RSW pump house, and the DGFOSV. The staff determined that the applicant's assessment of the site-specific structures is consistent with the methodology described in Sections 3H11.1 and 3H11.2 of the FSAR.

The staff's review of the design calculation confirmed that in all cases, a peak triangular pulse load of 4,555 kN (1,024 kips) in the horizontal direction and 1979.4 kN (445 kips) in the vertical direction was applied in the evaluation of the wall and the roof panel, respectively. The triangular pulse with the duration of 0.10 second was used as a dynamic forcing function. The dynamic load factor was limited to at least 1.0. The factors of safety against sliding and overturning meet the structural acceptance criteria. It was noted that the applicant has considered several different locations for the missile impact on a panel in order to determine the most critical impact location. The staff also confirmed that the applicant has appropriately used sections of ACI 349–97 (e.g., Subsection 11.12.1.2 for shear requirements and Appendix C for special provisions for impulsive and impact effects) to calculate the shear and flexure capacity of structural elements.

The calculations determined that (1) the hurricane loading does not control the overall design and stability of the UHS/RSW pump house; (2) the wall and slab designs are adequate to resist local damage from hurricane-induced missiles; and (3) when considering the total hurricane load in combination with all other applicable loads, the flexural and shear capacity of the wall and the roof panels impacted by the hurricane missiles is adequate.

For the DGFOSV, the design-basis hurricane loading controls some walls and slabs. The flexural and shear capacity of wall and roof panels impacted by hurricane missiles is adequate with the addition of some shear reinforcement over the tornado and seismic design.

The evaluation of the RSW piping tunnel wall and slab panels was performed by comparisons with the panel assessment of the UHS/RSW pump house, the DGFOSV, and the DGFOT for site-specific hurricane winds and missiles. The minimum thickness of the exterior wall and slab panels for the RSW piping tunnel access areas is 110 cm (36 in.) and 73.1 cm (24 in.), respectively. The staff agreed with the applicant's assessment that these panels would be adequate for hurricane winds and missiles based on the evaluations of similar panels for the DGFOSV and DGFOT.

The staff reviewed the detailed design calculations and reported design margins and found that the STP Units 3 and 4 site-specific seismic Category I structures are adequate for the hurricane wind and hurricane missile effects.

Evaluation of ABWR Standard Plant Structures

The ABWR standard plant seismic Category I structures consist of the RB, CB, and DGFOT. These structures are designed for tornado winds and missiles per DCD Tier 1, Table 5.0. FSAR Table 3H.11-6 compares the hurricane and tornado wind and missile parameters for these structures. The design-basis tornado windspeed is 482.7 km/h (300 mph) compared to the site-specific hurricane windspeed of 338 km/h (210 mph). However, hurricane wind pressure increases with height, whereas tornado wind pressures do not vary with height. FSAR Figures 3H.11-4 and 3H.11-6 compare hurricane wind pressures on the RB and the CB, respectively. Site-specific hurricane wind pressures on the RB are greater than the tornado wind pressures at approximately 183 m (60 ft) above grade. Also, the automobile missile impact velocity for the site-specific hurricane is greater than that for the DCD tornado wind. Therefore, the standard plant structures are evaluated for site-specific hurricane wind and automobile missile impacts. The staff reviewed the methodologies used for the local and global evaluations of the hurricane wind and auto missile impact loads on the RB, CB, and the DGFOT. The staff found that the general methodologies are acceptable.

The staff reviewed the hurricane missile impact evaluation of the DGFOT's roof and access region walls. The staff noted that for structures smaller than the automobile footprint (e.g., Table 3H.11-3), the 4,555 kN (1,024 kips) load is not used for automobile missile impact. In such cases, the applied peak impact load is appropriately scaled in proportion to the width of the structure. The staff found the applicant's evaluations acceptable because they followed the methodology for local and global evaluations. FSAR Table 3H.7-2 reports the results of the DGFOT stability evaluation. The minimum calculated safety factor against sliding and overturning for a hurricane wind is 1.23. However, the applicant proposes to provide a restraint for stability that considers the automobile missile impact during the detail design to maintain the minimum factor of safety of 1.1 against sliding and overturning. This is considered acceptable.

The global hurricane wind pressure on the RB is enveloped by the global tornado wind pressure from grade up to approximately 183 m (60 ft) above grade. From approximately 183 m (60 ft) above grade to the top of the RB, the global hurricane wind pressure exceeds the global tornado wind pressure. However, based on a comparison of the seismic shear versus the total hurricane shear shown in FSAR Figure 3H.11-5, the applicant determined that the shear forces from the hurricane loading are less than 10 percent of the shear forces from the seismic loading. Based on this finding, the applicant concluded that the load combination comprised of hurricane loading would be enveloped by the seismic loading for global effects of hurricane winds and missiles on the RB. The staff found the applicant's assessment acceptable considering that the seismic loading on the RB is significantly higher than the global effects of hurricanes.

FSAR Figure 3H.11-6 shows that the global hurricane wind pressure on the CB is enveloped by the global tornado wind pressure. A comparison of the seismic shear versus the total hurricane shear on the CB (FSAR Figure 3H.11-7) shows that the hurricane loading is significantly less than the seismic loading. Based on this finding, the applicant concludes that the hurricane loading has no impact on the global design of the CB. The staff found the applicant's assessment acceptable considering the significant margin in seismic loading compared to hurricane wind loading. FSAR Table 3H.2-5 shows the applicant's results of the CB stability evaluation for the hurricane loading.

The Supplement 3 response to **RAI 02.03.01-24** includes the applicant's results for local evaluations of the RB and CB panels. Based on the comparison of site-specific hurricane missiles per RG 1.221 and tornado missiles for DCD structures in FSAR Table 3H.11-6, the exterior wall panels of the RB and CB that are susceptible to a horizontal hurricane-generated automobile missile impact require additional panel assessments. The applicant's results of the panel evaluations for the RB and the CB are in FSAR Tables 3H.11-4 and 3H.11-5, respectively. To maximize the effects of the automobile missile impact on the flexure and shear, the applicant considered multiple impact locations (near the center of the panel for the flexure and near the support for the shear). Critical panels of the RB and CB selected for evaluation were based on panel thickness, span, and reinforcements. The staff reviewed the supporting calculations for the RB and CB panel evaluations during an audit held in July 2012 (ML12229A301). The staff found that the applicant has appropriately followed the methodology for local evaluations, and the flexure and shear demands were within corresponding capacities of the panels.

During the audit of July 2012, the staff noted that the lowest reported margin in the CB panel for the automobile missile impact load is 3 percent. The margin is controlled by a shear deformation of the CB wall panel. Furthermore, the calculation for the flexure capacity of the panels did not appear to include panel sizes and the panel aspect ratio; and entire panel widths were considered for the shear capacity evaluation. These concerns were discussed with the applicant, and the staff requested the following clarifications:

Provide the basis for calculating the panel flexural capacity. The formula for the flexure panel capacity used in the calculation is independent of the panel dimensions, panel aspect ratio, and the location of the impact load on the panel.

In the shear capacity calculation for CB panel, the entire panel width of 74.8' is considered. Please provide the basis for determining the effective panel width for shear resistance and the rational for selecting the entire panel width.

In the calculation of the shear capacity for the CB panel, a margin of 3 percent is shown in one case between the panel's shear resistance capacity and the automobile impact force. Please perform a sensitivity analysis of the assumed central automobile impact location, its orientation, and its configuration (full or partial automobile foot print) on the reported 3 percent margin to show that any other postulated impact location, orientation, or configuration will not result in a margin of less than 3 percent. Also, provide the basis for determining the effective shear area to calculate the shear resistance.

In calculating the flexure and shear capacity of the panel, large deformations of the panels are likely to occur. Please provide the estimated magnitude of such deformations and assurances that the function of any safe shutdown component or equipment attached to or in the vicinity of the panel is not affected due to large panel deformations.

In the Supplement 5 response to RAI 02.03.01-24 dated January 3, 2013 (ML13037A595), the applicant provides the clarifications discussed below:

- The formula used to determine panel flexure capacity is per its guideline document "Guidelines for Extreme Wind and Tornado Design," which states that the formula is applicable for a concentrated load anywhere on the slab. The same formula is also listed in Table 4-3 of Bechtel Topical Report BC-TOP-9-A. The applicant also refers to the text book *Reinforced Concrete Slabs,* Second Edition, (Park and Gamble; 2000) stating that for slabs with concentrated loads, the collapse mechanism involving curved yield negative moment lines are more critical than the straight-line mechanisms involving large triangular segments. The failure cone could have any radius that lies within the slab, so the ultimate concentrated load is the same for any position of the concentrated load and for any shape of the slab with fixed edges. The staff found the applicant's justification acceptable because the formula used for determination of flexural capacity of the panel resulted in the lowest capacity, when considering other failure mechanisms.

- To determine the shear capacity, the applicant refers to Sections 11.10.1 and 11.121.1 of ACI 349-97 and to Section 10.1 of the text book *Reinforced Concrete Slabs*. This book states that the shear strength of the slabs in the vicinity of concentrated loads is governed by the more severe of two conditions: either beam action or two-way action. In beam action, the slab fails as a wide beam with the critical section for the shear extending along a section in a plane across the entire width of the slab. In the two-way action, the slab fails in a local area around the concentrated load. The applicant also stated that the punching shear of the slab was checked separately in another section of the hurricane evaluation calculation. The staff reviewed the applicant's response and concluded that the applicant has followed the provisions of ACI 349-97 for determining the shear capacity of wall panels subjected to concentrated loads. The methodology is also recognized as an acceptable practice in publications on the subject. Therefore, the applicant's method to determine the shear capacity of wall panels is considered technically acceptable.

- The applicant's response states that the 3 percent margin in the shear capacity of the CB panel against shear demand in one case meets the acceptance criterion, and there is no need to provide a certain amount of margin. In addition, the minimum impact force of 4,555 kN (1,024 kips) considered in the design has about a 23 percent margin over the maximum impact force of 3705 kN (833 kips), which was calculated using the Bechtel Topical Report BC-TOP-9A. Based on this finding, there is no need for an additional sensitivity analysis. The basis for an effective shear area in this case is that the shear will distribute in both directions at the impact location (considering a 45 degree dispersion from the impact area) because of the panel's size. The staff agreed with the applicant that the conservatism in the estimate of the impact force precludes the need for any additional sensitivity study to demonstrate the adequacy of the CB panel in shear. The staff also agreed with the applicant's conclusion that it is reasonable to consider that the shear will distribute in both directions and will be resisted by three sides of the panel using a 45 degree dispersion from the impact area, because the impact location for this case is at one end of the panel (4.66 m wide x 22.8 m long [15.3 ft wide x 74.8 ft long]) and the automobile missile impact area is 2.01 m x 1.31 m (6.6 ft x 4.3 ft).

- In response to the staff's concern about potentially large panel deformations from the missile impact, the applicant states that although local non-linear behavior is permitted for hurricane missile evaluations, there will be no gross failure (i.e., no perforation or scabbing) of the impacted panel or of the structure. Thus, no secondary missiles will be generated inside the structure. Considering how massive the structure is with respect to the impacting missile, the imparted shock from the hurricane missile will be localized and may be critical only for the instruments directly attached to the impacted wall near the point of contact. However, during a hurricane event, only a limited number of instruments/equipment is required for the safe shutdown. The layout and support of instruments are part of the detailed design, where such instruments are generally supported by instrument racks attached to the floor slabs. Considering this design configuration during a hurricane event, the safe shutdown function is not affected by the impact from a hurricane missile. The staff reviewed the applicant's response, and agrees with the applicant's assertion that safe shutdown function is not affected by the impact of hurricane missiles. The reasoning is that such an impact will not cause any gross failure of the panels or of the structure, and any potential local effect on safe shutdown components will be taken into consideration during the detailed design.

Based on the above review, the staff found that the design of ABWR standard plant structures is adequate for the hurricane wind and hurricane missile effects at the STP Units 3 and 4 site.

(d) Hurricane Evaluations for Non-Seismic Category I Structures

The applicant evaluates the potential impact from hurricane wind effects on the interactions between non-seismic Category I structures and seismic Category I structures. The non-seismic Category I structures considered in this evaluation are the SB, TB, RWB, CBA, and the RB stack.

In FSAR Subsection 3H.11.3.7, the applicant describes the evaluation of the hurricane wind on non-seismic Category I structures. The applicant performs a global assessment of hurricane loads to determine the impact on the structural stability of the non-seismic Category I structures. For this evaluation, the applicant compares the total seismic driving forces with the total hurricane driving forces on these buildings. The applicant determines that in all cases, the seismic driving forces govern the stability of these non-seismic Category I structures. The staff found the applicant's assessment acceptable, because the seismic driving forces are greater than the hurricane driving forces for the stability evaluation. For the RB stack, the applicant compares the impacts from tornado and hurricane wind forces on the stack; the hurricane wind pressure is enveloped by the tornado wind pressure on the RB stack. This finding is considered acceptable because the automobile missile is not credible for the RB stack, and other missiles are also enveloped by the tornado missiles. The applicant also revises the ITAAC design requirements for the TB, SB, RWB, and CBA seismic II/I interaction to include hurricane wind and hurricane missile parameters for the design of the lateral load-resisting system (see STP COL application Part 9, Tables 3.0-21 through 24).

(e) Protection of Openings of Seismic Category I Structures

The applicant provides design features such as missile-proof covers and doors, or labyrinth walls for the openings of seismic Category I structures for protection from the passage of hurricane-generated missiles through the openings in the roof slabs and exterior walls. The applicant also provides positive design measures by installing a heavy steel grating at the top of each fan enclosure compartment to protect the UHS/RSW pump house fan enclosure from the hurricane missile impact.

In the Supplement 1 response to **RAI 02.03.01-24**, the applicant also describes structural design measures for protecting the openings of seismic Category I structures—including the UHS/RSW pump house fan enclosure—from the hurricane missile impact. This response proposes FSAR revisions to Section 3H.6.2, Subsection 3H.6.3.2, and Table 3H.9-1; and adds new Section 3H.11. The staff concluded that the measures taken by the applicant to protect the openings in seismic Category I structures will provide positive barriers for protection against missiles. These measures are therefore acceptable.

Based on the above review, the staff found that the STP Units 3 and 4 structures are adequately designed for hurricane winds and hurricane missiles—according to the guidance in RG 1.221—and meet all applicable SRP acceptance criteria for design. Therefore, the issue of the hurricane loading and the application of RG 1.221 is resolved. FSAR markups in the Supplement 3, 4, and 5 responses to **RAI 02.03.01-24** that include additional clarifications are to be incorporated into the next FSAR update. Verifications that these proposed changes are included in next FSAR update are being tracked as **confirmatory items**.

2. *<u>Changes to FSAR Sections 3.3, 3.4, and 3.5</u>*

In the Supplement 1 response to **RAI 02.03.01-24**, the applicant provides FSAR markups of Sections 3.3, 3.4, and 3.5 that add references to the new Departure STP DEP 3.5-2 and to FSAR Section 3H.11 for the hurricane design. These changes are editorial, and the staff found them acceptable.

3. *<u>Revision to COL Application Part 7, Departure Reports</u>*

In the Supplement 1 response to **RAI 02.03.01-24**, the applicant provides the COL application Part 7 markup to include the new site-specific departure for including hurricane winds in the STP design. The applicant's evaluation of the departure states that pursuant to the requirements in 10 CFR Part 52, Appendix A, Section VII.B.5, the departure has no significant impact on the frequency or consequences of any accident or malfunction of an SSC important to safety that was previously evaluated. There is also no impact on the frequency or consequences of any ex-vessel severe accident that was previously evaluated. This change has no impact on any Tier 1, Tier 2*, technical specifications, bases for the technical specifications, or operational requirements information. Based on this information, the applicant concludes that this departure does not need prior NRC approval.

The applicant's evaluation determined that this departure does not require prior NRC approval in accordance with 10 CFR Part 52, Appendix A, Section VIII.B.5. Within the review scope of this section, the staff found it reasonable that the departure does not require prior NRC approval. The applicant's process for evaluating departures and other changes to the DCD is subject to NRC inspections.

4. *<u>Revision to COL Application Part 9, ITAAC Tables</u>*

In the Supplement 1 response to **RAI 02.03.01-24**, the applicant revises COL application Part 9 to add site-specific ITAAC Table 3.0-25, "Reactor Building–Design for Hurricane"; Table 3.0-26, "Control Building–Design for Hurricane"; and Table 3.0-27, "Reactor Building Stack–Category I/I Design for Hurricane." The staff reviewed these new ITAAC tables and found them technically acceptable for addressing the new hurricane wind design requirements for these structures. The applicant also includes in this response markups of revisions to ITAAC tables for the UHS (Table 3.0-1), the RSW system (Table 3.0-5), and the DGFOSV (Table 3.0-17) to include hurricane winds and hurricane missiles in the natural phenomena for the design of these seismic Category I structures. The applicant also provides revisions to ITAAC Tables for the TB (Table 3.0-21), SB (Table 3.0-22), RWB (Table 3.0-23), and CBA (Table 3.0-24) to include hurricane wind and hurricane missile parameters for designing lateral load-resisting systems for these structures to preclude seismic II/I interactions. The staff reviewed these revised ITAAC tables and found them technically acceptable.

3.8.4.5 *Post Combined License Activities*

The applicant identifies the following COL license information items resolutions:

- COL License Information Item 3.23 on foundation waterproofing by placing waterproofing membrane near the top elevation of the concrete fill

- COL License Information Item 3.25 on structural integrity test by performing the test in accordance with ITAAC Table 2.14.1 Item #3

The applicant identifies the following site-specific ITAAC:

- ITAAC Table 3.0-1 to verify the design requirements of the UHS, including a design report to reconcile the as-built data with the structural design basis.

- ITAAC Table 3.0-5 to verify the design requirements of the Reactor Service Water System, including a design report to reconcile the as-built data with the structural design basis.

- ITAAC Table 3.0-13 for a friction coefficient to meet the sliding requirements of water-proofing material.

- ITAAC Table 3.0-15 to field measure actual settlement seismic Category I structures to ensure maximum allowable tilt acceptance criteria is met.

- ITAAC Table 3.0-17 to verify the design requirements of the DGFOSV, including a design report to reconcile the as-built data with the structural design basis and an inspection of the vault.

- ITAAC Table 3.0-21 to verify the design requirements of the TB, including a structural analysis to confirm that the lateral load resisting system as designed and constructed meets the applicable design requirements and an inspection of the TB.

- ITAAC Table 3.0-22 to verify the design requirements of the service building, including a structural analysis to confirm that the lateral load resisting system as designed and constructed meets the applicable design requirements and an inspection of the service building.

- ITAAC Table 3.0-23 to verify the design requirements of the RWB, including a structural analysis to confirm that the lateral load resisting system as designed and constructed meets the applicable design requirements and an inspection of the RWB.

- ITAAC Table 3.0-24 to verify the design requirements of the CBA, including a structural analysis to confirm that the lateral load resisting system as designed and constructed meets the applicable design requirements and an inspection of the CBA.

- ITAAC Table 3.0-25 to verify the site-specific hurricane wind and hurricane missile design requirements of the RB and the DGFOT, including a structural analysis to reconcile the as-built data with the design requirements.

- ITAAC Table 3.0-26 to verify the site-specific hurricane wind and hurricane missile design requirements of the CB, including a structural analysis to reconcile the as-built data with the design requirements.

- ITAAC Table 3.0-27 to verify the site specific hurricane wind and hurricane missile design requirements of the RB stack, including a structural analysis to reconcile the as-built data with the design requirements.

3.8.4.6 *Conclusion*

NRC staff findings related to information incorporated by reference are in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to other seismic Category I structures that were incorporated by reference have been resolved.

The staff compared the supplemental information in the COL application to the relevant NRC regulations, acceptance criteria guidance in Section 3.8.4 of NUREG–0800, and other NRC RGs. The staff's review concluded that the application has provided sufficient information on the COL license information item and Tier 1 departures to satisfy the requirements and guidance in Section 3.8.4 of NUREG–0800. The staff found it reasonable that the identified Tier 2 departures are characterized as not requiring prior NRC approval per 10 CFR Part 52, Appendix A, Section VIII.B.5.

The staff found that and the application will be in compliance with NRC regulations upon the satisfactory resolution of the confirmatory items, with the verification of proposed changes in the next revision of the COL application.

3.8.5 Foundations

3.8.5.1 *Introduction*

This FSAR section addresses the foundations for all seismic Category I Structures. The ABWR design employs separate reinforced-concrete mat foundations for major seismic Category I Structures. The RB foundation, which is integral to the containment foundation, supports the containment structure, reactor pedestal, other internal structures, and the balance of the RB structure. Even though the containment structure foundation is integral to the RB foundation, it is a portion of the foundation within the perimeter of the containment structure. Therefore, the foundation is designed as a part of the containment boundary.

The criteria for the foundation design include the following:

- Description of the foundations
- Applicable codes, standards, and specifications
- Loads and load combinations
- Design and analysis procedures
- Structural acceptance criteria
- Materials, quality control, and special construction techniques
- Testing and in-service inspection requirements

3.8.5.2 *Summary of Application*

Section 3.8.5 of the STP Units 3 and 4 COL FSAR, Revision 9, incorporates by reference Section 3.8.5 of the certified ABWR DCD, Revision 4, referenced in 10 CFR 52, Appendix A. In addition, in FSAR Section 3.8.5 and Appendix 3H.6, the applicant provides the following:

Tier 1 Departure

- STD DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category 1 to Non-Seismic

This departure revises the seismic category of the RWB substructure from seismic Category I to non-seismic.

COL License Information Items

- COL License Information Item 3.23 — Foundation Waterproofing

In FSAR Subsection 3.8.6.1, the applicant states that foundation waterproofing is carried out by placing a waterproofing membrane near the top elevation of the concrete fill. The remainder of the concrete fill is then poured on top of the waterproofing material. A waterproofing membrane that could degrade the ability of the foundation to transfer loads is not used.

- COL License Information Item 3.24 — Site Specific Physical Properties and Foundation Settlement

In FSAR Subsection 3H.6.4.2 and Section 2.5S.4, the applicant provides physical properties of the site-specific subgrade medium and settlement of foundations.

Supplemental Information

In FSAR Sections 3H.6 and 3H.7, the applicant provides additional information about the foundation analysis and design of the DGFOT, UHS/RSW pump house, RSW tunnels and the DGFOSV.

3.8.5.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is in NUREG–1503. In addition, the relevant requirements of the Commission regulations for foundations, and the associated acceptance criteria, are in Section 3.8.5 of NUREG–0800.

In addition, in accordance with Section VIII, “Processes for Changes and Departures,” of, “Appendix A to Part 52 --- Design Certification Rule for the U.S. Advanced Boiling Water

Reactor," the applicant identifies one Tier 1 departure that requires prior NRC approval. Tier 1 departures require prior NRC approval and are subject to the requirements of 10 CFR Part 52, Appendix A, Section VIII.A.4.

Review of COL License Information Items 3.23 and 3.24 are subject to the acceptance criteria and guidance in SRP Section 3.8.5.

3.8.5.4 *Technical Evaluation*

As documented in NUREG–1503, NRC staff reviewed and approved Section 3.8.5 of the certified ABWR DCD. The staff reviewed Section 3.8.5 of the STP Units 3 and 4 COL FSAR and checked the referenced ABWR DCD to ensure that the combination of the information in the COL FSAR and the information in the ABWR DCD appropriately represents the complete scope of information relating to this review topic.[1] The staff's review confirmed that the information in the application and the information incorporated by reference address the required information relating to this section.

The staff reviewed the information in the COL FSAR:

<u>Tier 1 Departure</u>

- STD DEP T1 2.15-1 — Reclassification of Radwaste Building from Seismic Category 1 to Non-Seismic

The review and approval of this departure is in Subsection 3.8.4.4 of this SER.

<u>COL License Information Items</u>

- COL License Information Item 3.23 — Foundation Waterproofing

In ABWR DCD, this COL license information item requires the applicant to evaluate the capability of the foundation to transfer shear loads where foundation waterproofing is used. The COL application states that foundation waterproofing will include placing a chemical agent on the exposed concrete surface of the mudmat, and a waterproof membrane will be installed on the walls of all Category I structures to an elevation of 30.5 cm (1 ft) below grade with a waterproof coating being applied from that level up to the flood level. To evaluate the effectiveness of the proposed waterproofing, **RAI 03.08.04-5**, **RAI 03.08.04-19,** and **RAI 03.08.04-28** requested the applicant to clarify the issues listed below to provide additional information regarding the membrane, and to update the FSAR as appropriate.

1. <u>*Physical and Chemical Properties*</u>. **RAI 03.08.04-5 Item (a)** asked the applicant to provide details on the proposed chemical agent including application procedures, its performance to accommodate any potential cracking of the mudmat due to placement of the massive concrete foundation, and its ability to be effective as foundation waterproofing. The staff asked the applicant to provide supporting information that this type of waterproofing is adequate to protect the concrete foundations against degradation from aggressive soil/groundwater. The applicant's response to **RAI 03.08.04-5** dated September 15, 2009 (ML092610377), states that (a) the

[1] See *"Finality of Referenced NRC Approvals"* in SER Section 1.1.3, for a discussion on the staff's review related to verification of the scope of information to be included in a COL application that references a design certification.

waterproofing membrane is a spray-on, high-viscosity liquid that cures when exposed to air; (b) the material may be applied by brush, roller, or airless spray equipment; (c) the specific material for the waterproof membrane will be selected during detailed design; (d) the waterproofing will be placed and sandwiched within the concrete fill at about 30.5 cm (1 ft) down from the top of the fill and will be extended vertically on the walls up to about 30.5 cm (1 ft) under the finished grade level; (e) rebar and foundation embedments are not present in either of these structural concrete fill lifts; (f) the surface of both the structural concrete fill and the exterior walls will be prepared in accordance with procedures that will be determined during the material qualification testing program; (g) the surface of the wall will be prepared as necessary to assure that the waterproof coating application can bridge the small gaps and corners of the transition; and (h) the cured membrane has a degree of flexibility that allows it to accommodate mudmat concrete shrinkage and thermal cracking, thermal expansion, and other minor movements between substrate members.

Follow-up **RAI 03.08.04-19, Item 1** asked the applicant to describe the specific material that will be used for the waterproof membrane and to provide data showing that the selected waterproofing will adequately protect the concrete foundations against degradation from soil/groundwater conditions at the STP Units 3 and 4 site. The staff also asked the applicant to describe the final thickness of the membrane based on the physical properties of the selected material. In the Revision 1 response to **RAI 03.08.04-19** dated April 29, 2010 (ML101250162), the applicant states that the material used for the waterproof membrane will be a two-coat, color-coded Methyl-Methacrylate resin, which is an elastomeric “spray-on” membrane with physical properties that are specifically designed to cope with the rigorous requirements of below grade conditions. The applicant also states that in the detailed design phase, the specific material for the waterproof membrane will be selected and specified to protect the concrete foundations against degradation from soil /groundwater conditions. Regarding the thickness layer, the applicant states that the final thickness of all coats of the waterproofing membrane will be a nominal 0.305 cm (120 mils). The staff found this response adequate because it addresses the general questions regarding the description and qualification of the waterproofing membrane under the RB and CB. The proposed FSAR markups were included in Revision 6 of the FSAR. Thus, this issue is resolved.

2. *Testing for Friction Resistance*. **RAI 03.08.04-5, Item (b)** asked the applicant to

   (a) Provide the value of the coefficient of friction assumed between the concrete foundation and the mudmat with the chemical agent applied on top.

   (b) Provide the basis for the assumed value.

   (c) Describe how this value compares with the coefficient of friction assumed in the standard ABWR design in determining the factor of safety against sliding.

   The applicant’s response to **RAI 03.08.04-5** (ML092610377), states that:

   - The coefficient of friction of the waterproofing material will be determined by testing and will be sufficient to transfer site-specific SSE seismic loads. Because the waterproofing material is a COL license item and is only required to transfer loads for the site-specific seismic loads and soil conditions, the coefficient of friction of the waterproofing material may be different from those considered for the standard ABWR design.

- The coefficient of friction will be determined by a qualification program before procurement of the membrane material. The qualification program will:
  - o Be developed to demonstrate that the selected material will meet the waterproofing and friction requirements and will address at a minimum, the following:
    - ▪ chemical properties of the membrane material
    - ▪ physical properties of the membrane material
    - ▪ surface finish and preparation requirements
    - ▪ installation procedures necessary to achieve the required properties and coefficients of friction
  - o Include testing to demonstrate that the waterproofing requirements and the coefficient of friction that is required to transfer seismic loads for STP Units 3 and 4 have been met.
- Testing methods will simulate field conditions to demonstrate that the minimum required coefficient of friction is achieved by the structural concrete fill—the waterproof membrane structural interface.
- A technical report will document the basis for determining that the material meets the required friction factor and waterproofing requirements.
- Application procedures will be developed based on the results of the qualification testing to assure that the conditions and assumptions of the qualification tests are maintained during product application.

The staff issued follow-up **RAI 03.08.04-19, Items 4 and 5** asking the applicant to provide the following information for the proposed testing program:

(a) Describe the tests demonstrating that the waterproofing requirements and the coefficient of friction required to transfer seismic loads for STP Units 3 and 4 have been met.

(b) Describe the methods for testing that simulate field conditions to demonstrate that the minimum required coefficient of friction is achieved by the structural concrete fill-waterproof membrane structural interface.

(c) Provide documentation summarizing the basis for determining that the material will meet the friction factor and the waterproofing requirements.

In addition to the information in the response to **RAI 03.08.04-5** in the April 29, 2010, response to **RAI 03.08.04-19** (ML101250162) the applicant states that:

- The test program will be based on the test methods in ASTM D1894-06, "Standard Test Method for Static and Kinetic Coefficients of Friction of Plastic Film and Sheeting," which defines the coefficient of friction as the ratio of the force required to move one surface over another to the total force applied normal to those surfaces. The tests will be performed with the expected range of normal compressive stresses. The test fixture assembly will be designed to obtain a series of shear/lateral forces and the corresponding applied normal compressive loads. The test data will be generally represented by a best fit straight line whose slope is the coefficient of friction.

The RAI response also includes a proposed site-specific ITAAC (i.e., Table 3.0-13). However, the staff found that this proposed ITAAC does not provide any acceptance criteria for the minimum coefficient of friction. Subsequently, in the response to RAI 03.08.04-28 dated March 7, 2011 (ML110730066) the applicant identifies the acceptance criteria of 0.6 as the minimum coefficient of friction at the waterproofing membrane and structural concrete fill interface. As discussed in Item 4 below, the minimum static coefficient of friction was raised from 0.6 to 0.75 to meet the requirement for higher friction than provided at the gravel/soil interface. The staff found this response adequate because it provides the requested design information regarding the friction and testing requirements of the membrane under the RB and CB. The proposed FSAR markups were included in Revision 6 of the FSAR. Thus, this issue is resolved.

3. *Qualification and Application Procedures*. In **RAI 03.08.04-5, Item (c),** the staff asked the applicant to provide a detailed description of the type of waterproofing membrane proposed to be used, including operating experience with the use of such membranes at the site or elsewhere, and vendor or operating experience data demonstrating that the type of waterproofing membrane retains adequate water-retarding properties under aggressive soil conditions that are comparable to the site for long periods of time, without degrading. The applicant's response to **RAI 03.08.04-5** dated September 15, 2009 (ML092610377) states that the specific material for the waterproof membrane will be selected during detailed design. The waterproofing will be selected to assure that it is adequate to protect the concrete foundations against degradation from soil/groundwater conditions at the STP Units 3 and 4 site. **RAI 03.08.04-19, Item 3** asked the applicant to describe in detail the application procedures for all aspects of the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repairs. The applicant's response to **RAI 03.08.04-19** (ML101250162) states that the vendor for the waterproofing membrane materials has not been selected. However, the application procedures and aforementioned properties will be determined based on manufacturer's recommendations and the results of the qualification testing. The staff found this response adequate because the applicant has committed to qualify and apply the membrane in compliance with staff requirements regarding resistance against deterioration and long-term serviceability. The proposed FSAR markups were included in Revision 6 of the FSAR. Thus, this issue is resolved.

4. *Sliding Evaluation of RB and CB*. **RAI 03.08.04-19, Item 6** asked the applicant to describe in detail the site-specific sliding evaluation for the RB and the CB to demonstrate that the coefficient of friction—which is needed to maintain the minimum factor of safety against sliding—is available at all sliding interfaces between the structures and foundation soil. The applicant's response to **RAI 03.08.04-19**

(ML101250162) states that the site-specific sliding evaluation for the RB and the CB demonstrates that the coefficient of friction needed to maintain the minimum factor of safety against sliding is available at all sliding interfaces between the structures and foundation soil. The smallest coefficient of friction needed to maintain the minimum factor of safety against sliding is: RB = 0.47 and CB = 0.47. The soil friction capacity exceeds these values. In order to transfer the loadings from the foundation to the soil, the loads must be transferred from the reinforced concrete foundation through the structural concrete fill to the soil. For the concrete interfaces, ACI 349–97, Subsection 11.7.4.3 specifies the coefficient of friction (μ) for concrete placed against hardened concrete that is not intentionally roughened as 0.6. In NUREG–1503, Section 3.8.5, pages 3-56 and 3-57 state:

> "…a layer of gravel will be placed on the excavated foundation surface for the soil site … before pouring concrete and placing the waterproofing material. The treated foundation surface will increase the friction between the structural foundation and the supporting foundation surface…."

The placement of a gravel layer on the excavated soil surface for STP Units 3 and 4, followed by placement of the concrete fill on the gravel layer, will improve the friction between the concrete fill and the supporting foundation soil. When placed, the wet structural concrete fill will surround the aggregate particles at the gravel surface and will also penetrate some distance into the gravel layer. Once the concrete has hardened, the shear resistance to sliding will be governed by gravel to gravel strength (μ equals 0.75–0.84 within the gravel layer) on a plane below the concrete-to-gravel interface. The use of angular particle shapes in the gravel will create intimate contact with the soil surface to assure that the shear resistance at the underlying gravel-to-soil interface will be governed by the shear resistance of gravel or soil, whichever is less. The applicant's response provides a summary of the coefficient of friction for each of the sliding interfaces between the structures and foundation soil. At each sliding interface, μ provided is greater than μ required. Following a discussion during the NRC audit on March 14 through18, 2011, where staff noted that sliding will occur at the interface with the lowest friction coefficient (i.e., the concrete/concrete and concrete/membrane interface with a coefficient of 0.60) and thus not necessarily at the assumed gravel/soil interface, the staff requested the applicant to address this issue. In the Supplement 1 response to **RAI 03.08.04-19** dated April 25, 2011 (ML11119A077), the minimum required static coefficient of friction for concrete to concrete and concrete to waterproofing membrane was revised to 0.75. Furthermore, the response states that for the concrete-to-concrete interfaces, ACI 349–97, Sections 11.7.4.3 and 11.7.9 specify the coefficient of friction as μ = 1.0 for concrete that is placed against hardened concrete with the surface intentionally roughened. The applicant adds that this requirement for intentional concrete roughening will be added to the construction drawings; based on this requirement, the minimum static coefficient of friction for concrete to concrete and concrete to waterproofing membrane was revised to 0.75. Since the staff requested the applicant to add the concrete roughening requirement to the FSAR, the applicant has provided in the Supplement 2 response to **RAI 03.08.04-19** dated June 16, 2011 (ML11168A168), a corresponding markup for FSAR Subsection 3.8.6.1. The applicant also states that the coefficients of friction for all interfaces above the soil are raised to 0.75 or higher to ensure that any sliding would occur in the soil, which is limited by the shear capacity of soil with an equivalent coefficient of friction equal to or less than 0.7. The staff found this response adequate because it addresses the requested design information for the RB and CB, thus stating that the smallest friction resistance occurs at the interface gravel/excavated soil (0.70 static/0.47 dynamic), while the other interfaces (concrete to concrete; concrete to membrane; concrete to gravel; and gravel to gravel) provide higher friction values (0.75 static). The proposed FSAR markups were included in Revision 6 of the FSAR. Thus, this issue is resolved.

5. *Description of Structural Fill*. In **RAI 03.08.04-19, Item 7,** the staff asked the applicant to provide the specification and to describe the properties of the structural concrete fill below the RB and CB foundations. The applicant's response to this RAI (ML101250162) states that the structural concrete fill below the RB and CB foundations will be unreinforced normal weight concrete with a minimum compressive strength (f'c) of 20.69 MPa (3,000 psi). The staff found this response adequate because it provides the requested design information regarding the concrete fill under the RB and CB structures. In addition, the proposed concrete is considered acceptable because the concrete is stronger than the underlying foundation subgrade. The proposed FSAR markups were included in a subsequent revision of the FSAR. Thus, this issue is resolved.

6. *Testing for Chemical Resistance*. In **RAI 03.08.04-28, Item 1,** the staff asked the applicant to describe the requirements of the below-grade conditions, how the chemical properties of the membrane meet these requirements, and to also include in the FSAR a description and thickness of the material used for the waterproof membrane. In the Revision 1 response to **RAI 03.08.04-28** dated March 7, 2011 (ML110730066), the applicant states that the waterproofing membrane is applied on the structural concrete fill, and thus pertains to the load path between the basemat and soil, thus requiring both capacities—water retaining and friction— to be maintained at the interface over the life of the plant. The applicant adds that the waterproofing membrane will be tested under conditions that simulate actual exposure according to ASTM C267-01, "Standard Test Methods for Chemical Resistance of Mortars, Grouts, and Monolithic Surfacings and Polymer Concretes," for its resistance to the concrete mix chemistry, the actual backfill material chemistry, and ground water chemistry found on site. Furthermore, the additional testing of the waterproofing membrane's ability to resist the chemical reagents as specified through accelerated aging will be done per ASTM G114-07, "Standard Practices for Evaluating the Age Resistance of Polymeric Materials Used in Oxygen Service." The results will be used to show that there is a negligible change in the material properties or composition for at least the 60-year life of the plant. The COL FSAR markup for the description and thickness of the membrane material as well as the requirement to test for resistance to the concrete mix chemistry, the actual backfill material chemistry, and ground water chemistry found on the site are included with this response. The staff reviewed the response and determined that it adequately addresses the issue, because it provides and clarifies the requested design information regarding the testing and chemical resistance of the membrane. The proposed FSAR markups were included in Revision 6 of the FSAR. Thus, this issue is resolved.

7. *Testing for Mechanical Resistance*. In **RAI 03.08.04-28, Item 2,** the staff asked the applicant to describe and to explain how the waterproofing requirements are established and how they will be tested to demonstrate that the selected membrane is adequate considering long-term performance. The applicant was also asked to update the FSAR as appropriate. In the response to **RAI 03.08.04-28**

(ML110730066), the applicant states that the evaluation and mechanical testing of the waterproofing membrane consider the following:

- Testing will be in accordance with ASTM D5385-93, “Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes” to a hydrostatic pressure of 0.69 MPa (100 psi).

- The margin provided by the test pressure of 0.69 MPa (100 psi) over the design pressure of 0.28 MPa (40 psi) (hydrostatic pressure at a depth of the RB foundation [27.43 m (90 ft)]), along with the results from accelerated age testing described in Item 6 above, will ensure the resistance over its intended lifetime of 60 years.

- Additional testing may be performed to demonstrate adequate performance under applicable mechanical conditions, including pressures from the backfill and foundation bearing.

- Test conditions will simulate the environmental pressures at the walls and the base level.

The staff agreed with the proposed testing program because it subjects the membrane to the actual mechanical and environmental conditions at the site. Furthermore, the response incorporates into Revision 6 of the FSAR the requested design information. Therefore, this issue is resolved.

8. <u>*ITAAC Description*</u>. In **RAI 03.08.04-28, item 3,** the staff asked the applicant to provide, and to incorporate the information listed below in a proposed ITAAC in the response to RAI 03.08.04-19.

   - Clarify whether the thickness of the specimen tested will be the same as that used for the membrane.

   - As described below, clarify which value of the coefficient of friction will be used for the acceptance criteria of the ITAAC:

   The proposed acceptance criterion for the ITAAC specifies that:

   > “A report exists and documents that the waterproof system (mudmat-waterproofing-mudmat) has a coefficient of friction to support the analysis against sliding.”

   The response to RAI 03.08.04-19 states that the minimum coefficient of friction needed to maintain the minimum factor of safety against sliding for the RB and the CB is 0.47, whereas in Table RAI 03.08.04-19a of the same response the minimum coefficient of friction provided at the structural concrete fill and waterproofing membrane interface is 0.6.

   - Include in the FSAR the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface

In the response to **RAI 03.08.04-28** (ML110730066), the applicant states that the thickness of the membrane to be tested will be the same as the actual nominal thickness used for the membrane, and has revised ITAAC Table 3.0-13 in COL application Part 9, accordingly with the acceptance criterion for the minimum coefficient of friction of 0.6. Furthermore, a proposed COL application markup indicates that the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface is 0.6. As discussed in Item 4 above, the minimum static coefficient of friction was raised from 0.60 to 0.75 in a later revision of the FSAR. The staff found this response adequate because it provides and incorporates into the FSAR the requested design information. The proposed FSAR markups were included in Revision 6 of the FSAR. Therefore, this issue is resolved.

9. _Friction Between Gravel and Soil_. The applicant states in Item (6) in the response to **RAI 03.08.04-19** (ML101250162) that the coefficient of friction provided at the interface of the bottom of the gravel layer and soil to be the smaller of 0.6 and shear capacity of the soil. The applicant also states that the soil capacity exceeds the value of 0.47 that is needed for maintaining the minimum factor of safety against sliding of the RB and CB. Therefore, in **RAI 03.08.04-28, Item 4,** the staff asked the applicant to provide a clarification concerning the minimum coefficient of friction available at the bottom of the gravel-and-soil interface based on site-specific soil properties and to explain how it is determined. In the Revision 1 response to this RAI dated March 7, 2011 (ML110730066), the applicant states that the bottom of the gravel-and-soil interface is governed by the friction forces that develop under the RB and CB and that depend on the properties of the existing materials under the buildings. The coefficient of friction and cohesion value for gravel and soil interfaces mobilize the full soil shear strength. The static resistance however requires adjustment to account for seismic cyclic loading. Based on experimental data from Makdisi and Seed (Makdisi, F.I. and Seed, H.B. 1978, "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformations," Journal of the Geotechnical Engineering Division, ASCE Vol. 104, No. GT7, p 849-867), the recommended adjustment factors are: 67 percent for sand friction and 80 percent for cohesion resistance. Using the soil friction angles and cohesion values given in COL FSAR Table 2.5S.4-37B and Table 2.5S.4-38B for RB and CB respectively, the coefficient of friction at the interface between the bottom of gravel and sandy soil has a value of 0.70 (=tangent (phi)), and the clay layers have a cohesion value of 162.8 kPa (3.4 ksf) for static loading. Applying the aforementioned adjustment factors for cyclic loading, these values reduce to 0.47 and 130.2 kPa (2.72 ksf). The static gravel to gravel coefficient of friction is given as 0.75 to 0.84, and is thus higher than the sand layer. For any embedded building, resistance against sliding is provided by both: lateral soil pressure and friction. In order to obtain the prescribed sliding safety factor for the CB, using a friction coefficient of 0.47 resulted in partially activating the lateral passive pressure. Achieving a safety factor of 1.11 against sliding for the Unit 4 RB, which is founded on top of clay layers, required developing 40 percent of the available lateral passive pressure, while cohesion contributed 50 percent to the sliding resistance. This result is conservative for the Unit 3 RB, which is only partially founded on clay. Asked about the justification for using the cyclic loading adjustment factors at the STP site, the applicant responded in Supplement 1 to the response to **RAI 03.08.04-28** dated April 25, 2011 (ML11119A077), that Makdisi and Seed show a range of maximum cyclic shear strains of 0.1 to 1 percent as indicative of the range of maximum cyclic shear strains they associate with the availability of 80 percent or

more of the static undrained strength of clayey soils subjected to such maximum cyclic shear strains. COL FSAR Figure 2.5S.2-47 indicates that the mean maximum cyclic shear strains are less than 0.07 percent in the soils within the approximately 30.5 m (100 ft) maximum depth range occupied by the Category 1 buildings of Units 3 and 4, and therefore under the strength-reduction-causing strain range. Regarding the adjustment of the coefficients of friction of the sandy soils, the applicant responded that under shear displacement, dense sands first exhibit their peak shear resistance, and then move toward a residual shear resistance. This effect can be observed in the triaxial shear test results (Figure 2.5.4-32) in the Units 1 and 2 updated FSAR (Revision 15), as well as direct shear test results reflected in a report from the Federal Highway Administration (U.S. Department of Transportation, Federal Highway Administration, 2006. "A Laboratory and Field Study of Composite Piles for Bridge Substructures," Report Number FHWA-HRT-04-043, Chapter 3). These test results indicate that it is reasonable to assign a friction coefficient after movement of 67 percent of the static values. In this evaluation, the staff noted that at the transition between natural soil and gravel fill, lateral soil resistance is provided by friction, cohesion or a combination of both, whereby friction resistance is stronger than cohesion; that both values are determined from static soil testing and that the design values are reduced to account for dynamic loading effects (i.e., for seismic). Stability analyses performed using these soil parameters indicate that to meet the required sliding safety factors, only a fraction of the available passive lateral soil resistance at the embedded walls had to be activated, thus demonstrating that additional safety margins are still available. Therefore, the staff concluded that the applicant has adequately addressed the issue and has provided the requested design information for the RB and CB. The proposed FSAR markups are included in Revision 6 of the FSAR. Thus, this issue is resolved.

As described above, the applicant's responses adequately address the staff's requests for additional information and clarification in **RAI 03.08.04-5**, and followup **RAI 03.08.04-19** and **RAI 03.08.04-28**. Therefore, these RAIs are closed.

- COL License Information Item 3.24 Site Specific Physical Properties and Foundation Settlement

<u>Foundation Settlement</u>

In FSAR Subsection 3.8.6.2, "Site Specific Physical Properties and Foundation Settlement," the applicant refers to FSAR Subsections 3H.6.4.2 and 2.5S.4 to address COL License Information Item 3.24, which requires that the physical properties of the site-specific subgrade medium be determined and the settlement of foundations and structures, including seismic Category I structures, be evaluated. In FSAR Subsection 2.5S.4.10.4, the applicant provides a settlement evaluation of the structures and concludes that for all evaluated structures, the resulting tilt was within the acceptable limit of 1/300. Since it was not clear what limit was used for the design of the standard plant structures, in **RAI 03.08.05-3** and follow-up **RAI 03.08.05-5** the staff asked the applicant to confirm that the angular distortions/tilts due to differential settlement determined for the STP site are enveloped by the corresponding values used in the ABWR DCD, and to provide detailed information regarding the items listed below. The applicant was also requested to update the FSAR to clearly state how this COL information item is addressed.

1. *<u>Acceptable Tilt Values</u>*. Since it was not clear whether the ABWR DCD standard plant structures were designed using the 1/300 tilt criteria, in **RAI 03.08.05-3** the staff asked

the applicant to confirm that the angular distortions/tilts due to differential settlement determined for the STP site are enveloped by the corresponding values used for design of ABWR DCD standard plant structures, and if not, to justify the acceptability of angular distortions determined for these structures for the STP site. In the Revision 1 response to **RAI 03.08.05-3** dated April 14, 2010 (ML101090143), the applicant states that the ABWR DCD does not contain any criteria for settlement-related angular distortions/tilts. The response also concludes that the generally accepted engineering standard of practice is to allow for an angular distortion of no greater than 1/300, and that according to FSAR Subsection 2.5S.4.10.4 angular distortions of Category I buildings can be expected to be smaller than 1/800, thus well under the generally accepted limit of 1/300. Nevertheless, in this response, the applicant proposes that an angular distortion value of 1/500, as recommended by Bjerrum (Bjerrum, L., 1963, "Allowable Settlement of Structures", Proceedings European Conference on Soil Mechanics and Foundation Engineering, Vol. III), is more conservative and will be used as an acceptance criterion for the Category I structures. Furthermore, the applicant states that it is expected that by the time building superstructures are ready to receive equipment and/or piping, a significant amount (i.e., more than half) of foundation settlements are expected to have already taken place; resulting in the angular distortions/tilts values to be half of those presented in the COL application. Tilt angles will therefore be well within the criterion of the 1/500 limit. Tilt of the seismic Category I structures of the STP Units 3 and 4 is therefore acceptably low. A corresponding markup to revise FSAR Subsections 2.5S.4.10, 2.5S.4.11 and 2.5S.4.13 was included in the response. Even though the DCD tilt limit considered in design is not explicitly given, the staff agreed that limiting the maximum tilt, measured between center and edge of the mat foundation, to 1/500 is in accordance with current engineering practice, and thus is also representative of the DCD design criteria. The proposed FSAR markups were included in Revision 4 of the FSAR. Thus, this issue is resolved.

2. *Stresses Due to Tilting*. In **RAI 03.08.05-5, Item 1**, the staff asked the applicant to provide a quantitative evaluation to demonstrate that the ABWR standard plant structures would not be adversely stressed as a result of the tilt. In the response to **RAI 03.08.05-5** dated September 15, 2010 (ML102630145), the applicant states that as noted in the response to **RAI 03.08.05-3** (ML101090143), the induced stresses due to flexibility of the structure/basemat and the supporting soil are accounted for through the use of the FEA in conjunction with the use of appropriate springs representing the foundation soil. Regarding the induced stresses due to rigid body tilt the applicant states that the associated lateral loads are about 0.2 percent of gravity, and as such, is negligible compared to the about 30 percent of gravity lateral loads from SSE, and that compared to the total design loads the contribution of tilting appears to be even smaller and thus of no significance for seismic Category I structures. In this evaluation, the staff noted that actual tilt angles for the standard buildings are close but exceed the limit angle for the CB (1/400 and 1/450), see Table 2.5S.4-42, "Estimated Foundation Settlements." The small amount of the exceeded angle is acceptable as the tilt angle limits are more of an empirical nature, and the estimated tilt values are based on the conservative assumption of flexible foundations; further, construction sequence will mitigate some of the differential settlements, thus more realistic values are expected to be one half the estimated values. As the standard structures under consideration are characterized by having rather high stiffness, the additional loading results primarily from lateral projection of gravity loads and not from member distortions, the staff agreed that the additional stress level induced by the

rigid body tilt is negligible in comparison with other load cases and therefore is not relevant. Therefore, this part of the RAI is resolved. No COL application revision resulted from this part of the RAI.

3. *Differential Settlements in DCD.* In **RAI 03.08.05-5, Item 2**, the staff asked the applicant to provide a quantitative evaluation to demonstrate that the maximum differential settlements for the ABWR standard plant structures at the STP site would be within the values accounted for in the design of these structures. In the Supplement 1 response to **RAI 03.08.05-5** dated November 17, 2010 (ML103230128), the applicant states that the certified design of the ABWR standard plant structures considers a range of soil properties varying from rock to soft soil. As described in responses to **RAI 03.07.01-7** dated September 15, 2009 (ML092610377), and **RAI 03.07.01-20** dated February 10, 2010 (ML100550613), the STP Units 3 and 4 site-specific soil profiles yield subgrade modulus comparable to, or higher than, those considered for the ABWR certified design. Also, referring to the response to **RAI 03.07.01-28** dated March 7, 2011 (ML110730067), it is shown that the induced stresses within the foundation are rather insensitive to variations in the subgrade modulus. Considering the above, the ABWR certified design of the standard plant structures can adequately accommodate the maximum differential foundation settlements within the boundary of each foundation at the STP site. The maximum calculated ratio of differential foundation settlements (between adjacent points in the mat finite element model) within the boundary of each of the Standard Plant seismic Category I structures due to non-transient loads are as follows:

   (a) RB/RCCV foundation = 1/1697
   (b) CB foundation = 1/928
   (c) DGFOT = 1/1700

   The above maximum calculated differential foundation settlements are based on the consideration of both Winkler and pseudo-coupled methods of analysis (Winkler Method for foundation soil springs, per Coduto, Donald P., “Foundation Design Principles and Practices”, Second Edition, Prentice Hall: New Jersey, 2001). The response makes reference to the Supplement 1 response to **RAI 03.08.05-4** dated November 11, 2010 (ML103230128), for additional information regarding modeling the subsoil with soil springs, and states that COL FSAR Sections 3H.1.6 and 3H.2.6 will be revised to include the above settlement values for the RB and CB, and that the settlement value for the DGFOT will be included in the FSAR markup, along with other design information for the tunnels, to be provided with the Supplement 1 response to **RAI 03.08.04-30** (ML110770440). The staff’s evaluation noted that the applicant is using industry accepted procedures to simulate the static soil structure interaction by using both Winkler and pseudo-coupled soil springs. This approach is a recognized method for estimating basemat deformations. The applicant presents the maximum settlement ratios as the quotient between the differential nodal displacement and the respective distance between two neighboring nodes. These values are thus not directly comparable to the values given in FSAR Subsection 2.5S.4.10.4, which are based on differential settlements between center and edge of foundation. The differential soil settlement values (between center and edge of the foundation) given in FSAR Table 2.5S.4-42 are obtained from applying a vertical uniform load to a flexible foundation and as such represent conservative estimates that could be significantly reduced with increased foundation and superstructure stiffness. This effect can be quantified with the factor K_r mentioned in FSAR Subsection 2.5S.4.10.4, which shows that differential settlements may be negligible for stiff structures. Considering that the DCD does not provide numerical values of acceptable differential settlements, the staff considered that the estimated distortion ratios provide reasonable acceptable values of differential settlements for the above structures for the STP site. The proposed FSAR markups are included in Revision 6 of the FSAR. Thus, this issue is resolved.

As described above, the applicant's responses adequately addressed the staff's requests for additional information and clarification in **RAI 03.08.05-3** and follow-up **RAI 03.08.05-5**. Therefore, these RAIs are resolved.

<u>*Supplemental Information*</u>

A. Evaluation of Design Issues Related to All Site-Specific Seismic Category I Structures.

FSAR Section 3H.6 provides detailed information related to the design of site-specific Category I structures. During the review, the staff noted that the applicant did not include sufficient information about design and stability evaluations of foundations. In order to determine whether the foundations for site-specific seismic Category I structures meet the acceptance criteria in SRP Section 3.8.5, in **RAI 03.08.05-1** and follow up **RAIs 03.08.05-2** and **RAI 03.08.05-4**, the applicant was asked to include in the appropriate subsections of FSAR Section 3H.6 information about design and stability evaluation of foundations of site-specific seismic Category I structures including the methodology used for design of the foundations, how differential settlement is considered in the design of foundations, calculated static and dynamic bearing pressures, method of evaluation of stability of foundations including consideration of buoyant forces, coefficient of friction used in foundation evaluation including its basis, consideration of active and passive pressures on foundation walls for stability evaluation as well as design of foundation walls, and the results of foundation design and stability evaluation. The applicant's response to the above questions is discussed in the following paragraphs.

A.1 <u>*Lateral Soil Pressures:*</u>

In **RAI 03.08.05-1** and later in **RAI 03.08.05-2** the staff asked the applicant to consider the active and passive pressures on foundation walls for stability evaluation as well as for the design of the foundation walls. In the response to **RAI 03.08.05-1** dated September 22, 2009 (ML092660655), the applicant states that the lateral pressure calculations on foundation walls are performed as described in COL FSAR Subsection 3H.6.6.2, and the analysis and design results of lateral pressures on foundation walls would be provided in a supplemental response to **RAI 03.07.01-13**. The applicant subsequently provided the information in several responses which were discussed and evaluated in detail in various subparts in SER Subsection 3.8.4.4. Lateral soil pressures on standard plant structures are discussed under the subpart C, "Lateral Soil Pressures on Standard Plant Structures," in SER Subsection 3.8.4.4.1. Lateral soil pressures on site-specific structures are discussed in SER Subsection 3.8.4.4.2. The general issues pertaining to lateral soil pressures on all site-specific structures are discussed in SER Subsection 3.8.4.4.2, under the Subpart A.4 (item e), "Lateral Soil Pressures." Lateral soil pressures on individual site-specific structures are discussed in SER Subsection 3.8.4.4.2, under the Subpart B.1.4 (item f) for UHS/RSW pump house and RSW tunnels, Subpart B.2.4 (item b) for DGFOSV, and Subpart B.3.4 (item b) for DGFOT. Impact on lateral soil pressures due to the use of SM in SSI analysis is discussed in SER Subsection 3.8.4.4.4, "DNFSB Issue: Resolution of Issues with Subtraction Method of Analysis."

A.2 Differential Settlements

In **RAI 03.08.05-1** the staff asked the applicant to describe how differential settlements are considered in the design of foundations including the magnitudes of the differential settlements and how the differential settlements were used in the analysis of these structures. In the September 22, 2009, response to **RAI 03.08.05-1** (ML092660655), the applicant states that the differential settlements will be determined based on detailed settlement calculations considering time rate of settlements and construction sequence, and additional information on settlements is provided in the September 21, 2009, response to **RAI 02.05.04-30** (ML092710096). The applicant also states that the impact of the differential settlements on the member design forces will be accounted for by including the applicable differential settlements in the analysis of these structures. The staff noted that the information provided in the response to **RAI 02.05.04-30** was of general nature, and did not provide any information regarding the magnitudes of the differential settlements, and how the differential settlements were considered in the analysis of site-specific Category I structures. Therefore, in **RAI 03.08.05-2** the staff asked the applicant to clearly describe the magnitudes of differential settlements considered for design of site-specific seismic Category I structures, and also to explain how differential settlements were accounted for in the analysis of these structures. In the Revision 2 response to **RAI 03.08.05-2** dated May 13, 2010 (ML101340651), the applicant states that there are three different effects of settlements which need to be considered in design of the structures, as discussed in the following paragraphs.

a. Rigid Body Angular Distortions/Tilts. COL FSAR Subsection 2.5S.4.10 presents conservatively calculated angular distortions/tilts based on conservatively estimated differential settlements of each structure. The calculation assumed a perfectly flexible structure with no applied reduction due to buoyancy or structural rigidity. As explained in the response to **RAI 03.08.05-3** (ML101090143), the calculated tilt values are acceptable and no additional consideration is needed in the design of structures for these tilt values.

b. Differential Settlement due to Flexibility of Structure/Basemat and Supporting Soil. Settlements due to flexibility of structure/basemat and supporting soil induce stresses within the structure. In the analysis and design of the site-specific seismic Category I structures, this effect is accounted for through the use of FEA in conjunction with foundation soil springs. FEA representation of the structure accounts for the flexibility of structure/basemat, and the soil springs with their stiffness based on subgrade modulus, which is a function of the foundation settlement, account for the flexibility of the supporting soil medium. The information on the analysis and design of the site-specific structures was provided in the Supplement 2 response to **RAI 03.07.01-13** (ML100050225).

c. Differential Settlement Between Buildings. Differential settlements due to structural backfill, loading of other structures and consolidation of clay layers result in differential settlements between the buildings and angular distortions/tilts. These differential settlements and angular distortions/tilts will impact the design of commodities and tunnels running between the buildings and the seismic gaps among the adjacent buildings. The magnitude of these impacts will be minimized by delaying final connections to a time when the majority of the differential settlements and angular distortions/tilts have already taken place. The timing for the final connection of such commodities and tunnels will be established based on the time-rate of settlement analyses described in the response to **RAI 02.05.04-30**

(ML092710096). The total movement for design of commodities and tunnels running between buildings and seismic gaps of the adjacent buildings are determined considering the differential settlements and angular distortions/tilts from the time-rate of settlement analysis and any additional movement during a seismic event. Based on the results of the time-rate of settlement analyses, the differential movements for the design of commodities running between the site-specific seismic Category I structures (RSW piping tunnels and DGFOT) and the adjoining structures (CB, RSW pump house, and DGFOSV) as shown in Table 3H.6-15 in the May 13, 2010, response to **RAI 03.08.4-25** (ML101340651). This table was revised later in Revision 6 of the FSAR to include the gaps between the DGFOT and RB.

The staff's assessment of the applicant's response is given below:

(a) <u>Rigid Body Angular Distortions/Tilts</u>

The applicant's response refers to COL FSAR Subsection 2.5S.4.10 for conservatively estimated angular distortions/tilts. In the response to **RAI 03.08.05-3** (ML101090143), the applicant explains why the calculated angular distortions/tilts may be considered acceptable. Justifying an acceptable tilt value of 1/500 for the seismic Category I structures at STP, the applicant references several published materials that were based on observations of cracking and other structural damage of commercial structures. However, the information the applicant referred to did not provide any estimates of the amount of additional stress that may be imposed on these structures as a result of the tilt. Therefore, in **RAI 03.08.05-4 Item 1,** the staff asked the applicant to provide an estimate of the amount of additional stresses that may be imposed on the structure as a result of the tilt. In the response to **RAI 03.08.05-4** dated September 15, 2010 (ML102630145), the applicant evaluates the induced stresses in site-specific seismic Category I structures resulting from rigid body tilt that considered a maximum allowed rigid body tilt of 1/500. The applicant states that under a maximum rigid body tilt of 1/500, the structure will be subjected to additional lateral loads equal to 0.002 times the gravity loads (or 0.2 percent of gravity loads). All site-specific seismic Category I structures qualify for a site-specific SSE (i.e., 0.13g modified RG 1.60 spectra). Conservatively assuming no in-structure amplification, the minimum lateral seismic load for the design of site-specific seismic Category I structures is equal to 0.13 times the gravity loads (or 13 percent of gravity loads). Therefore, for STP site-specific seismic Category I structures, the induced stresses from a 1/500 rigid body tilt at about the E-W or N-S axis will be less than 1.5 percent (i.e., 0.2/13 = 0.015) of the stresses from design lateral seismic loads due to N-S or E-W excitations, respectively. The applicant adds that the induced stresses resulting from a maximum rigid body tilt of 1/500 compared to the total governing design stresses (i.e., stresses due to all loads within the governing load combinations, such as dead load and live load in combination with seismic loads) will be far less than 1.5 percent of the governing design stresses. Thus, the induced stresses from a maximum rigid body tilt of 1/500 are negligibly small, and a maximum rigid body tilt of 1/500 is therefore considered acceptable without any explicit evaluation. The staff agreed with the applicant's evaluation and found this response adequate because with the additional stress induced in the structure, the maximum allowable tilt is shown to be insignificant. Furthermore, the applicant includes a site-specific ITAAC (Table 3.0-15, "Settlement") to verify that the field measurement of the settlement of structures 3 months before fuel load does not exceed maximum allowable tilt of 1/600, which is more conservative than the tilt value the structures are evaluated for. Therefore, this item is resolved. No revision of the FSAR was submitted regarding this issue.

(b) Differential Settlement due to Flexibility of Structure/Basemat and Supporting Soil

In the Revision 2 response to **RAI 03.08.05-2** (ML101340651), the applicant states that the effect of settlement resulting from the flexibility of the structure/basemat and supporting soil is accounted for through the use of the FEA, in conjunction with the foundation soil spring. Because the foundation subgrade modulus may vary over a wide range across the foundation footprint, it was not clear whether the applicant's analysis had considered the horizontal variation of the foundation subgrade modulus over the entire area of the foundation. Therefore, in **RAI 03.08.05-4 Item 2,** the staff asked the applicant (1) to describe how the variation of the subgrade modulus over the foundation footprint was considered in the analysis; and (2) to list in the FSAR the values of maximum differential foundation settlements for which each seismic Category I structure is designed. In the Supplement 1 response to **RAI 03.08.05-4** (ML103230128), the applicant provides the following:

(1) Variation of the Subgrade Modulus over the Foundation Footprint

The applicant states that a variation of the subgrade modulus within the foundation footprint may arise from the following two sources:

- Location Within the Foundation Boundary. The most common approach in the design of mat foundations is to use the Winkler method to account for SSIs. This approach assumes that soil has a uniform subgrade modulus under the entire mat, and the springs representing the soil are considered to be linear and act independently. This method calculates the uniform subgrade modulus, Ks, as the average subgrade modulus. Referring to DCD Tier 2, Subsection 3H.1.5.2, the applicant states that the certified design is premised on the uniform subgrade modulus, where the uniform subgrade modulus is multiplied by the mat nodal point tributary area to compute the spring stiffness value at a given node. Using the Winkler method, a uniformly loaded flexible mat foundation will exhibit a uniform settlement under the entire mat. In reality, however, due to overlapping stress bulbs beneath the foundation, the springs representing the soil are not independent of each other and the settlement at the center of the mat will therefore be greater than the settlement along the mat edges. To account for this effect, a coupled method may be used where the dependence of adjacent soil springs is represented by additional springs. Because implementing this approach is rather complicated and may require the development of custom software, the use of alternate methods such as the pseudo-coupled method has yielded acceptable results. In the pseudo-coupled method (described in Section 10.2 of the following reference: Coduto, Donald P., *Foundation Design Principles and Practices*, Second Edition, Prentice Hall: New Jersey, 2001), different subgrade modulus values are assigned to different areas (zones) of the mat foundation. Initially, and consistent with the DCD certified design, the design of the site-specific seismic Category I structures was carried out using the Winkler method, where the average subgrade modulus for each foundation was calculated using the settlements for nine points of each foundation presented in COL FSAR Table 2.5S.4-42. Subsequent to the discussions with NRC in the August 4, 2010 public meeting, the site-specific seismic Category I structures were re-analyzed and re-designed considering both the Winkler and the pseudo-coupled methods. The staff considered the pseudo-coupled method of consideration for foundation springs to be acceptable, because this method is recognized as an acceptable method in the industry (e.g., ACI 336.2R-88, "Suggested Analysis and Design Procedures for Combined Footings and Mats") and in the published literature on the subject.

- Variation in Soil Profile. The large foundation area of the individual STP Units 3 and 4 structures creates a stress bulb that extends to a considerable depth below the bottom of each foundation. Thus, the settlement of the foundation at any location is determined by the combined compression of many soil layers below that location (or point). Variations in the properties or thickness of individual layers do not cause significant differences in the computed settlement and the modulus of subgrade reaction between corresponding points on the foundation. For example, this conclusion is supported by using the settlement values under the building weight alone of the Unit 3 RB to compute the modulus of subgrade reaction at the nine locations on the foundation (shown in COL FSAR Table 2.5S.4-42). The resulting modulus of subgrade reaction values on the north edge and north corners of the Unit 3 RB foundation are not significantly different from the values at corresponding locations on the south edge and south corners of that foundation. This finding is in spite of the fact that the uppermost soil layers beneath the north edge and north corners of the foundation of the Unit 3 RB are different from those in the remainder of the foundation area (as described in COL FSAR Table 2.5S.4-37A). Therefore, variations in the modulus of the subgrade reaction as a result of variations in the soil layers from point to point beneath the large foundations of the STP Units 3 and 4 structures are insignificant in terms of their influence on the design of the basemats for these structures. During the evaluation the staff noted that for purposes of the analysis, the soil elasticity is represented by its modulus of subgrade reaction, which in turn is obtained from the settlement values that are the result of the induced stress field in the supporting soil. Such values result from applying accepted geotechnical methods. These values show only a moderate variability across the foundation footprint and thus justify, in this case, the practice of using the average values as representative for design purposes. Therefore, staff found this response adequate and the issue is resolved. No revision of the FSAR resulted from the resolution of this issue.

(2) Maximum Computed Differential Foundation Settlements

In the Supplement 1 response to **RAI 03.08.05-4** (ML103230128), the applicant states that the maximum computed differential foundation settlements for the site-specific seismic Category I structures were calculated considering both the Winkler and the pseudo-coupled methods. The maximum calculated ratios of differential foundation settlements (defined as the quotient of the differential displacements between adjacent nodal points of the discretized foundation mat and the distance between the points) within the footprint of each site-specific seismic Category I structure due to non-transient loads are as follows:

i. UHS basin foundation = 1/860
ii. RSW pump house foundation = 1/1200
iii. RSW piping tunnel foundation = 1/3900
iv. DGFOSV foundations = 1/4860

The staff found the response acceptable, because the applicant has provided the maximum differential settlement ratios for the foundations that are the actual differential settlements the foundations are designed for.

The responses included markups for FSAR Subsection 3H.6.6.4 ("Foundations"), 3H.6.7 ("DGFOSV"), and 3H.6.9 ("References"). The proposed FSAR markups are included in Revision 6 of the FSAR. Therefore, the issue of differential settlements is resolved.

(c) Differential Settlement Between Buildings

Differential settlements between structures and angular distortions/tilts will impact the design of commodities and tunnels running between the buildings and the seismic gaps among the adjacent buildings. This topic is discussed in the following paragraphs addressing the effects of differential movements at the interfaces between the tunnels and the adjoining buildings.

A.3 Interface Between RSW Tunnels and DGFOT with the RB, CB and UHS/RSW Pump House

In FSAR Subsections 3.8.5.8.1 and 3.8.5.9, the applicant states that seismic gaps between the DGFOT and the adjoining RB and DGFOSV, and seismic gaps between the RSW piping tunnels and the adjoining CB and RSW pump house are determined considering settlement and tilts obtained from the time rate of the settlement analysis that accounts for the construction sequence, seismic movements from the seismic analysis, and translations and/or rotations from the sliding and overturning stability evaluations. Detailed descriptions and evaluations of this issue are in SER Subsection 3.8.4.4.4 under Topic B.4, "Design Information Relating to the Interface between the RSW Tunnels, DGFOT, and Adjoining seismic Category I Structures."

A.4 Stability Evaluation

The foundations of the standard plant structures, site-specific seismic Category I structures, and non-Category I structures with the potential to interact with Category I structures must be designed so that the buildings are stable and have adequate factors of safety. Detailed descriptions and assessments of stability evaluations are in SER Section 3.8.4.4. For the stability evaluation of standard plant structures, see SER Subsection 3.8.4.4.1 under Subpart A.2 Item 3, "Stability Evaluation." SER Subsection 3.8.4.4.2 under Subpart A.5.2, "Stability Evaluations," describes the stability evaluation of seismic Category I structures. For the stability evaluation of non-Category I structures, see SER Subsection 3.8.4.4.3 under Part C, "Stability Evaluation of Non-Category I Structures." The stability evaluation of individual site-specific structures is presented under Part B, "Evaluation of Design Issues Related to Each Specific Category I Site-Specific Structure." located below.

1. Construction Sequence

During its review of foundations, the staff referred to SRP Section 3.8.5 and asked the applicant to describe how the structural design of foundations accounted for the effects of construction sequence, including differential settlements. In the Supplement 4 response to **RAI 03.08.04-18** dated August 28, 2012 (ML12249A035), the applicant provides a markup of FSAR Subsection 3.8.5.10, "Construction Sequencing for seismic Category I Foundations," and describes the construction sequence planning in order to ensure that construction loading does not result in excessive stresses on the foundation mat or on the superstructure. The elements of the applicant's proposed construction sequence are as follows:

- Construction should proceed so that major walls at the lowest level, those providing foundation mat stiffness, are constructed across essentially the entire foundation before applying loads from the walls and slabs above.

- Loads should be uniformly applied to the foundations.

- The overall foundation tilt would remain within 1/600.

The applicant also states that the construction specifications will include the following requirements:

- The concrete placement for the superstructure will be such that the superstructure is erected uniformly considering the following:
    - Concrete pours for major walls will be limited to the lesser of 6.1 m (20 ft) or to the floor above, until all of the major walls at that elevation are poured.
    - Concrete pours for major floor slabs will be essentially completed for the entire floor before concrete pours are started for floor above.

- The RSW pump house/UHS foundation is different from the other structures in that the top of the foundation for the UHS is 9.78 m (32 ft) above the RSW pump house foundation. For the RSW pump house/UHS foundation, the following sequence will be specified:
    - Excavate the RSW pump house/UHS to the bottom of the UHS foundation.
    - Place the UHS foundation concrete to a construction joint within 3.05 m (10 ft) of the junction with the RSW pump house.
    - Drive the sheet piling along the wall of the RSW pump house that is adjacent to the UHS and excavate to the bottom of the RSW pump house foundation.
    - Place the RSW pump house foundation concrete.
    - Place the RSW pump house concrete walls up to the UHS foundation level.
    - Complete the concrete placements for the UHS foundations and the RSW pump house slabs at the top of the UHS foundation level.
    - For the remaining portions of the UHS basin and the RSW pump house above the UHS basemat level, the concrete pour of the major walls will be limited to the lesser of about 6.1 m (20 ft) or to the floor above, until all of the major walls at that elevation are poured.

- For the buried tunnels, the following sequence will be specified:
    - Construct the tunnels uniformly and level by level to a construction joint within about ten feet of the junction with the terminating structure.
    - After placing backfill around and above each tunnel, place the last tunnel segment adjacent to the terminating structure.

Per FSAR Subsection 2.5S.4.5.4.5.4, major structural foundations are monitored for movement during construction. A Settlement Monitoring Program will be established to monitor settlement and angular distortion (tilt) to verify foundation performance and to provide for corrections.

The staff noted that the proposed construction sequence provides reasonable assurance of avoiding non-uniform load distributions during construction, thereby minimizing the risk of excessive differential settlements that could result in overstressing the foundation and superstructure. Furthermore, monitoring the settlement during construction will assure that deformations remain within design limits. The staff found the proposed construction specifications acceptable, because they meet the intent of SRP Section 3.8.5 to control the stresses in the foundation mat and superstructure in the construction sequence. The FSAR update with this change is still pending.

B. Evaluation of Design Issues Related To Each Specific Category I Site-Specific Structure

The foundation analysis and design information in the FSAR for the different site-specific structures is discussed in the following paragraphs and the SRP 3.8.5 acceptance criteria are employed. There are seven acceptance criteria: (1) description of the foundation; (2) applicable codes, standards, and specifications; (3) loads and load combinations; (4) design and analysis procedures; (5) structural acceptance criteria; (6) materials, quality control, and special construction techniques; and (7) testing and inservice surveillance programs. These are all addressed in SER Section 3.8.4 for every Category I building, because the foundation is designed as part of the structure and not as a separate item. SER Section 3.8.4 also includes a discussion on the general methods used to evaluate stability. However, a description of how stability is addressed for each specific Category I structure follows:

B.1. Foundation Information for the UHS/RSW Pump House

In FSAR Subsection 3.8.5.9.1, the applicant includes a description of the foundation stating that the UHS/RSW pump house foundation is a 3.05-m (10-ft) reinforced concrete basemat placed over 0.61 m (2 ft) of thick lean concrete mud mat. The foundation is an integral part of the structure that is described and evaluated in more detail following the SRP acceptance criteria in SER Subsection 3.8.4.4.2, under Subpart B.1, “Design Information for UHS/RSW Pump House and RSW Piping Tunnels.”

Stability Analysis of the UHS/RSW Pump House

Stability analyses of the UHS/RSW pump house are described in the paragraphs that follow. There are stability checks for floatation, sliding, and overturning that consider both an empty and full basin. The staff reviewed the stability calculations for the UHS/RSW pump house during the audit in May 2011. The loadings considered for the stability evaluation include the site-specific SSE (0.15g peak ground acceleration [PGA]), design wind speed (215.6 km/h [134 mph]), design-basis tornado for Region II (321.9 km/h [200 mph]), and the design flood level of 12.2 m (40 ft) from the main cooling reservoir dike break. The lateral soil pressures used for the stability evaluation are shown in Figures 3H.6-45 through 3H.6-50. Load combinations described in FSAR Subsection 3H.6.4.5 were used for stability evaluation. The calculated safety factor for sliding requires less than half of the available passive pressure to be engaged for sliding resistance. The factor of safety for sliding against the SSE is calculated assuming the minimum operable water level inside the UHS and the maximum ground water level of 8.53 m (28 ft). The evaluations of seismic overturning moments and the sliding account for the simultaneous application of seismic forces in three directions use the 100-40-40 combination rule. For SSE load combinations, the hydrodynamic effects of the basin water are considered. In addition, buoyancy due to the maximum ground water level of 8.53 m (28 ft) is considered for the sliding check. For the SSE load combination, another sliding evaluation is performed for the empty UHS basin. The calculated safety factor for sliding requires less than half of the available passive pressure to be engaged for sliding resistance. The factors of safety for the stability evaluation are reported in Table 3H.6-5 and exceed the minimum value specified in SRP Section 3.8.5.

B.2. Foundation Information for RSW Piping Tunnels

In FSAR Subsection 3.8.5.9.2, the applicant includes a description of the foundation stating that the foundation for the RSW tunnels is a 0.91-m (3-ft) thick reinforced concrete basemat placed over a 0.61-m (2-ft) thick lean concrete mud mat. The foundation is an integral part of the structure that is described and evaluated in more detail following the SRP acceptance criteria in SER Subsection 3.8.4.4.2, Subpart B.1, “Design Information for UHS/RSW Pump House and RSW Piping Tunnels.”

Stability Analysis of the RSW Piping Tunnels

The stability evaluation of the RSW piping tunnels uses the loads and load combinations described in FSAR Subsection 3H.6.4.5. The driving seismic soil pressure used for the stability evaluation is the dynamic active soil pressure computed with the Mononobe-Okabe method (“Design of Earth Retaining Structures for Dynamic Loads,” Proceedings of the Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures, ASCE, NY, pp. 103–147, Seed, H.B., and Whitman, R.V., 1970). The inertial load is based on a conservative acceleration of 0.21g acting on the tunnel mass. During the February 2012 audit, the applicant was asked to update the stability analysis using the most current amplified motion based on the MSM of analysis. In the Supplement 3 response to **RAI 03.07.01-29** (ML12103A369), the applicant states that the SSI soil pressures were scaled to account for the change in the amplified motion due to the use of the MSM. Furthermore, in the Supplement 4 response to **RAI 03.07.02-13** Item D (ML11335A232), the applicant states that the amplified input motions used in the stability analyses for the RSW piping tunnels were obtained from SSI analyses using the MSM or the DM. The same response also states that seismic sliding forces and overturning moments from the SSI and SSSI analyses were less than the ones used for the stability calculations based on dynamic active pressure, as described above. The total driving force resulting from the SSI and SSSI was thereby obtained by integrating the nodal forces over the buried part of the tunnel perimeter. The staff questioned the ability of this procedure to correctly capture the effects of nearby heavy structures on the driving and resisting forces. Therefore, during the February 2012 audit, the applicant was asked to discuss whether a conservative approach using the absolute maximum driving soil pressures (i.e., from Figures 3H.6-212 through -215) obtained from the SSSI model or correspondingly from the SSI model, would still result in satisfactory factors of safety.

In the Supplement 5 response to **RAI 03.07.02-13** Item 6 (ML12103A369), the applicant refers to an investigation performed for the DGFOT in which it was shown that at no time step did the required SSI or SSSI resisting forces exceed the full passive soil force, and the same result is obtained if the maximum absolute SSI or SSSI driving forces are compared with the full passive soil force. The applicant also states that (a) this results from this investigation are also applicable to the stability of the RSW tunnel, and (b) the stability evaluation based on the Mononobe-Okabe method described above is adequate. Furthermore, the portion of the RSW tunnel closest to the RB is sandwiched between the RB and the RWB, which provides additional confinement of the RSW tunnel against sliding. The staff found the applicant's assertion technically acceptable, because confinement of the tunnel between the RB and the RWB will provide stability against sliding. In addition, the tunnel walls are designed for the lateral soil pressures considering the SSSI effects. The stability analysis was performed for a typical tunnel cross section and for the access regions. The safety factors for the stability evaluation of the RSW tunnel are in Table 3H.6-16. It is shown that the factors of safety against overturning, sliding, and floatation exceed the specified values in the SRP. The staff conducted a review of the RSW tunnel stability report during the February 2012 audit (ML120660018) and noted that whenever sliding is involved (besides the active static and dynamic lateral pressure components), the stability analysis includes the hydrodynamic pressure as a driving force. No surcharge is considered on either the driving or the resisting forces. To achieve a sliding factor of safety of 1.1, about 33 percent of the full passive force (Kp = 3) was required. Although the surcharge loads as part of the driving force is not considered, the staff deemed the approach acceptable because enough margin is still available on the resisting side.

B.3. Foundation Information for DGFOSV

In FSAR Subsection 3.8.5.9.3, the applicant includes a description of the foundation stating that the DGFOSV foundation is a 1.8-m (6-ft) thick reinforced concrete basemat placed over a 0.61-m (2-ft) thick lean concrete mud mat. The foundation is an integral part of the structure that is described and evaluated in more detail following the SRP acceptance criteria in SER Subsection 3.8.4.4.2, under Subpart B.2, "Design Information for DGFOSV."

Stability Analysis of the DGFOSV:

The stability evaluation of the DGFOSV described in FSAR Subsection 3H.6.4.5 used the loads and load combinations, whereby the driving seismic soil pressure is the dynamic active soil pressure computed using the Mononobe-Okabe method. The inertial loads acting directly on the vault mass and the dynamic at-rest soil pressure incremental loads are obtained from the 3-D DGFOSV model. During the February 2012 audit (ML120660018), the applicant was asked to update the stability analysis using the most current amplified motion based on the MSM of analysis.

In the Supplement 4 response to **RAI 03.07.02-13** Item D (ML11335A232), the applicant states that the amplified input motions used in the stability analyses for the DGFOSV are obtained from the SSI analyses using the MSM of analysis. Furthermore, the response states that though the SSSI analyses involving the DGFOSV are based on the SM of analysis, the impact on the soil pressures was expected to remain under 10 percent based on more detailed examinations of the effects from the SASSI method of solution on the SSSI soil pressures in the Supplement 1 response to **RAI 03.07.01-29** (ML113250374). Under Item A, the response states that seismic sliding forces and overturning moments from the SSI and SSSI analyses were less than the ones used for the stability calculations based on the dynamic active pressure as described above, with a margin of at least 50 percent. Because this minimum margin of 50 percent is significantly more than the expected 10 percent difference from the SASSI method of analysis, no further evaluation is required. The staff agreed with the applicant's basis for technical adequacy of stability evaluation of the DGFOSV

However, the total driving force resulting from the SSI and SSSI was obtained by integrating the nodal forces over the buried part of the DGFOSV perimeter. The staff questioned the ability of this procedure to correctly capture the effects of nearby heavy structures on the driving and resisting forces. Therefore, during the February 2012 audit the applicant was asked to discuss whether a conservative approach using the absolute maximum driving soil pressures (i.e., from Figures 3H.6-226 through -231) obtained from the SSSI model or correspondingly from the SSI model, would still result in satisfactory factors of safety. In the Supplement 5 response to **RAI 03.07.02-13** Item 6 (ML12103A369), the applicant refers to an investigation performed for the DGFOT that is more critical in terms of having an adjacent heavy structure. This investigation showed that at no time step did the required SSI or SSSI resisting forces exceed the full passive soil force, and the same result is obtained if the maximum absolute SSI or SSSI driving forces are compared with the full passive soil force. The applicant also states that (a) the results of this investigation are also applicable to the stability of the DGFOSV, and (b) the stability evaluation based on the Mononobe-Okabe method described above is adequate. The staff agreed with the applicant's assertion and found it to be adequate.

The reported safety factors in Table 3H.6-12 include an implicit additional margin because the reported sliding resistance is based only on a partial activation of the passive pressure (less than full passive pressure), which can be seen in Figures 3H.6-256 and -257. The results also show that the factors of safety against overturning, sliding, and floatation meet the specified values in the SRP. In addition, whenever sliding is involved—besides the active static and dynamic lateral pressure components—the stability analysis includes the hydrodynamic pressure as a driving force. No surcharge is considered on either the driving or resisting forces. To achieve a sliding factor of safety of 1.1 required about two-thirds of the full passive force ($K_p = 3$). Although the surcharge loads as part of the driving force is not considered, the staff deemed the approach acceptable because enough margin is still available on the resisting side.

B.4. Foundation Information for DGFOT

In FSAR Subsection 3.8.5.8.1, the applicant includes a description of the foundation stating that the DGFOT foundation is a 0.61-m (2-ft) thick reinforced concrete basemat placed over a 0.61-m (2-ft) thick and lean concrete mud mat. The foundation is an integral part of the structure that is described and evaluated in more detail following the SRP acceptance criteria in SER Subsection 3.8.4.4.2, Subpart B.3, "Design Information for DGFOT."

Stability Analysis of the DGFOT

The stability evaluation of the DGFOT used the loads and load combinations described in FSAR Subsection 3H.7.4.5, whereby the driving seismic soil pressure is the dynamic active soil pressure computed using the Mononobe-Okabe method. The inertial loads acting directly on the tunnel mass were obtained from the 3-D DGFOT model. As the presence of the nearby RB influences the response of the tunnel, the amplified input motion obtained from the 3-D RB-SSI model is used as the seismic excitation. During the February 2012 audit (ML120660018), the applicant was asked to update the stability analysis using the most current amplified motion based on the MSM of analysis.

In the Supplement 4 response to **RAI 03.07.02-13** Item D (ML11335A232), the applicant states that the amplified input motions used in the stability analyses of the DGFOT were obtained from the SSI analyses using the MSM or DM and the SSSI analyses based on the SM of analysis. However, based on more detailed examinations performed for other SSSI situations, the impact on the soil pressures was expected to remain under 10 percent. Furthermore, under Item (A), the response states that seismic sliding forces and overturning moments from the SSI and SSSI analyses were less than the ones used for the stability calculations based on the dynamic active pressure, as described above. The total driving force resulting from the SSI and SSSI was obtained by integrating the nodal forces over the buried part of the tunnel perimeter. The staff questioned the ability of this procedure to correctly capture the effects of the adjacent RB on the driving and resisting forces. During the February 2012 audit, the applicant was asked to discuss whether a conservative approach using the absolute maximum driving soil pressures (i.e., from Figures 3H.7-5 through -8) obtained from the SSSI model or correspondingly from the SSI model, would still result in satisfactory factors of safety.

In the Supplement 5 response to **RAI 03.07.02-13** Item 6 (ML12103A369), the applicant refers to an investigation performed for the DGFOT, which shows that at no time step did the required SSI or SSSI resisting forces exceed the full passive soil force, and the same result is obtained if the maximum absolute SSI or SSSI driving forces are compared with the full passive soil force. In addition, the portion of the DGFOT closest to the RB is sandwiched between the RB and the CFRW, which will remain in place after construction. Based on the above information, the applicant concludes that the stability evaluation based on the Mononobe-Okabe method described above is adequate. During the February 2012 audit, the staff reviewed the DGFOT stability evaluation report and noted that:

- The vertical and horizontal ZPA used 0.30g to calculate the dynamic soil effects. This value is conservative since the site-specific PGA is 0.13g.

- The equivalent horizontal acceleration of the tunnel section used 0.45g to compute the inertial forces. This value is conservative since this was based on 0.3g PGA.

- The full static passive loading is used to resist the driving forces; the surcharge lateral loads are not included.

- Whenever sliding is involved in the stability analysis, the active static and dynamic lateral pressure components with surcharges are included as driving forces. Differing from the other analysis, the hydrodynamic pressure was not part of the driving forces. However, Figure 3H.7-3 (which was submitted with the Supplement 5 response to **RAI 03.07.02-13** [ML12103A369]) includes in the driving forces the hydrodynamic component.

- The stability analysis was performed for a typical tunnel cross section and for the access regions.

Based on the above review, the staff concluded that the applicant has provided an adequate technical basis for the stability evaluation of the DGFOT by comparing its evaluation with more conservative alternative methods. FSAR Table 3H.7-2 provides the safety factors for the stability evaluation of the DGFOT. Figures 3H.7-3 and 3H.7-4 provide the driving and resisting lateral earth pressures, respectively, for the stability evaluation. The factors of safety against overturning, sliding, and floatation meet the specified values in the SRP. To achieve adequate safety against missile impact, in FSAR Subsection 3H.7 the applicant states that during the detail design horizontal restraints can be added to both access shafts to ensure that the minimum factor of safety of 1.1 is maintained. The reported safety factors in Table 3H.7-2 against overturning, sliding, and floatation meet or exceed the specified values in SRP Section 3.8.5.

3.8.5.5 *Post Combined License Activities*

There are no post COL activities related to this section.

3.8.5.6 *Conclusion*

The NRC staff's findings related to information incorporated by reference are in NUREG–1503. NRC staff reviewed the application and checked the referenced DCD. The staff’s review confirmed that the applicant has addressed the required information, and no outstanding information is expected to be addressed in the COL FSAR related to this section. Pursuant to 10 CFR 52.63(a)(5) and 10 CFR Part 52, Appendix A, Section VI.B.1, all nuclear safety issues relating to foundations that were incorporated by reference have been resolved.

The staff compared the supplemental information in the application to the relevant NRC regulations, the guidance in Section 3.8.5 of NUREG–0800, and other NRC RGs. The staff’s review concluded that the applicant has adequately addressed the COL license and supplemental information items and Tier 1 departures in compliance with NRC regulations.