PMSTPCOL PEmails

From: Chakrabarti, Samir
Sent: Friday, June 25, 2010 12:27 PM
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Subject: STP 3.8 Follow-up Questions
Attachments: RAI_Tracking_Table(11-18-09)-r.doc; RAI 4834b FINAL.doc; RAI 4832b FINAL.doc; RAI 4833b FINAL.doc

Ravi,

Attached please see the FINAL versions of the follow-up questions after addressing Kim's comments. There may be some more changes after OGC review. That is expected to be mostly editorial and language format. This is the FINAL version for now.

Please update the assessment to reflect the final questions, and clean it up. Also, you need to update the tracking table. I am attaching a copy of the table prepared earlier by Hassan. You may update that Table.

We are planning to have an interaction with STP after the questions go out to clarify the questions. We will need your participation in the interaction. It may be either a conference call, or public meeting. I will let you know details later.

I will be on vacation next week, and will be back on July 2, 2010.

Thanks.

Samir Chakrabarti
NRO/DE/SEB2
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MESSAGE	893	6/25/2010 12:26:50 PM
RAI_Tracking_Table(11-18-09)-r.doc	999930	
RAI 4834b FINAL.doc	36346	
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RAI 4833b FINAL.doc	59898	

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Tracking Table of South Texas Project (STP) Response to RAIs Related to Section 3.8		Date: 12/15/09	
Reviewer:	Hassan Sedarat	Revision No:	0
Based on RAI dated	11/18/2009	Revision Date:	12/15/2009
Based on TER	11/27/2009	Page:	*1 of 71*
Based on follow up question dated:	11/30/2009		

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.01-1	Use of new version of IBC code.	**RAI Text** In FSAR Section 3.8, page 3.8-1, the applicant references the departure STD DEP 1.8-1, "Tier 2* Codes, Standards, and Regulatory Guide Edition Changes." One of the changes included in this departure updates Tier 2 to refer to the 2006 International Building Code (IBC) in place of the 1991 Uniform Building Code (UBC). These building codes are not endorsed by the NRC in their entirety, and use of these codes is evaluated by the staff on a case by case basis. The staff had previously evaluated only the use of 1991 UBC for the ABWR standard design. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two codes as they apply to the ABWR standard design, and provide justification for any differences in order for the staff to evaluate the use of the 2006 IBC. Also, Section 3.8.1.3.1, item (2)(b), has been revised to read "Section 9.3 of ASCE Standards 7-88 and Section 1613 of the International Building Code (IBC) specify that …" This has created an apparent inconsistency, since IBC Section 1613 does not refer to Section 9.3 of the ASCE standard for relevant information. The applicant is requested to clarify the inconsistency. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** As applied to the ABWR standard design, the DCD references the 1991 Uniform Building Code specifically for seismic analysis methodology and requirements for other structural loads and load combinations (e.g., environmental-related loads) for the design of non-seismic Category I structures. Accordingly, STPNOC will provide a comparison of the 1991 Uniform Building Code and the 2006 International Building Code requirements for seismic analyses and other structural loading requirements. This comparison will include identification of differences between the two codes as they apply to the ABWR design described in the STP 3 and 4 FSAR and justification for use of the newer code requirements. STPNOC plans to submit this comparison by October 29, 2009. With regard to COLA Part 2, Tier 2, Section 3.8.1.3.1, item (2)(b), prior to COLA Revision 2, this item referenced requirements in Section 9.3 of ASCE Standard 7-88 and Section 2334(a) of the 1991 Uniform Building Code which both require that a minimum of 25 percent of the floor live loads should be considered for the computation of design seismic forces for storage and warehouse type occupancies. COLA Revision 2 replaced the reference to Section 2334(a) of the 1991 Uniform Building Code with a reference to Section 1613 of the International Building Code. STPNOC

acknowledges that the referenced section of the International Building Code does not contain explicit requirement for seismic design forces (i.e., minimum of 25 percent of floor live loads) as was contained in the previous reference. Rather, Section 1613 of the International Building Code (2006) specifies the use of American Society of Civil Engineers (ASCE) 7-05. Section 12.7.2 of ASCE 7-05 requires that a minimum of 25 percent of floor live load be used for design seismic forces for storage areas, consistent with the previous reference. In order to more explicitly identify the applicable source codes for the requirements cited in COLA Part 2, Tier 2, Section 3.8.1.3.1, Item 2(b) is revised as provided below. The revision also includes a minor editorial correction to conform the COLA to the language used in ASCE 7-88.

3.8.1.3.1 Normal Loads

(2)

The criteria for consideration of live loads for the designs of structural elements of the Reactor Building and *Control Building ~~and the Radwaste Building~~ are provided in Subsections 3H.1.4.3.1~~,~~* and *3H.2.4.3.1, ~~and 3H.3.4.3.1,~~ respectively.*

(b) *Section 9.3 of ASCE Standards 7-88 and ~~Section 2334(a) of the 1991 Uniform Building Code~~* ~~Section 1613 of the International Building Code (IBC)~~ Section 12.7.2 of ASCE 7-05 *specify that a minimum of 25% of the floor live loads should be considered for the computation of design seismic forces for storage and warehouse type occupancies. The variation in live load intensity and occurrence in operating nuclear plants is expected to be no higher than that for storage ~~in~~*and *warehouse occupancies. A 25% of full live loads is, therefore, equally applicable to the nuclear plants.*

COLA Impact

UBC and IBC will be compared and COLA will be revised to ASCE only.
In order to more explicitly identify the applicable source codes for the requirements cited in COLA Part 2, Tier 2, Section 3.8.1.3.1, Item 2(b) is revised as provided above. The revision also includes a minor editorial correction to conform the COLA to the language used in ASCE 7-88.

Staff Assessment

		The applicant's response to Question 03.08.01-1 states that the differences between the UBC and the IBC will be prepared by October 29, 2009, and addressed in the COL application. Upon submittal of this document on October 29, it will be reviewed. This part of the question remains open. Also, the applicant revised FSAR Subsection 3.8.1.3.1 to address the inconsistent references to the American Society of Civil Engineers (ASCE) Code and the International Building Code (IBC). The applicant removed references to the IBC, which resolves this issue. The staff found this response acceptable. This part of the question is a confirmatory item, pending the FSAR update. **Follow-up Question to the Partial Response** None. **Status** Open – Incomplete response. Final response due on 10/29/2009

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.01-2	Error in identifying STD DEP 1.8-1	**RAI Text** In FSAR Appendix 3H, page 3H-1, the applicant references (site specific) departure STP DEP 1.8-1. However, no such departures could be located in Part 7, Departures Report, which instead included STD DEP 1.8-1. This appears to be an error. The applicant is requested to correct the error, or clarify this. Application Response Reference File<U7-C-STP-NRC-090105.pdf> The typographical error on Page 3H-1 will be corrected by revising STP DEP 1.8-1 to STD DEP 1.8-1 The COLA will be revised as shown below as a result of this response. **3H Details and Evaluation Results of Seismic Category 1 Structures** The information in this appendix of the reference ABWR DCD, including all subsections, tables, and figures is incorporated by reference with the following departures and supplement. STD DEP T1 2.15-1 STP DEP T1 5.0-1 STPD DEP 1.8-1 **COLA Impact** The COLA will be revised as shown below as a result of this response. **Staff Assessment** The applicant's response to Question 03.08.01-2 corrects the error in FSAR Appendix 3H by revising STP DEP 1.8-1 to STD DEP 1.8-1 and providing a mark-up of the FSAR. The correction is reflected in Revision 3 of the FSAR. This RAI is therefore resolved.

		Follow-up Question to the Partial Response None. **Status** Resolved

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.01-3	Design information for water-tight doors.	**RAI Text** In FSAR Appendix 3H, Section 3H.1.4.2, sub-item (3), the applicant stated that the maximum flood level for STP units 3 & 4 site is 442 cm above grade against the corresponding ABWR standard design value of 0.305 m below grade (departure STP DEP T1 5.0-1). In Part 7 of the application, the applicant performed an evaluation of this departure, and stated that STP 3 & 4 safety-related SSCs are designed for or protected from this flooding event by watertight doors to prevent the entry of water into the Reactor Buildings and the Control Buildings in case of a flood. The applicant also stated that the exterior doors located on the 12300 floor of the Reactor Building and Control Building are revised to be watertight doors. Since these doors play a significant role in protecting safety-related SSCs and constitutes a special design feature, the staff requests the applicant to include in the FSAR sufficient design information about these doors including locations, seismic and other design criteria, seismic classification, redundancy features, if any, and if these doors will be used for normal access and egress to and from the Reactor Building and the Control Building. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** The watertight doors for the Reactor Building to be utilized for protection against external flooding consist of the five exterior doors and the exterior Large Equipment Access Building door shown in COLA Part 2 Tier 1 Figure 2.15.10j. The watertight doors for the Control Building to be utilized for protection against external flooding consist of the exterior equipment access door and an access door between the Control Building and the Service Building shown in DCD Tier 1 Figure 2.15.12g and an additional access door between the Control Building and Radwaste Building Access Corridor. Since the function of these watertight doors is to protect safety-related SSCs in the event of a Probable Maximum Flood (PMF), they will be considered safety-related and designed as Seismic Category I for the site-specific loading. Exterior openings of the Reactor Building and Control Building which could make safety related SSCs vulnerable to tornado missiles are protected by separate barriers or doors designed to resist tornado missiles. The exterior watertight doors will be designed for the wind, tornado wind and pressure drop discussed in COLA Tier 2, Section 3.3 as applicable.

		The watertight doors will be seated such that the force of the water helps maintain the watertight seal. The watertight doors are designed to be leak tight. The door openings which provide access for maintenance will be normally closed and will not be used for normal access to and from the Reactor Building and the Control Building. The door openings between the Control Building and the Service Building and between the Control Building and Radwaste Building Access Corridor provide access and egress from the Control Building. The flood resistant doors in these openings will be normally open and closed only upon indication of an imminent flood. Separate access doors which function as fire doors will be normally closed, but will be compliant with the requirements of NFPA 101 for egress. The operation of the watertight doors will be controlled by station procedures. COLA Part 2, Tier 2, Section 3.8.6.4 will be revised as provided below.

3.8.6.4 Identification of Seismic Category I Structures

<Add the following paragraphs at the end of this section>

In addition to the above structures, watertight doors are required on the Reactor and Control Buildings to protect the buildings from the external design basis flood. These watertight doors are considered site-specific Seismic Category I components.

The watertight doors for the Reactor Building to be utilized for protection against external flooding consist of the five exterior doors and the exterior Large Equipment Access Building door shown in COLA Part 2 Tier 1 Figure 2.15.10j. The watertight doors for the Control Building to be utilized for protection against external flooding consist of the exterior equipment access door and an access door between the Control Building and the Service Building shown in DCD Tier 1 Figure 2.15.12g and an additional access door between the Control Building and Radwaste Building Access Corridor.

Since the function of these watertight doors is to protect safety-related SSCs in the event of a Probable Maximum Flood (PMF), they are considered safety-related and designed as Seismic Category I for the site-specific loading.

Exterior openings of the Reactor Building and Control Building which could make safety related SSCs vulnerable to tornado missiles are protected by separate barriers or doors designed to resist tornado missiles. The exterior watertight doors are designed for the wind, tornado wind and pressure drop discussed in Section 3.3 as applicable.

The watertight doors are seated such that the force of the water helps maintain the watertight seal. The watertight doors are designed to be leak tight.

The door openings which provide access for maintenance are normally closed and are not used for normal access to and from the Reactor Building and the Control Building. The door openings between the Control Building and the Service Building and between the Control Building and Radwaste Building Access Corridor provide access and egress from the Control Building. The flood resistant doors in these openings are normally open and closed only upon indication of an imminent flood. Separate access doors which function as fire doors are normally closed, but are compliant with the requirements of NFPA 101 for egress. The operation of the watertight doors are controlled by station procedures.

COLA Impact

		COLA Part 2, Tier 2, Section 3.8.6.4 will be revised **Staff Assessment** The applicant's response to Question 03.08.01-3 revises FSAR Subsection 3.8.6.4 to include a description of the watertight doors, the seismic classification of the doors, identification of the locations of the doors, descriptions of the normal operating positions of the doors, and a statement that the normally open watertight doors will be closed upon the indication of an imminent flood. The applicant also states that these doors will be designed as Seismic Category I for site-specific loads. However, the applicant does not include in the FSAR sufficient design information for these doors according to the guidance in the SRP Acceptance Criteria of SRP 3.8.4 for Seismic Category I structures. Also, it is not clear from this response whether the seismic classification of the doors is captured in other relevant FSAR sections, or how the station procedures for the doors will be evaluated and implemented. The applicant also does not indicate whether there are any redundancy features for the doors. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.01-3 (RAI 2962, Rev.2) The applicant's response to Question 03.08.01-3 identifies the watertight doors that will be required to protect safety-related systems and components against a probable maximum flood (PMF) and states that these doors are designed as Seismic Category I for site-specific loads. The applicant also states that the watertight doors between the Control Building and the Service Building and between the Control Building and the Radwaste Building Access Corridor (1) provide access to and egress from the Control Building, (2) will normally remain open and will be closed only upon the indication of an imminent flood, and (3) are controlled by station procedures. Because these doors play a significant role in protecting safety-related systems, structures, and components (SSC) and constitute a special design feature, the staff requests the applicant to provide additional information about these doors and to update the FSAR as necessary, as stated below, in order for the staff to complete the evaluation: Include 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, or explain why it is not necessary to do so. Identify the location of the additional watertight door between the Control Building and the Radwaste Building Access

		Corridor, which is not clear from the response (please identify the location of this door in a drawing). Clearly state in the FSAR the (a) site-specific loads and load combinations, (b) applicable codes and standards, (c) design and analysis procedures, (d) structural acceptance criteria, (e) materials and quality control, and (f) testing and in-service surveillance programs used to design, construct, install, and maintain these doors and all of the components following the guidance in SRP 3.8.4 (SRP Acceptance Criteria 1 through 7), or explain why it is not necessary to do so. Explain what mechanism is 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. Also confirm whether the adequacy of the station procedures to effectively close these doors when needed has been evaluated. Describe whether any redundancy features were considered for the watertight doors, particularly those that are normally open. Clarify what appears to be access doors between the Control Building and the Reactor Building that are not identified as watertight doors to be utilized for protection against external flooding. Since there is a gap between these buildings, explain what design feature is provided to ensure that flood water cannot enter the Reactor Building and the Control Building through these access areas. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.01-4	Evaluation of standard plant structures for site-specific flood load.	**RAI Text** In FSAR Appendix 3H, Section 3H.1.6, "Site Specific Structural Evaluation," the applicant addressed the effect of increased maximum flood level (STP DEP T1 5.0-1) for STP units 3 & 4 on the design of the Reactor Building (RB). In this section the applicant stated that "the load due to the revised flood level on the RB is less than the ABWR Standard Plant RB seismic load, and hence it does not affect the Standard Plant RB structural design." The staff considers this evaluation to be very qualitative, and the evaluation does not adequately address all issues associated with increased flood level. Therefore, the staff requests the applicant to provide a quantitative evaluation considering all effects due to the increased flood level including wave effects, if any, potential loadings due to flow and drag, overall stability of the structure considering floatation, etc. Also, it is not understood why the factor of safety for foundation stability considering buoyant forces from design basis flood reported in Table 3H.1-23 of the ABWR Standard Plant is not considered affected by the increased flood level. The same issue applies to the site specific structural evaluation of the control Building presented in Section 3H.2.6, and factor of safety for foundation stability reported in Table 3H.2-5 of the ABWR Standard Plant. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** The following is based on the Main Cooling Reservoir (MCR) embankment breach analysis results provided in Attachment 1 of letter U7-C-STP-NRC-090012, dated February 23, 2009: Maximum calculated water level near the safety-related structures is at elevation 38.8 ft. Design flood level is conservatively established at elevation 40 ft. Maximum hydrodynamic force is 44 pounds per square foot of the projected submerged area. The plant grade is at elevation 34 ft. Considering design flood level of 40 ft, the out-of-plane load on the above grade exterior walls of the Reactor Building (RB) and Control Building (CB) under flooded condition will be due to the hydrostatic pressure and hydrodynamic force of 44 lb/ft2. This load is only applicable from elevation 34 ft to elevation 40 ft. For the below grade portions of the exterior walls, under flooded condition, the walls will be subjected to an increase of static water pressure due to 7 ft (from ground water elevation of 33 ft to design basis flood level of 40 ft) of water

		head. Impact on the above grade walls The calculated out-of-plane shear and moment demand for exterior walls of the RB and CB due to induced loading from MCR breach and safe-shutdown earthquake, SSE, are as follows: For Reactor Building: Out-of-plane shear and moment demands due to MCR breach are 2.13 k/ft and 5.44 k-ft/ft, respectively. Out-of-plane shear and moment demands due to SSE are 3.03 k/ft and 15.16 k-ft/ft, respectively. For Control Building: Out-of-plane shear and moment demands due to MCR breach are 2.07 k/ft and 5.1 k-ft/ft, respectively. Out-of-plane shear and moment demands due to SSE are 2.16 k/ft and 9.13 k-ft/ft, respectively. Impact on the below grade walls The increase in the out-of-plane load on the exterior walls of the RB and CB under flooded condition will be equal to 7 ft of water head or 7x62.4 = 436.8 psf. Referring to DCD Tier 2, Figures 3H.1-11 and 3H.2-14, the minimum seismic lateral soil pressure considered for design of below grade exterior walls of the RB and CB is 39.26 kPa or 819.96 psf which exceeds the 436.8 psf due to flood. Based on the above, the out-of-plane flood loading on the exterior walls of the RB and CB are enveloped by out-of-plane SSE loading and thus the exterior walls of the RB and CB are adequate for resisting the induced flood loads from MCR embankment breach. Impact on the stability safety factors The flood load (excluding buoyancy) is only applicable to the bottom 6 ft of the above grade portion of the RB and CB and thus the total flood load on these two structures is substantially less than total seismic load which will be based on SSE excitation of the entire structure. Therefore, the sliding and overturning stability is not impacted. The effect of

flooding on flotation safety factors is addressed below:

Per DCD Tier 2 Tables 3H.1-23 and 3H.2-5, the flotation safety factors for the RB and CB are 2.43 and 1.42 respectively. These flotation safety factors are based on maximum ground water level being one foot below grade (i.e. elevation 33 ft). Considering design flood level of 40 ft, the increased buoyancy force will result in revised flotation safety factors of 2.24 and 1.3 for RB and CB, respectively. These revised flotation safety factors are acceptable since they exceed the required flotation safety factor of 1.1 in accordance with Standard Review Plan Section 3.8.5.

COLA Part 2, Tier 2, Section 3H.1.6 and 3H.2.6 will be revised as provided below.

3H.1.6 Site Specific Structural Evaluation

As documented in Subsection 3.4, the STP 3 & 4 site has a design basis flood elevation that is 182.9 cm (6 ft) above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level on the exterior above and below grade RB walls is less than the ABWR Standard Plant RB seismic load, hence it doesn't affect the Standard Plant RB structural design. Increased flood level also increases the buoyancy force resulting in a revised flotation factor of safety of 2.24. This factor exceeds required factor of safety of 1.1.

3H.2.6 Site Specific Structural Evaluation

As documented in Subsection 3.4, the STP 3 & 4 site has a basis flood elevation that is 182.9 cm (6 ft) above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level on the exterior above and below grade CB walls is less than the ABWR Standard Plant seismic load, hence it does not ~~a~~affect the Standard Plant CB structural design. Increased flood level also increases the buoyancy force resulting in a revised flotation factor of safety of 1.3. This factor exceeds required factor of safety of 1.1.

COLA Impact

COLA Part 2, Tier 2, Section 3H.1.6 and 3H.2.6 will be revised.

Staff Assessment

The applicant's response Question 03.08.01-4 refers to U7-C-STP-NRC-090012. In Section 2.4S.4.3 of that reference, the applicant states that the design flood level is 40 ft. Subsection 2.4S.4.2.2.4.3 of the same reference also states that the maximum hydrodynamic pressure is 44 pounds per square foot, which is only applicable from grade elevation (34 ft) to elevation (40 ft). Therefore, the applicant concludes that there are 6 ft of flood pressure above grade and 7 ft of flood pressure below grade. The applicant summarizes the impact on the above grade and below grade walls. The applicant's response compares the out-of-plane shear and the moment demands due to flood pressure with those due to the seismic load for the above grade walls, and concludes that because these quantities for the seismic load are more than those from the flood load; the flood load will not result in higher stresses than those obtained from the seismic analysis. The staff agrees with the methodology used by the applicant for the evaluation since both flood load and seismic load can be considered as extreme environmental loads, and treated similarly in loading combinations. However, the applicant did not explain in the response how the shear and moment demands for flood load and seismic load were determined for performing the evaluation. For the below grade walls the applicant calculated the out-of-plane load on the exterior walls due to flood, and compared that with the minimum seismic lateral soil pressure considered for design of below grade exterior walls of the Reactor Building (RB) and the Control Building (CB) of the ABWR standard design. The staff considers this part of the response acceptable, since the minimum seismic lateral soil pressure considered in the design of the below grade walls of the RB and the CB was higher than the out-of-plane load on these walls due to flood.

In response to the question regarding the impact of the flood load on the stability safety factor, the applicant states that the flood load (excluding buoyancy) is only applicable to the bottom 6 ft of the above grade portion of the RB and CB, and thus the total flood load on these two structures is substantially less than the total seismic load. The applicant then concludes that the sliding and overturning stability is not impacted.

The staff reviewed the response and found that the applicant does not clearly indicate whether the hydrodynamic load was included in this evaluation. Also, it is not understood why buoyancy was excluded in the evaluation. Buoyancy will reduce the restoring effect of vertical force against sliding and overturning, and therefore needs to be considered in the computation of the sliding and overturning factor of safety. Therefore, the applicant is requested to provide a quantitative evaluation of the sliding and overturning stability due to flooding that includes both buoyancy and hydrodynamic load, or provide justification for not doing so.

		Finally, the applicant's response to the same question provides the revised factors of safety due to floatation of the RB and the CB, which are different from the values reported in Tables 3H.1-23 and 3H.2-5 of the ABWR DCD. The applicant revised FSAR Sections 3H.1.6 and 3H.2.6 to report the revised factors of safety against floatation. However, the applicant's response does not include a revision to the above ABWR DCD tables. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.01-4 (RAI 2962, Rev.2) The staff reviewed the applicant's response to Question 03.08.01-4 addressing the evaluation of standard plant structures for the increased flood level and needs the following additional information to complete the review: The applicant's response compares the out-of-plane shear and moment demands due to flood pressure with those due to the seismic load. The applicant did not include in its response any description or explanation about how the out-of-plane shear and moment demand for flood load and seismic load were obtained for the evaluation. Therefore, the staff requests the applicant to provide a detailed description of how the representative wall elements for the reactor building (RB) and the control building (CB) were selected for the evaluation, and how the reported shear and moment demands for flood and seismic load were determined. In its evaluation for impact of increased flood level on sliding and overturning stability, the applicant considered only the flood load acting on the bottom 6 ft of the above ground portion of the RB and the CB excluding buoyancy, and made a qualitative statement that the flood load is substantially less than the seismic load. Please explain why sliding and overturning of the structures due to flooding need not consider the hydrodynamic loads and the buoyancy effects on the structures, and provide a quantitative evaluation of sliding and overturning stability due to flooding. Please also update the FSAR to reflect that sliding and overturning of the RB and the CB were evaluated for the increased flood load on these structures. The applicant's response revises the factors of safety due to floatation for the RB and the CB, which are different from the values reported in Tables 3H.1-23 and 3H.2-5 of the ABWR DCD and in revised FSAR Sections 3H.1.6 and 3H.2.6. However, the applicant's response does not include the revision to the above ABWR DCD tables. Because the values of the floatation safety factors reported in DCD Tables 3H.1-23 and 3H.2-5 are no longer valid for the STP Units 3 and 4, the applicant is requested to address the issue appropriately.

		Status Open

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.01-5	Impact of increased Pool Swell loads on containment and internal structures.	**RAI Text** In FSAR Section 3G, "Response of Structures to Containment Loads," the applicant stated that the information in this section is incorporated by reference to the ABWR DCD. However, a review of Appendix 3B, "Containment Hydrodynamic Loads," Table 3B-1, "Pool Swell Calculated Values," indicate that there has been significant increase in pool swell height and pressure loads for STP units 3 & 4 compared to those reported in the ABWR DCD. Therefore, the applicant is requested to confirm that the results of 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 & 4, and is appropriate to be incorporated by reference. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** As noted in Section 3G.1 of the ABWR DCD, the containment loads considered for structural dynamic response analysis are Condensation Oscillation (CO), Pool Chugging (CH), Horizontal Vent Chugging (HV), Safety/Relief Valve discharge (SRV), and Annulus Pressurization (AP). Although the Pool Swell (PS) load causes impact and drag loads for structures within the Suppression Pool area, it does not cause vibration of the Reactor Building, and is not considered a Reactor Building Vibratory (RBV) load. As such, COLA Part 2, Tier 2, Appendix 3G is unaffected by the Pool Swell loads reported in Appendix 3B, and is appropriate to be incorporated by reference to the ABWR DCD. Changes in loads on the containment internal structures due to increase in pool swell height and pressure are addressed during the detailed design phase. There is no COLA change required for this response. **COLA Impact** None.. **Staff Assessment**

		The applicant's response to Question 03.08.01-5 references Section 3G.1 of the ABWR DCD and identifies the loads considered for structural dynamic response analysis of structures for containment loads. The applicant stated that although Pool Swell (PS) load causes impact and drag loads for structures within the Suppression Pool area, it does not cause vibration in the Reactor Building and is not considered a Reactor Building Vibratory load. Therefore, response of structures to containment loads reported in Appendix 3G of ABWR DCD are not affected by changes in PS loads. The applicant also notes that the effects of pool swell height and pressure load on the containment internal structure will be addressed during the detailed design phase. The staff reviewed ABWR DCD Appendix 3G.1, and confirmed that PS load was not included for calculation of dynamic response of structures to containment loads, and agrees with the applicant's conclusion that structural dynamic response analysis results reported in ABWR DCD Appendix 3G are not affected by changes in the PS loads. However, the staff is concerned that the increase in pool swell height and pressure may affect the design of the concrete containment and the containment internal structures. Furthermore, the results of an evaluation of containment internal structures reported in ABWR DCD Subsection 3H.1.5.5.2 may potentially change due to the increased pool swell height and pressure load, thus affecting the appropriateness of incorporation by reference. The applicant is requested to provide additional information that clarifies the effect of the increased pool swell height and pressure loads on the design of the concrete containment and the containment internal structures and confirm that is appropriate to incorporate by reference from the ABWR DCD. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.01-5 (RAI 2962, Rev.2) The applicant's response to Question 03.08.01-5 states that the changes in loads on the containment internal structures due to the increase in pool swell height and pressure will be addressed during the detail design phase. However, ABWR DCD Subsection 3H.1.5.5.2 describes the design of the containment internal structures, load combination (including pool swell loads), and analysis and design results that are incorporated by reference in FSAR Section 3H. Also, pool swell loads are used in loading combinations for design of containment structure, and analysis and design results for the containment structure are reported in Appendix 3H. Since the changes in loads due to increase in pool swell height and pressure on the concrete containment and containment internal structures are not addressed at this time, the applicant is requested to provide a quantitative evaluation and confirm 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 it is appropriate to incorporate by reference the analysis and design results for the containment and the

		containment internal structure reported in Appendix 3H of ABWR DCD. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-1	Lateral soil pressure on standard plant structures.	**RAI Text** ABWR Standard Plant Design Control Document Section 3H.2.4.3.1.4 and STP units 3 & 4 RCOLA Section 3H.2.4.3.1.4 state that shear wave velocity is one of the parameters used in computation of lateral soil pressures. However, Section 3H.2.6 of the application states that "Shear wave velocity is not used as an input in the calculation of lateral soil pressures. Therefore, change in shear wave velocity has no impact on calculation of lateral soil pressures." The staff requests the applicant to clarify this apparent inconsistency, and describe how lateral soil pressures were calculated for STP units 3 & 4, and how the calculated lateral pressures compare with those used in the ABWR Standard Plant design. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** At-rest seismic lateral earth pressure in non-yielding walls of structures with deep foundations such as the Reactor and Control Buildings will be determined using the method described in Section 2.5S.4.10.5.2 of the Attachment 3 to the STP Letter U7-C-STP-NRC-090012, dated February 23, 2009. In this method, the at-rest seismic lateral earth pressure computation will utilize site-specific shear wave velocity. The impact of site-specific shear wave velocity on the design of exterior walls of these structures 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. Also, the at-rest pressure includes the effect of hydrostatic load, surcharge load etc, in addition to the dynamic pressure caused by the earthquake. As noted in FSAR Section 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 final earth pressure calculations, including actual surcharge loads, structural fill properties, and final configuration of structures, will be added following completion of the project detailed design in an update to the FSAR in accordance with 10CFR 50.71(e) (COM 2.5S-3).

COLA Part 2, Tier 2, Section 3H.2.6 will be revised as provided below.

3H.2.6 Site Specific Structural Evaluation

<Replace the following paragraph>

~~At rest lateral earth pressure in non yielding walls of structures with deep foundations such as the Reactor and Control Buildings will be determined using the method described in Reference 2.5S.4-62. In this method, the at rest seismic lateral earth pressure computation will utilize site specific shear wave velocity. The impact of site specific shear wave velocity on the design of exterior walls of these structures 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 pressure only affects the out of plane load. Also, the at rest pressure includes effect of hydrostatic load, surcharge load etc., in addition to the dynamic pressure caused by the earthquake.~~At-rest seismic lateral earth pressure on the Control Building exterior walls are determined using the method described in Section 2.5S.4.10.5.2. In this method, the at-rest seismic lateral earth pressure computation will utilize site-specific shear wave velocity. The impact of site-specific shear wave velocity on the design of 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. Also, the at-rest pressure includes the effect of hydrostatic load, surcharge load etc, in addition to the dynamic pressure caused by the earthquake.

As noted in Section 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. STP commits to include the final earth pressure calculations, including actual surcharge loads, structural fill properties, and final configuration of structures, following completion of the project detailed design in an update to the FSAR in accordance with 10CFR 50.71(e) (COM 2.5S-3).

COLA Impact

COLA Part 2, Tier 2, Section 3H.2.6 will be revised.

Staff Assessment

		The applicant's response to Question 03.08.04-1 refers to "at-rest seismic lateral earth pressure." The applicant's to RAI 03.08.04-1 (STP letter U7-C-STP-NRC-090136 dated September 15, 2009 and Section 2.5S.4.10.5.2 of Attachment 3 to the STP Letter U7-C-STP-NRC-090012, dated February 23, 2009), refers to "at-rest seismic lateral earth pressure." The staff reviewed the applicant's response and identified the following four parts in the response: In general, "at-rest" soil pressure relates to static lateral soil pressure on non-yielding walls due to the self-weight of soil (which is the case for STP structures). Hydrostatic pressure and surcharge pressure should be added to the at-rest soil pressure. The dynamic soil pressure has to be calculated separately based on the soil shear wave velocity and added to other static loads (e.g., at-rest, hydrostatic, surcharge, etc.). Therefore, the applicant should further clarify the meaning of "at-rest seismic lateral earth pressure." **This part of the RAI is therefore tracked as Open Item 03.08.04-1**. In regards to dynamic soil pressure calculations for the UHS basin and RSW pump house (Question 03.07.02-12) and on the RB/CB (Questions 03.07.02-11 and 03.07.02-14) due to the SSE, the applicant is currently performing an SSI analysis of these structures and will provide the information later. **This part of RAI will be followed up in Section 3.7**. In regard to the impact of site-specific shear wave velocity on the design of exterior walls of structures with deep foundations such as RB and CB, the applicant in FSAR Revision 3 states that "this impact 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 pressure only affects the out-of-plane load". Although it is true that the design of the walls is controlled by in-plane and out-of-plane actions, it may not be appropriate to conclude that the effect of the lateral soil pressure is insignificant because it is an out-of-plane action only. Walls need to be designed for both in-plane and out-of-plane actions, and the relative significance of each component can be determined only if their magnitudes are known and their effect on design evaluated. Furthermore, the applicant states that "The final earth pressure calculations, including actual surcharge loads, structural fill properties, and final configuration of structures, will be added following completion of the project detailed design in an update to the FSAR in accordance with 10CFR 50.71(e) (COM 2.5S-3)". In order to conclude that the ABWR standard plant structures with deep foundations, such as RB and CB, can be used at the STP site, it is necessary to ensure that the design loads used for these structures envelop the site specific design loads. Lateral soil pressure is one such load. Therefore, the applicant needs to provide the soil pressure computations and demonstrate that the lateral soil pressures at STP units 3 and 4 are enveloped by the lateral soil pressure used in the ABWR Standard Plant design. **This part of**

		the question therefore is tracked as Open Item. Also, the applicant has revised FSAR Section 3H.2.6 of Revision 3 to address the inconsistency in the lateral soil pressure calculation. The following sentence has been deleted from the FSAR: "Shear wave velocity is not used as an input in the calculation of lateral soil pressures. Therefore, change in shear wave velocity has no impact on calculation of the lateral soil pressures". **Therefore, this part of the question is tracked as Closed Item**. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-1 (RAI 2964 Rev.2) The staff reviewed the applicant's response to Question 03.08.04-1 and needs the following additional clarification and information to complete its review: In its response the applicant uses the term "at-rest seismic lateral earth pressure in non-yielding walls". In general, "at-rest" soil pressure relates to static lateral soil pressure on non-yielding walls due to the self-weight of soil including effects due to hydrostatic pressure and surcharge pressure. The dynamic soil pressure is calculated separately and added to the lateral pressure due to static loads (e.g., at-rest, hydrostatic, surcharge, etc.). Therefore, the applicant is requested to clarify the terminology of "At-rest seismic lateral earth pressure" used to describe lateral loads in the response to this RAI. To conclude that the design of structures with deep foundations, such as the Reactor Building (RB) and Control Building (CB), is satisfactory for the site, the site-specific design loads are needed to compare with the design loads used for the DCD. Lateral soil pressure is one such load. Therefore, please provide the lateral soil pressures for the RB and the CB, and compare these calculated pressures with those used in the ABWR standard plant design. Please also confirm if the effects of adjacent structures are considered in computing the lateral soil pressures, and if not, provide justification for not doing so. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-2	Design information for Radwaste Building structure.	**RAI Text** In FSAR Section 3.8.4, page 3.8-2, the applicant references the departure STD DEP T1 2.15-1 that reclassified the Radwaste Building Substructure from seismic category I to non-seismic, and removed all design information from FSAR Section 3.8.4 and Appendix 3H.3. While the staff agrees with the reclassification of the Radwaste Building Substructure as stated in the FSAR, the staff believes that the design information for this building still needs to be included in the FSAR in order for the staff to ensure that the design of the Radwaste Building Substructure has been performed in accordance with the guidance provided in RG 1.143 to meet the regulatory requirements contained in the General Design Criteria 2, and 60 of 10 CFR 50, Appendix A. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** The Radwaste Building (RWB) is a reinforced concrete structure located about 20 feet west of the Reactor Building (RB). It will be designed in accordance with the requirements of Regulatory Guide (RG) 1.143, Revision 2. Also, since the above grade height of this building exceeds the distance to the RB, to ensure that the integrity of the RB is maintained, the RWB design shall satisfy II/I requirements (i.e. it can not collapse or come in contact with the RB under Safe-Shutdown Earthquake (SSE) and tornado loads). The RWB is classified as RW-IIb (Hazardous) in accordance with RG 1.143. Considering the above, the analysis and design of the RWB will be based on the following conservative criteria: A) Criteria for Design Basis: • Design basis analysis and design will be per requirements of RG 1.143, Revision 2 for RW-IIb classification. • Loads, load combinations, codes and standards, and capacity criteria will be in accordance with Tables 1, 2, 3, and 4 of RG 1.143. Design of structural components will be per ACI 349-97 and AISC/N690 (1984).

		Earthquake loading will be per ASCE 7-95 Category III. B) Criteria for II/I evaluation: II/I evaluations will be performed for both SSE and Tornado. Conservatively, the II/I evaluations will be based on elastic design. The seismic response spectra will be the envelope of 0.3g RG 1.60 response spectra and the resulting SSE response spectra at the foundation level of the RWB considering the effect of presence of the RB when subjected to site-specific SSE. This satisfies the requirement noted in item (3) of DCD Tier 2, Section 3.7.2.8. Tornado design parameters will be those defined in DCD for the Standard Plant Seismic Category I structures (i.e. 300 mph tornado). Please note that this exceeds the site specific tornado for Region II (i.e. 200 mph tornado). The analysis and design results will be available for review following the completion of the initial design of the RWB currently scheduled for December 2010. COLA Part 2, Tier 2, Section 3H.3 will be revised as provided below.

3H.3 *Radwaste Building*~~Not Used~~

STD DEP T1 2.15-1

~~Due to the re-classification of the Radwaste Building substructure from seismic Category 1 to non-seismic, this subsection of the DCD, including all tables and figures, has been deleted.~~

The Radwaste Building is a reinforced concrete structure located about 20 feet west of the Reactor building. It is designed in accordance with the requirements of RG 1.143. Also, since the above grade height of this building exceeds the distance to the Reactor building, to ensure that the integrity of the Reactor Building is maintained, the Radwaste Building design shall satisfy II/I requirements (i.e. it can not collapse or come in contact with the Reactor Building under SSE and tornado loads).

The RWB is classified as RW-IIb (Hazardous) in accordance with RG 1.143.

The analysis and design of the Radwaste building are based on the following:

A) Criteria for Design Basis:

- Design basis analysis and design are per requirements of Revision 2 of RG 1.143 for RW-IIb classification.
- Loads, load combinations, codes & standards, and capacity criteria are in accordance with Tables 1, 2, 3, and 4 of RG 1.143.
- Design of structural components is per ACI 349-97 and AISC/N690 (1984).
- Earthquake loading is per ASCE 7-95 Category III.

B) Criteria for II/I evaluation:

- The II/I evaluations are performed for both SSE and Tornado.
- The II/I evaluations are based on elastic design.
- The seismic response spectra are the envelop of 0.3g RG 1.60 response spectra and the resulting SSE response spectra at the foundation level of the Radwaste Building considering the effect of presence of the Reactor Building when subjected to site-specific SSE. This satisfies the requirement noted in item (3) of DCD Tier 2 Section 3.7.2.8.
- Tornado design parameters will be those for the Standard Plant Seismic Category I structures (i.e. 300 mph tornado).

		COLA Impact COLA Part 2, Tier 2, Section 3H.3 will be revised. **Staff Assessment** The applicant's response to Question 03.08.04-2 describes the design requirements for the Radwaste Building. The applicant also states that the RWB is classified as RW-IIb (Hazardous) in accordance with RG 1.143, and the analysis and design are based on the design criteria per RG 1.143. This analysis and design results will be available for NRC review in December 2010. The staff considers this question an open item because the applicant does not include any design parameters regarding loads (e.g., live loads, seismic loads, thermal loads, flood loads, etc.). Also, to be able to evaluate the design of this building, the analysis and the design procedure should be included. Once this information becomes available, the staff will evaluate the applicant's design according to the guidance in RG 1.143. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-2 (RAI 2964 Rev.2) The applicant's response to Question 03.08.04-2 states that the Radwaste Building (RWB) will be designed in accordance with the requirements of RG 1.143, Revision 2. The applicant also discusses the design criteria for this building for seismic category II/I evaluation. In order for the staff to conclude that the Radwaste Building design meets the requirements of RG 1.143, and also meets the requirement in ABWR DCD Section 3.7.2.8, item (3), the FSAR needs to include sufficient design information for the building to demonstrate that the design meets the pertinent design criteria. Guidance provided in SRP Section 3.8.4 may be used for providing such information. Therefore, the applicant is requested to provide design information for the RWB in the FSAR that includes more detailed description of the structure; applicable codes, standards and specifications; loads and load combinations including live loads, seismic loads, thermal loads, flood loads, tornado loads, lateral soil pressure, etc.; design and analysis procedures; structural acceptance criteria; materials and quality control; design of critical sections, stability evaluation, etc. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-3	Removal of seismic category I tunnels from Section 3H.5.3.	**RAI Text** FSAR Section 3H.5.3 has removed the Seismic Category I Tunnels from the items to be included in the Structural Analysis Reports. The only departure referenced in Section 3H.5 (STD DEP T1 2.15-1) does not address this removal. Therefore, the applicant is requested to explain removal of Seismic Category I Tunnels from the Structural Analysis Reports, or identify in this Section where in the application this information may be found. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** Under the Departure STD DEP T1 2.15-1, the Radwaste Building substructure was reclassified from Seismic Category I to non-Seismic Category I. Based on this, the Radwaste Tunnel also is classified as non-seismic Category I. Because of this, the Seismic Category I tunnels were removed from the scope of the Structural Analysis Report in Section 3H.5.3 along with the Radwaste Building substructure. However, the tunnels for the diesel-generator fuel oil piping routed between the storage tanks and the Reactor Building are Category I. Therefore, Section 3H.5.3 is revised to restore the Seismic Category I tunnels. The Departure Report in COLA Part 7 will be revised to clarify that the scope of STD DEP T1 2.15-1 also includes Radwaste Tunnel. Section 3H.5.5 is revised to include Radwaste Tunnels under its scope for the Structural Analysis Reports. The COLA will be revised as provided below.

3H.5.3 Structural Analysis Report for the Reactor Building and Control Building (Including Seismic Category I Tunnels)

3H.5.5 Structural Analysis Report For The Radwaste Building (Including Radwaste Tunnels) and TheTurbine Building

STD DEP 1.8-1

STD DEP T1 2.15-1

For material properties and dimensions, assess compliance of the as-built structure with design requirements in the International Building Code (IBC) for the Turbine Building and Regulatory Guide 1.143 for the Radwaste Building (including Radwaste Tunnels)*and in the Table 3.2-1 and paragraph 3.7.3.16.*

Construction deviations and design changes will be assessed to determine appropriate disposition.

This disposition will be accepted "as-is," provided the following acceptance criteria are met:

- *The structural design meets the acceptance criteria and load combinations of the* IBC *code* for the Turbine Building and Regulatory Guide 1.143 for the Radwaste Building (including Radwaste Tunnels)

The RW/B (including Radwaste Tunnels) and *T/B* are *not classified as Seismic Category 1 structures. However, the buildings* are *designed such that damage to safety-related functions does not occur under seismic loads corresponding to the safe shutdown earthquake (SSE) ground acceleration.*

COLA Impact

The COLA will be revised.

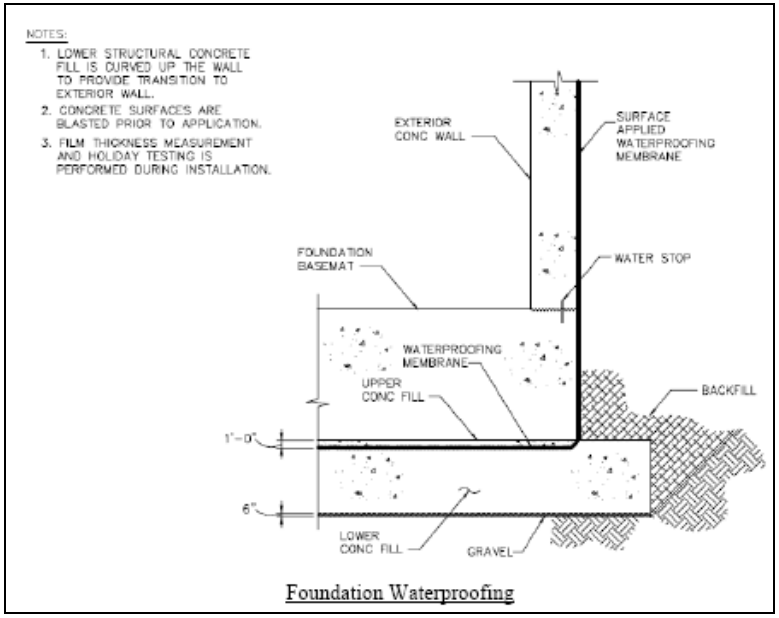
Staff Assessment

		The applicant's response to Question 03.08.04-3 states that FSAR Section 3H.5.3, Revision 3, has been revised to include the Seismic Category I tunnels in the structural analysis reports. The applicant also notes that the Departure Report in COLA Part 7 will be revised to clarify that the scope of STD DEP T1 2.15-1 also includes the Radwaste Tunnel. Furthermore, FSAR Section 3H.5.5 has been revised to include Radwaste Tunnels within the scope of the structural analysis reports. The staff found this response acceptable since the applicant proposed to revise FSAR Section 3H.5.3 to include Seismic Category I Tunnels, and provided FSAR mark-up in the response that was reviewed by the staff. The proposed change has since been incorporated in FSAR Revision 3. Therefore, this item is closed. **Follow-up Question to the Partial Response** None. **Status** Closed.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-4	Inconsistency in Section 3H.5.5 regarding the code used for RWB structure design.	**RAI Text** ABWR Standard Plant Design Control Document Section 3H.5.5 was titled "Structural Analysis Report for the Turbine Building," and contained references to the Uniform Building Code (UBC) for design requirements. In FSAR Section 3H.5.5, the applicant changed the title to read "Structural Analysis Report for the Radwaste Building and the Turbine Building," and did not identify any departure addressing the change. Further, the subsection refers to the UBC (changed to International Building Code (IBC) via STD DEP 1.8-1) for design requirements for both buildings. However, in the description of departure STD DEP T1 2.15-1 in Part 7 of the application, the applicant stated that "The detailed guidance for the design of the radwaste processing systems, structures, and components is provided in Regulatory Guide 1.143. This departure commits to follow the guidance of Regulatory Guide 1.143." The staff requests the applicant to explain this apparent inconsistency. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** The STD DEP T1 2.15-1 is listed under Section 3H.5.5 and the text revised to show compliance with Regulatory Guide 1.143 for the Radwaste Building. This information is provided in Section 3H5.5 as enclosed in the response to RAI 03.08.04-3. No additional COLA change is required for this response. **COLA Impact** None. **Staff Assessment** The applicant's response to Question 03.08.04-3 revised the title for Section 3H.5.5 to read, "Structural Analysis Report for the Radwaste Building (Including Radwaste Tunnels) and the Turbine Building" to correctly identify the scope of this section. The applicant also revised the text in this section to correctly refer to RG 1.143 for design of Radwaste Building, and removed the inconsistency identified in the question. Further, the applicant also included

		reference to STD DEP T1 2.15-1 in this section. Therefore, all issues identified in the question are addressed. The proposed changes have been incorporated in FSAR Revision 3, and this question is closed. **Follow-up Question to the Partial Response** None. **Status** Closed.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-5	COL License Information Item 3.23 - Foundation waterproofing.	**RAI Text** In FSAR Section 3.8.6.1, page 3.8-4, the applicant addressed the COL License Information Item 3.23, and stated that foundation waterproofing is done by placing a chemical agent on the exposed concrete surface of the mudmat, and the concrete foundation is poured directly onto the concrete mudmat. Also, in FSAR Section 3H.6.6.4 the applicant stated that a chemical waterproofing agent will be applied to the exposed concrete surface of the mudmat for site-specific category I structures, and, in addition, a waterproof membrane will be installed on the walls up to one foot below grade, with a waterproof coating being applied from that level up to the flood level. The staff requests the applicant to provide the following information in order to understand the effectiveness of the proposed foundation waterproofing: 1. Provide details of the chemical agent proposed to be used, how it will be applied, and how it will accommodate any potential cracking of the mudmat due to placement of the massive concrete foundation and still be effective as foundation waterproofing. Provide information to support that this type of waterproofing is adequate to protect the concrete foundations against degradation due to aggressive soil/groundwater. 2. Provide the value of the coefficient of friction assumed between the concrete foundation and the mudmat with the chemical agent applied on top, the basis for the assumed value, and how it compares with the coefficient of friction assumed in the standard ABWR design in determining the factor of safety against sliding. 3. Describe in detail the type of waterproofing membrane proposed to be used including operating experience with use of such membranes at the site or elsewhere, and vendor or operating experience data which demonstrate that the type of waterproofing membrane retains adequate water-retarding properties under aggressive soil conditions comparable to the site for long period of time without degrading. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** (a) The waterproofing system is an elastomeric “spray-on” waterproof membrane. The membrane is applied as a high-viscosity liquid that cures after exposure to air. This material may be applied by brush, roller or airless spray equipment. The specific material for the waterproof membrane will be selected during detailed design. The waterproofing will be placed in the concrete fill as described below and as shown in the attached figure.



Foundation Waterproofing

The Reactor Building and the Control Building will be founded on structural concrete fill. The structural concrete fill is ten feet thick for the Reactor Building and two feet thick for the Control Building. A layer of gravel will be placed on the excavated foundation surface. The structural concrete fill will be placed over the gravel to about one foot below its finished elevation. When this lower structural concrete fill has reached the specified strength, a layer of waterproof membrane will be applied to the entire top of the slab. A portion will be extended vertically up to meet the wall surface. The final portion (about one foot high) of the structural concrete fill will then be placed, sandwiching the waterproof membrane.

Rebar and foundation embedment are not incorporated in either of these structural concrete fill lifts; therefore installation of such elements will not puncture the waterproofing membrane.

		Final thickness of the membrane will be specified based on the physical properties of the selected material but is expected to be on the order of 80 to 120 mils. The membrane may be applied in multiple coats to achieve the required thickness. The surface of both the structural concrete fill and the exterior walls will be prepared in accordance with procedures that are consistent with the surface preparation requirements determined during the material qualification testing program described in Item (b), below. At the transition between the lower structural concrete fill and the exterior wall, a small transition between the structural concrete fill and wall will be provided to allow a smooth transition for the membrane. 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. It should be noted that the cured membrane has a degree of flexibility which allows it to accommodate mudmat concrete shrinkage and thermal cracking, thermal expansion and other minor movements between substrate members. The application procedures will address all aspects of the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repair procedures. (b) The coefficient of friction of the waterproofing material will be determined by testing and will be sufficient to transfer site-specific safe-shutdown earthquake seismic loads. Since the waterproofing material is a COL license item and 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 than those considered for the standard ABWR design. The coefficient of friction will be determined with a qualification program prior to procurement of the membrane material. The qualification program will be developed to demonstrate that the selected material will meet the waterproofing and friction requirements. This qualification program will address, as a minimum, the following: • chemical properties of the membrane material, • physical properties of the membrane material, • surface finish and preparation requirements, and

- installation procedures necessary to achieve the required properties and coefficients of friction.

The qualification program will include testing to demonstrate that the waterproofing requirements and the coefficient of friction required to transfer seismic loads for STP 3 & 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 - waterproof membrane structural interface. A technical report will document the basis for determining that the material will meet the required friction factor and waterproofing requirements.

Application procedures will be developed based on the results of qualification testing to assure that the conditions and assumptions of the qualification tests are maintained during product application.

(c) 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 due to soil/groundwater conditions at the STP 3 & 4 site.

COLA Part 2, Tier 2, Section 3.8.6 will be revised as provided below.

3.8.6 COL License Information

3.8.6.1 Foundation Waterproofing

The following standard supplement addresses COL License Information Item 3.23.

Foundation waterproofing is done by placing a ~~chemical agent on the exposed concrete surface of the mudmat.~~ waterproofing membrane near the top elevation of the concrete fill. The ~~concrete foundation is poured directly onto the concrete mudmat.~~ remainder of the concrete fill is then poured on top of the waterproofing material. A waterproof membrane that could degrade the ability of the foundation to transfer loads is not used.

COLA Impact

		COLA Part 2, Tier 2, Section 3.8.6 will be revised. **Staff Assessment** The applicant's response to Question 03.08.04-5 provides summary responses to Items (a), (b), and (c). The applicant states that additional descriptions will be provided during the detailed design phase. Based on the guidance of SRP 3.8.5, the applicant needs to show that the foundation is capable of receiving a load and transmitting it from the structure to soil media with an appropriate factor of safety. Also per the guidance of SRP 3.8.5, if a new material not used in previously licensed cases is employed, the applicant should provide sufficient test and user data to establish the acceptability of that material. Therefore, the applicant is requested to provide quantitative data along with test information to show the effectiveness of the proposed foundation waterproofing. Therefore, this question is being tracked as an open item. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-5 (RAI 2965, Rev.2) The applicant's response to Question 03.08.04-5 regarding placing a chemical agent on the exposed concrete surface of the mudmat provides descriptive explanations of the waterproofing. Per the SRP 3.8.5 guidance, the applicant needs to show that the foundation can transfer the forces from the structure to soil with the proper factor of safety. Also, because a new material is being used, the applicant needs to provide additional data on testing and other relevant information to meet guidance of SRP 3.8.5. Therefore, the applicant is requested to provide the following additional information, and update FSAR as appropriate: the specific material that will be used for the waterproof membrane; sufficient 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 final thickness of the membrane based on the physical properties of the selected material the application procedures for all aspects of the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repairs

		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 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; and documentation summarizing the basis for determining that the material will meet the friction factor and waterproofing requirements site-specific sliding evaluation for the Reactor Building (RB) and the Control Building (CB) to demonstrate that the minimum coefficient of friction needed for maintaining the minimum factor of safety against sliding is available at all sliding interfaces between the structures and foundation soil specification and properties of the structural concrete fill below the RB and CB foundations **Status** Open

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-6	COL License Information Item 3.25 - Details of Containment Structural Integrity Test.	**RAI Text** In FSAR Section 3.8.6.3 the applicant addressed the COL License Information Item 3.25 regarding the Structural Integrity Test (SIT), and stated that the details of the test and instrumentation, as required for such test, will be provided to NRC for approval. The applicant is requested to include this information in this section, or provide information about when this will be available for review, and what tracking mechanism is established to ensure compliance. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** Details of the Test and Instrument Plan for the SIT (such as specific locations designated for recording displacements, strains and temperature during the test) will be defined in the Construction Specification. 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 prototype 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. The following is a summary of SIT requirements for STP 3 & 4 based on Article CC-6000 of ASME Section III, Division 2. These will be included in the ASME Construction Specification for the Containment. Details of the Test: The containment is subjected to integrity tests that include both an overall internal pressure test and a differential pressure test. The overall SIT will be performed at a test pressure of at least 1.15 times the containment design pressure in both the drywell and suppression chamber simultaneously. The differential pressure test will be performed at a test pressure of at least 1.0 times the maximum design differential pressure. The test pressure will be held for at least 1 hour. Predictions of displacements and strains will be made prior to the start of the Unit 3 test.

		During the SIT tests, the suppression chamber and spent fuel pool will be filled with water to the normal operational water level. Atmospheric air will be used as the testing medium for both the overall and the differential pressure test. The Designer or his designee will perform a pretest visual examination of the accessible portions of the primary containment vessel prior to the structural integrity (SI) test in accordance with CC-6210 of ASME Section III, Division 2. The Designer or his designee will witness the SI test and will monitor displacement measurements. Instrumentation: Instrumentation for the measurement of pressure, displacement, strain, crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing. Displacement, strain (for Unit 3), and temperature measurements will be made in accordance with CC-6300 of ASME Section III, Division 2. Displacement, strain, and temperature will be recorded at the locations specified in the test and instrument plan as defined in the Construction Specification. The test plan will be available prior to start of construction of the concrete containment so that sufficient time is available for placement of instrumentation to be embedded in concrete or otherwise installed during construction. The primary containment will be pressurized and depressurized at rates not to exceed 20% of the test pressure per hour in accordance with CC-6321 of ASME Section III, Division 2. Test data will be collected in accordance with CC-6340 of ASME Section III, Division 2. For the prototype Unit 3 Containment, strains and associated temperatures will be measured for a minimum period of 24 hours prior to the SI test to evaluate the strain variations resulting from temperature change. Concrete crack patterns will be mapped at locations specified by the Designer before the tests, at maximum pressure, and after the tests in accordance with CC-6350 of ASME Section III, Division 2. Mapped areas will include areas where high surface tensile strain is predicted. A post-test examination will be made within one (1) week of depressurization. Details of the post-test examination will

		be the same as those of the pretest examination required by CC-6210 of ASME Section III, Division 2. Test Acceptance Criteria: Crack and strain (for Unit 3) measurements will be reviewed by the Designer for evaluation of the overall test results. The primary containment will be considered to have satisfied the structural integrity test if the minimum requirements specified in CC-6410 of ASME Section III, Division 2 are met. If measurements and studies by the Designer indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2. Structural Integrity Test Report: The results of structural integrity tests will be submitted to the Designer. The report will meet the minimum requirements of CC-6530. COLA Part 2, Tier 2, Section 3.8.6.3 will be revised as provided below.

3.8.6.3 Structural Integrity Test Result

The following standard supplement addresses COL License Information Item 3.25.

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 first containment will be considered a prototype and its SIT performed accordingly. The details of the test and the instrumentation, as required for such a test, will be provided ~~to NRC for approval.~~ in the ASME Construction Specification. The test and instrument plan for the Unit 3 SIT will conform to the requirements for prototype 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.

3.8.6.3.1 Details of the Test:

The containment is subjected to integrity tests that include both an overall internal pressure test and a differential pressure test. The overall SIT will be performed at a test pressure of at least 1.15 times the containment design pressure in both the drywell and suppression chamber simultaneously. The differential pressure test will be performed at a test pressure of at least 1.0 times the maximum design differential pressure. The test pressure will be held for at least 1 hour. Predictions of displacements and strains will be made prior to the start of the Unit 3 test.

During the SIT tests, the suppression chamber and spent fuel pool will be filled with water to the normal operational water level. Atmospheric air will be used as the testing medium for both the overall and the differential pressure test. The Designer or his designee will perform a pretest visual examination of the accessible portions of the primary containment vessel prior to the structural integrity (SI) test in accordance with CC-6210 of ASME Section III, Division 2. The Designer or his designee will witness the SI test and will monitor displacement measurements.

3.8.6.3.2 Instrumentation:

Instrumentation for the measurement of pressure, displacement, strain, crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing.

Displacement, strain (for Unit 3), and temperature measurements will be made in accordance with CC-6300 of ASME Section III, Division 2. Displacement, strain, and temperature will be recorded at the locations specified in the test and instrument plan as defined in the Construction Specification. The test plan will be available prior to start of construction of the concrete containment so that sufficient time is available for placement of instrumentation to be embedded in concrete or otherwise installed during construction.

The primary containment will be pressurized and depressurized at rates not to exceed 20% of the test pressure per hour in accordance with CC-6321 of ASME Section III, Division 2.

Test data will be collected in accordance with CC-6340 of ASME Section III, Division 2. For the prototype Unit 3 Containment, strains and associated temperatures will be measured for a minimum period of 24 hours prior to the SI test to evaluate the strain variations resulting from temperature change. Concrete crack patterns will be mapped at locations specified by the Designer before the tests, at maximum pressure, and after the tests in accordance with CC-6350 of ASME Section III, Division 2. Mapped areas will include areas where high surface tensile strain is predicted.

A post-test examination will be made within one (1) week of depressurization. Details of the post-test examination will be the same as those of the pretest examination required by CC-6210 of ASME Section III, Division 2.

3.8.6.3.3 Test Acceptance Criteria:

Crack and strain (for Unit 3) measurements will be reviewed by the Designer for evaluation of the overall test results. The primary containment will be considered to have satisfied the structural integrity test if the minimum requirements specified in CC-6410 of ASME Section III, Division 2 are met. If measurements and studies by the Designer indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2.

3.8.6.3.4 Structural Integrity Test Report:

The results of structural integrity tests will be submitted to the Designer. The report will meet the minimum requirements of CC-6530.

COLA Impact

COLA Part 2, Tier 2, Section 3.8.6.3 will be revised.

Staff Assessment

The applicant's response to Question 03.08.04-6 provides description of the Structural Integrity Test (SIT), and states that the details of the test and the instrumentation will be provided in the ASME Construction Specification and the test performed in accordance with ITAAC Table 2.14.1, Item #3. The applicant also states that the Test and Instrument Plan for the STP Units 3 and 4 SIT will conform to the requirements for prototype containments delineated in Article CC-6000 of ASME Section III, Division 2. The applicant also proposed to revise FSAR Section 3.8.6.3 and described details of the test and requirements for instrumentation, acceptance criteria and test report, and provided a mark-up of the FSAR. Since the applicant included description and requirements for the SIT in the FSAR following requirements of ASME Section III, Division 2, Article CC-6000, and the test will be performed according to ITAC Table 2.14.1, Item #3 using details of the test to be included in the ASME Construction Specification, the staff considers the response to be technically acceptable. However, this question is a confirmatory item pending incorporation of the proposed FSAR mark-up in a future FSAR revision.

		Follow-up Question to the Partial Response Follow-up 1 to Question 03.08.04-6 (RAI 2965, Rev.2) The applicant states in its response to Question 03.08.04-6 that the details of the Structural Integrity Test (SIT) and the instrumentation required for the test will be provided in the ASME Construction Specification, but does not indicate when the information will be available for review by the staff. Since COL License Information Item 3.25 requires that the applicant provide the details of the SIT and the instrumentation for review and approval by the NRC, the applicant is requested to either provide the information for staff review, or provide plans to meet the requirements of the license information item using guidance provided in RG 1.206, Section C.III.4.3. **Status** Confirmatory pending inclusion of the modification to the FSAR.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-7	Discrepancy in designation of Figures, illegible Figures.	**RAI Text** In FSAR Section 3H.6.1, the applicant stated that the site specific seismic category I structures are shown in Figures 1.2-32 through 1.2-37. The staff has the following questions regarding these figures: (a) Figures 1.2-32 and 1.2-33 show the arrangement drawing for the Turbine Building, which is not a seismic category I structure. The applicant needs to correct the reference to the figures. (b) Figures 1.2-34 through 1.2-37 are not legible. The applicant needs to provide legible copies of these drawings. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** The correction to the typographical error in figure numbers (from Figures 1.2-32 through 1.2-37 to Figures 1.2-34 through 1.2-36), and legible copies of the revised Figures 1.2-34 through 1.2-36 are provided in Section 3H6.1 as enclosed in the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009). No additional COLA change is required for this response. **COLA Impact** None. **Staff Assessment** The applicant's response to Question 03.08.04-7 corrects the figure numbers and provides legible copies of the requested figures. The staff considers this response acceptable, and this question is closed. **Follow-up Question to the Partial Response** None. **Status**

		Closed.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-8	Incomplete list of codes and standards in Section 3H.6.4.1.	**RAI Text** In FSAR Section 3H.6.4.1, the applicant described the design codes and standards to be used for site-specific seismic category I structures. The list appears to be incomplete, since it does not contain any steel code, welding code, and the regulatory guides that are usually listed in this section. Therefore, the applicant is requested to confirm that the list provided includes all major codes and standards which will be used for design of site-specific seismic category I structures. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** The design codes and standards and the regulatory guides used in the analysis and design of site-specific structures are provided in Section 3H.6.4.1 as enclosed in the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009). In addition to the regulatory guides listed in this section, the regulatory guides listed in FSAR Table 1.9S-1 are also used, as applicable. No additional COLA change is required for this response. **COLA Impact** None. **Staff Assessment** The applicant's response to Question 03.07.01-13 revises FSAR Section 3H.6.4.1 to include the following: American National Standard Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities (ANSI/AISC N690) Structural Welding Code – Steel (AWS D1.1) Regulatory Guide 1.76, Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants Regulatory Guide 1.61 – Damping Values for Seismic Design of Nuclear Power Plants

		The staff considers these codes acceptable, and since the FSAR changes are incorporated in FSAR Revision 3, this question is closed. **Follow-up Question to the Partial Response** None. **Status** Closed.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-9	Consideration of 25% of live load in seismic loading combinations.	**RAI Text** In FSAR Section 3H.6.4.3.1.2, the applicant stated 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% of the operating floor and roof live loads." SRP 3.7.2, SRP Acceptance Criteria 3.D, recognizes the use of 25% of the floor design live load in the dynamic model for computation of global seismic loads only. Therefore, the applicant is requested to provide detailed justification as to why seismic load combinations for design of seismic category I structures need to consider only the normal plant operating condition when only 25% of the design live load is assumed to be present. Also, the applicant is requested to describe the basis for the assumption that only 25% of the design live load would be present during normal plant operation, and demonstrate that the assumption meets industry standards for consideration of minimum live load to be used for design of seismic category I structures. **Application Response** **Reference File<U7-C-STP-NRC-090128.pdf>** As described in Section 3H.6.4.3.1.2 of the enclosure provided in the response to RAI 03.07.01-11, the only areas of the site-specific Seismic Category I structures requiring consideration of a live load are the operating floors and roof of the pump house and the floors of the Reactor Service Water (RSW) Piping Tunnels. The following normal live loads are used: Operating floors of Pump House: 200 psf (9.6 kPa) Roof of Pump House: 50 psf (2.4 kPa) RSW Tunnel floors: 200 psf (9.6 kPa) For computation of the global seismic loads, the live load is limited to the expected live load present during normal plant operation, L,. This load has been defined as 25% of the normal live loads shown above. However, design of local elements such as beams and slabs is based on consideration of full normal live load. This is similar to the criterion described in DCD Tier 2 Section 3.8.1.3.1.

There is no additional COLA revision required as a result of this response.

COLA Impact
None.

Staff Assessment
In its response to question 03.08.04-9 the applicant stated that for computation of global seismic load, the live load is limited to the expected live load present during normal plant operation. The applicant defined this load to be 25% of the normal live load. The applicant also stated that design of local elements such as beams and slabs is based on consideration of full normal live load. However, the applicant did not clearly state whether design of local elements will consider full live load for all governing load combinations including loading combinations involving seismic loads. From review of the various loading combinations described in FSAR Section 3H.6.4.3.4, it is noted that the reduced live Lo is used for loading combinations involving seismic loads. This is contrary to the guidance provided in SRP 3.8.4 and ACI 349.

Follow-up Question to the Partial Response
Follow-up 1 to Question 03.08.04-9 (RAI 2965, Rev.2)

In response to Question 03.08.04-9, the applicant states that for computation of global 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% of the operating floor and roof live loads. In FSAR Section 3H.6.4.3.4, the applicant has used a full live load for load combinations not involving a seismic load, and Lo for loading combinations involving seismic load. Although it is acceptable to consider 25% of design live load for computation of global seismic loads, the basis for considering only 25% of live load in loading combinations involving seismic load is not understood. The load combination that includes the seismic load needs to include the full live load effects per the guidance of SRP 3.8.4 and ACI 349. Therefore, the applicant is requested to clarify the use of a reduced live load (expected live load) in the seismic load combinations.

Status
Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-10	Including extreme snow load as an extreme environmental load in Section 3H.6.4.3.3.	**RAI Text** In FSAR Section 3H.6.4.3.3, "Extreme Environmental Load," the applicant included the tornado loads and the seismic loads. According to the guidance provided in the Interim Staff Guidance ISG-7 recently issued for public comments, the snow load due to the extreme winter precipitation event should also be considered as an extreme environmental load. Therefore, the applicant is requested to include the snow load due to the extreme winter precipitation event in this section, or provide justification for not doing so. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** Extreme snow load information is provided in Section 3H.6.4.3.3.5 as enclosed in the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009). No additional COLA change is required for this response. **COLA Impact** None. Staff Assessment The applicant's response to Question 03.08.04-10 provides the values of extreme snow load in Revision 3 of FSAR Subsection 3H.6.4.3.3.5 as 5.5 psf, which are based on a load equivalent to one-inch of water in FSAR Subsection 2.3S1.3.4. The applicant's response to RAI 03.08.04-14 states that "the maximum height of the parapet provided on top of ABWR standard plant safety-related buildings is less than nine inches". Therefore, accumulated water on the roof with a parapet would lead to the maximum water load of 47 lbs/sq. ft (9 inches of water). Therefore, the applicant is requested to elaborate, in this section, on the extreme snow load used in the load combination for the roof design with parapet. **Follow-up Question to the Partial Response**

		Follow-up 1 to Question 03.08.04-10 (RAI 2965, Rev.2) In FSAR Subsection 3H.6.4.3.3.5, Revision 3, the applicant defines extreme snow load (S_E) as 5.5 psf. The applicant has subsequently used this S_E in loading combinations. However, for load combinations involving extreme snow, the roof load due to an extreme winter precipitation event per ISG-7 should be considered. According to the applicant's response to Question 03.08.04-14, this load was determined to be 47 lbs/ft^2 based on the maximum accumulated water on roof during an extreme winter precipitation event. Therefore, the applicant is requested to elaborate in this section how the extreme snow load used in load combination for roof design was determined following the guidance provided in ISG-7, and report the design load to be used in load combination for roof design. This information is needed to establish consistency between load definition and its use in corresponding load combination. **Status** Open.

Tracking Table of South Texas Project (STP) Response to RAIs Related to Section 3.8		Date: 12/15/09	
Reviewer:	Hassan Sedarat	Revision No:	0
Based on RAI dated	11/18/2009	Revision Date:	12/15/2009
Based on TER	11/27/2009	Page:	*53 of 71*
Based on follow up question dated:	11/30/2009		

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-11	Performing overall damage prediction due to tornado loads.	**RAI Text** In FSAR Section 3H.6.4.3.3.1, “Tornado Loads (Wt),” item 3(b), the applicant stated that “the global overall damage prediction will be performed during the detailed design phase in accordance with Section 3.5.3.2.” Since all seismic category I structures must be designed for the effects due to the design basis tornado to meet the requirements of the General Design Criterion 2 of Appendix A to 10CFR50, the staff expects to see this evaluation performed before licensing. Please submit the evaluation for Staff review, or justify why it need not be provided. **Application Response** **Reference File<U7-C-STP-NRC-090136.pdf>** Listed in COLA Part 2, Tier 2, Section 3H.6.4.3.3.1 are the tornado parameters used for both local and global evaluations. Also listed are the parameters used for computation of tornado wind pressure. All these parameters are in accordance with Regulatory Guide 1.76, Revision 1, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants"and NUREG-0800, Revision 3, Standard Review Plan (SRP), Section 3.3.2. Section 3H.6.4.3.3.1 as enclosed in the response to RAI 03.07.01-3 provides global overall damage evaluations performed in accordance with SRP Section 3.5.3. In these evaluations, the tornado loads (i.e. Wt) to be included in combination with other applicable loads are per combination Wt = Ww + 0.5Wp + Wm (i.e. Ww = tornado wind pressure, Wp = tornado differential pressure, and Wm = load due to missile impact). For any critical missile hit location considered, the structure is analyzed for the resulting equivalent static load due to tornado missile impact in conjunction with tornado wind pressure and 50% of tornado differential pressure. The resulting induced forces and moments from this analysis are combined with the induced forces and moments due to other applicable loads within the load combination to determine the total demand for design of the structural elements. These analyses and design results will be provided in a supplemental response to RAI 03.07.01-13. The supplemental response is currently scheduled by December 31, 2009, in accordance with the schedule provided in Attachment 1 of letter U7-C-STP-NRC-090112 dated August 20, 2009.

		No additional COLA change is required for this response. **COLA Impact** None. **Staff Assessment** The applicant in its response described the analysis methodology used for tornado load, and stated that analysis and design results will be provided in a supplemental response to question 03.07.01-13 by December 31, 2009. This question is an open item, pending a review upon receipt of the applicant's response. **Follow-up Question to the Partial Response** None. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-12	Incomplete list of loads for design of site-specific structures.	RAI Text In FSAR Section 3H.6.4.3, "Design Loads and Load Combinations," the applicant described the various loads and load combinations that will be used for design of site-specific seismic category I structures. However, this section does not include any description 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 structures. Therefore, the applicant is requested to include the above information in this section, or provide justification for not doing so. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** The thermal loads, loads due to the probable maximum flood, hydrostatic loads, and calculated lateral soil pressures are provided in the following sections as enclosed in the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009): Thermal Loads: Section 3H.6.4.3.1.5 Loads due to PMF: Section 3H.6.4.3.3.4 Hydrostatic Loads: Section 3H.6.4.3.1.6 Lateral Soil Pressure: Sections 3H.6.4.3.1.4 and 3H.6.4.3.3.3 No additional COLA change is required for this response. **COLA Impac**t None. **Staff Assessment**

		The applicant's response to Question 03.08.04-12 refers to the following subsections of the FSAR mark-up provided with the response submitted for RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112, dated August 20, 2009): Thermal Loads: Subsection 3H.6.4.3.1.5 Loads Due To PMF: Subsection 3H.6.4.3.3.4 Hydrostatic Loads: Subsection 3H.6.4.3.1.6 Lateral Soil Pressure: Subsections 3H.6.4.3.1.4 and 3H.6.4.3.3.3 However, a review of these subsections indicates that the thermal, hydrostatic and lateral soil pressure load values are not specified in the above subsections, which only provide definitions of the terms. Therefore, the applicant is requested to provide the values of the thermal, hydrostatic and lateral soil pressure loads that are used in the analysis. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-12 (RAI 2965, Rev.2) The applicant's response to Question 03.08.04-12 refers to the response submitted for RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112, dated August 20, 2009). However, a review of the FSAR subsections identified in that response reveals that the response provided only a definition of these loads, and the thermal, hydrostatic and lateral soil pressure load values are not provided. Therefore, the applicant is requested to include in the FSAR the values of the thermal, hydrostatic and lateral soil pressure loads that are used in the analysis. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-13	Structural analysis and design information for all site-specific seismic category I structures.	**RAI Text** In FSAR Section 3H.6.6.1, "Structural Analysis and Design Summary," the applicant stated that "the structural analysis of the UHS Basin, UHS cooling tower enclosures, and RSW pump houses will be performed using a three-dimensional finite element model of the structures… A separate model will be developed for use in the evaluation of the RSW piping tunnels …" The applicant has not performed these analyses yet, and has not provided any final design details and results for these structures in the application. Therefore, the applicant is requested to include structural analysis and design information for all site-specific seismic category I structures in the FSAR using the guidance provided in SRP 3.8.4, and other applicable SRP sections and guidance documents. **Application Response** **Reference File<U7-C-STP-NRC-090105.pdf>** The structural analysis and design information for the site-specific seismic category I structures is provided in Section 3H.6 as enclosed in the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009). No additional COLA change is required for this response. **COLA Impact** No additional COLA change is required for this response. **Staff Assessment** In its response to question 03.08.04-13, the applicant referred to the revised FSAR Section 3H.6 provided in response to question 03.07.01-13 (Letter U7-C-STP-NRC-090112 dated August 20, 2009). The structural analysis and design information for site-specific structures pertinent to this question is included in FSAR Section 3H.6.6. The staff reviewed this section of the FSAR included in response to question 03.07.01-13, and noted that the response was lacking in providing details about the structural analysis and design information for site-specific structures. Also, many tables and figures referenced in the response were not included. The staff has requested information pertaining to analytical model, analytical approach, and stability evaluation for the site-specific structures in questions 03.07.02-2 through 03.07.02-10, and these will be reviewed in response to these questions. Therefore, only information

		pertaining to structural design for the site-specific structures will be evaluated in this question. The applicant has not included information pertaining to structural design results for site-specific structures in the above referenced response and included limited description about how the various elements of the site-specific structures are designed. Therefore, the applicant was requested to include in FSAR Section 3H.6.6.3 clear description about how the various elements of the site-specific structures were designed providing level of detail similar to that included in the ABWR DCD. This question is, therefore, considered to be an open item. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-13 (RAI 2965, Rev.2) In its response to Question 03.08.04-13, the applicant referred to FSAR mark-up provided in response to question 03.07.01-13 for structural analysis and design information for site-specific seismic category I structures (Letter U7-C-STP-NRC-090112 dated August 20, 2009). The staff noted that the above referenced response did not include all tables and figures referenced in the FSAR mark-up, and these are stated to be provided later. In addition, the level of detail included in FSAR Section 3H.6.6.3 regarding structural design of the various elements of site-specific structures is not sufficiently descriptive, and is not similar to that included in the ABWR DCD. Therefore, the applicant is requested to include in FSAR Section 3H.6.6.3 description of the various steel and concrete elements of the site-specific structures including how these elements are designed including design results. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-14	Extreme snow load for standard plant structures.	**RAI Text** The response to RAI 140, Question 02.03.01-6 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. Please provide details of 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 will prevent no more than 2.394 kPa (50 lbf/ft2) of water accumulation on the roof. **Application Response** **Reference File<U7-C-STP-NRC-090147.pdf>** The roofs of these buildings, which are required to resist tornado generated missiles, are reinforced concrete slabs with minimum thickness of 14 inches. The maximum height of the parapet provided on top of ABWR standard plant safety-related buildings is less than nine inches. Therefore, the maximum accumulated water height on top of these roofs will not exceed nine inches. The deflection of these roofs due to load corresponding to nine inches of water (i.e. 47 lb/ft2, which is less than the 50 lb/ft2 roof design live load) will be insignificant and thus ponding is not a concern. Therefore, even if the roof scuppers and roof drains clog, the roof loading will not exceed 2.394 kPa (50 lb/ft2). No COLA revision is required as a result of this response. **COLA Impact** None. **Staff Assessment** The applicant states that the maximum height of the parapet is less than nine inches. For 9 inches, the load due to water is 47 lb/ft^2, which is less than 50 lb/ft^2. It is not clear why any incidental live load on roof need not be considered during the extreme winter precipitation event. Also, the FSAR should include the information that the maximum parapet height is 9 inches and is the basis for computing the extreme winter precipitation load on roofs. Follow-up Question to the Partial Response

		Follow-up 1 to Question 03.08.04-14 (RAI 3323, Rev.2) The applicant's response to Question 03.08.04-14 explained that since the maximum parapet height for ABWR standard plant seismic category I structures is 9 inches, roof load during the extreme winter precipitation event may not exceed 47 lbs/ft^2, which is less than the roof design live load of 50 lbs/ft^2. The applicant is requested to explain why any potential incidental live loads on the roof need not be considered concurrent with the extreme winter precipitation event. Also, since the maximum parapet height of 9 inches is used as the basis for computing the extreme winter precipitation load on the roof, the applicant is requested to include this information in the FSAR. The requested information will establish the adequacy of roof design live load, and include in the FSAR critical design information. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-15	Design of interface between Service Water Tunnel and Control Building and Pump House.	**RAI Text** In FSAR Section 3H.6.3.4, "Reactor Service Water Piping Tunnels," the applicant stated that "The interfaces between the tunnels and the pump houses and control buildings are configured to allow relative movement between the tunnels and structures." Please provide a description of the interface configuration between the tunnels and the pump houses and the control buildings. Please also describe the analysis and design methodology for the interface including the loadings and load combinations used, and the amount of relative movement considered in the design along with technical basis, and demonstrate that the flexible connection used at the interface is adequate for the design loads and deformations. **Application Response** **Reference File<U7-C-STP-NRC-090160.pdf>** A conceptual detail of the interface between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings is shown in the attached Figure 03.08.04-15A. This 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 interfaces will be designed to the applicable loads and loading combinations described in COLA Part 2, Tier 2, Section 3H.6.4.3. The calculations and design of the interface will be finalized as part of the detailed design. No COLA change is required as a result of this RAI response. **COLA Impact** None. **Staff Assessment**

		The applicant's response to RAI 03.08.04-15 provides a schematic that shows the conceptual detail of the interface between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings. The applicant states that the design will be finalized in the detailed design stage. The schematic does not (1) show sizes or dimensions, (2) provide sufficient information for extracting capacity of the joint, or (3) demonstrate an adequate safety factor for the joint relative displacement. Also, it is not clear from this schematic whether the tunnel can become misaligned during a load condition, and if so, what the consequences are and how the tunnel will be realigned. The applicant is requested to address all of these concerns, and include in the FSAR relevant design information. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-15 (RAI 3323, Rev.2) The applicant's response to Question 03.08.04-15 provides a conceptual design for the interface connection between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings. The applicant states that the interface design will be finalized during detailed design. The response does 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, the applicant is requested 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 is 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. Please also address potential degradation of the interface material due to groundwater, in-service inspection of the interface material, and measures against potential in-leakage of groundwater. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.04-16	Modeling of hydrodynamic loads for UHS basin. Also, not listing thermal and flood loads for design of UHS structure.	**RAI Text** In FSAR Section 3H.6.6.2.1 the applicant described the loadings used for design of the Ultimate (UHS) Basin, UHS Cooling Tower Enclosure, and the Reactor Service Water (RSW) Pump House. Please explain why flood loads and temperature loads are not included in the list of loads considered for design of these structures. Also, please provide details of how the hydrodynamic loads were calculated and applied to the finite element model and discuss if the SRP 3.7.3 Acceptance Criteria 14 were met, or provide justification for not doing so. **Application Response** **Reference File<U7-C-STP-NRC-090147.pdf>** The flood loads and temperature loads have been included in the load combination considered for design of these structures, as described in the following sections of the enclosure to the response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009): Temperature Loads: Section 3H.6.4.3.1.5 Loads due to PMF: Section 3H.6.4.3.3.4 A soil structure interaction analysis has been performed in accordance with the guidance provided in Standard Review Plan (SRP) Section 3.7.2. The details of the analytical approach are provided in Section 3H.6.5.2.4 as provided with response to RAI 03.07.01-3 (see letter U7-C-STP-NRC-090136 dated September 15, 2009). Acceptance Criterion 14 of SRP 3.7.3, 'Methods for Seismic Analysis of Above-Ground Tanks', states that for above ground tanks the seismic analysis criteria and methods should consider hydrodynamic forces, tank flexibility, soil-structure interaction, and other pertinent parameters. The Ultimate Heat Sink (UHS) structural analysis was performed to meet these criteria as described below: Criterion A: The "Housner method" described in TID-7024 was used to determine the hydrodynamic impulsive and convective

		masses. The impulsive masses were applied to the walls of the UHS Soil-Structure Interaction (SSI) model. Since the fundamental sloshing (convective) mode was so low (0.135 cycles per second in the N-S direction and 0.078 cycles per second in the E-W direction), the convective mass was not included in the SSI model but was considered in the basic design. Since there is no roof to the UHS, the horizontal modes only affect the UHS basin walls. Criterion B: The impulsive hydrodynamic masses were assigned to the walls of the UHS SSI model, which does not assume that the basin is rigid. Therefore, the horizontal impulsive-mode spectral acceleration Sa1 was based on consideration of the flexibility of the tank. Moreover, an uplift check was performed and it was concluded that the effect of the limited uplift under the corner of the UHS foundation mat toe can be neglected. Criterion C: The impulsive hydrodynamic masses were assigned to the concrete walls of the UHS SSI model. Therefore the hydrodynamic spectral accelerations were based on the system damping associated with concrete as well as with the soil-structure interaction. Criterion D: A fluid damping ratio of 0.5 percent was used in determining the spectral acceleration in the horizontal convective mode. Criterion E: The maximum overturning moment, Mo, was obtained by the modal and spatial combination methods discussed in subsection II of SRP Section 3.7.2 and did not induce any uplift tension. Criterion F: The seismically induced hydrodynamic pressures on the tank walls were determined by the modal and spatial combination methods outlined in SRP Section 3.7.2 including the effects of soil-structure interaction. The hydrodynamic pressure was added to the hydrostatic pressure to determine the forces on basin walls in the basic design. The induced tension in the basin walls due to water pressure is accounted for in the design. Criterion G:

		Since the UHS basin is an open-top structure, acceptance criterion 14.G does not apply. Criterion H: The piping analysis will be performed during the detailed design at which time the structure will be designed to withstand seismic forces imposed by the piping anchors. Criterion I: The tank foundation was designed to accommodate the seismic forces imposed on it, including the hydrodynamic fluid pressures imposed on the base of the tank. Criterion J: Buckling of basin walls, failure of piping in the pump house and its effect on the pump house walls, and sliding of the tank were all considered for the basic UHS design. No COLA revision is required as a result of this response. **COLA Impac**t None. **Staff Assessment** The applicant's response to RAI 03.08.04-16 refers to the temperature and flood loads in Subsections 3H.6.4.3.1.5 and 3H.6.4.3.3.4, respectively, of the enclosure to the response submitted for RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112 dated August 20, 2009). In Question 03.08.04-12 the staff pointed out that in these subsections the thermal and lateral soil pressure loads values were not specified, and only the definitions of terms were provided. This issue is already addressed in a follow-up question to Question 03.08.04-12. In response to the request to provide details about how hydrodynamic loads were calculated and applied to the finite element model, the applicant explained in its response how all elements of SRP 3.7.3 Acceptance Criterion 14 were met in the analysis. The staff reviewed the response and concluded that applicant has appropriately used the pertinent SRP acceptance criteria for applying hydrodynamic loads to the Ultimate Heat Sink (UHS) structure. However, there was no FSAR update provided to include the information provided in the response. Therefore, the

		staff requested the applicant to include a summary description in the FSAR about modeling of the hydrodynamic load. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.04-16 (RAI 3323, Rev.2) The applicant in its response to Question 03.08.04-16 provided details of how hydrodynamic loads were included in the Ultimate Heat Sink (UHS) finite element model following the guidance provided in SRP 3.7.3, but did not include any information in the FSAR. The applicant is requested to include in the FSAR a summary description about how hydrodynamic loads were included in the UHS structure model to meet the guidance provided in SRP 3.7.3, Acceptance Criterion 14. **Status** Closed.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.05-1	Design and stability evaluation of foundations for site-specific seismic category I structures.	**RAI Text** In FSAR Section 3H.6, "Site-Specific Seismic Category I Structures," the applicant did not include sufficient information about design and stability evaluation of foundations in order for the staff to determine if the foundations for site-specific seismic category I structures meet the acceptance criteria provided in SRP 3.8.5.II. Please include in the COL FSAR appropriate subsections of 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 soil bearing pressures, method of evaluation of stability of foundations including consideration of buoyant forces, coefficient of friction used for evaluation of foundation sliding 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; or justify an alternative approach. **Application Response** **Reference File<U7-C-STP-NRC-090153.pdf>** As described in COLA Part 2, Tier 2, Section 3H.6.4.5, stability evaluations are performed in accordance with the loads and load combinations of Standard Review Plan (SRP) 3.8.5.II meeting the Minimum Required Factors of Safety specified in the SRP. Detailed information regarding how the sliding and overturning evaluations are performed is provided in the response to RAI 03.07.02-10 (see letter U7-C-STP-NRC-090136 dated September 15, 2009). The seismic forces are obtained from the seismic analysis described in COLA Part 2, Tier 2, Section 3H.6.5 and in the responses to RAI 03.07.02-4 and RAI 03.07.02-5 (see letter U7-C-STP-NRC-090136 dated September 15, 2009). The safety factor against Flotation is determined considering the maximum buoyant force under flooded condition (i.e. design flood level at elevation 40 ft - 0 in). The differential settlements will be determined based on detailed settlement calculations considering time rate of settlements and construction sequence. Additional information on settlements is provided in the response to RAI 02.05.04-30 (see letter U7-C-STP-NRC-090146 dated September 21, 2009). 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 lateral pressure calculations on foundation walls are performed as described in COLA Part 2, Tier 2, Section 3H.6.6.2. Analysis and design results including the coefficient of friction used for sliding evaluation; calculated factors of safety against overturning, sliding and flotation; factors of safety for static and dynamic bearing pressures; lateral pressures on foundation walls; and design details of foundation walls and mat will be provided in a supplemental response to RAI 03.07.01-13 by December 31, 2009, in accordance with the schedule provided in Attachment 1 of letter U7-C-STP-NRC-090112 dated August 20, 2009. No additional COLA change is required for this response. **COLA Impact** None. **Staff Assessment** In response to RAI question 03.08.05-1 (STP letter U7-C-STP-NRC-090153 dated September 22, 2009), the applicant states that "detailed information regarding how the sliding and overturning evaluations are performed is provided in the response to RAI 03.07.02-10 (see letter U7-C-STP-NRC-090136 dated September 15, 2009)." The applicant also states that "the seismic forces are obtained from the seismic analysis described in COLA Part 2, Section 3H.6.5 and in the responses to RAI 03.07.02-4 and RAI 03.07.02-5 (see letter U7-C-STP-NRC-090136 dated September 15, 2009)." These RAIs are still under review, and the issue will be resolved upon completion of review of the above RAIs. In addition, the applicant states that "the lateral pressure calculations on foundation walls are performed as described in COLA Part 2, Tier 2, Subsection 3H.6.6.2. Analysis and design results including the coefficient of friction used for sliding evaluation; calculated factors of safety against overturning, sliding and floatation; factors of safety for static and dynamic bearing pressures; lateral pressures on foundation walls; and design results of foundation walls and mat will be provided in a supplemental response to RAI 03.07.01-13 by December 31, 2009." These items will be reviewed upon receipt of the applicant's supplemental response to RAI 03.07.01-13. The applicant further states that "The differential settlements will be determined based on detailed settlement calculations considering time rate of settlements and construction sequence. Additional information on settlements is provided in the response to RAI 02.05.04-30 (see letter U7-C-STP-NRC-090146 dated September 21, 2009)." The impact of differential settlements on the member design forces will be accounted for by including the applicable differential settlements in the analysis of these structures." Although the applicant's response to RAI 02.05.04-30

		(letter U7-C-STP-NRC-090146) provides general information on the settlement study, the applicant did not provide any information regarding the magnitudes of differential settlements considered in the design and how these differential settlements were included in the analysis of the site-specific seismic category I structures. **Follow-up Question to the Partial Response** Follow-up 1 to Question 03.08.05-1 (RAI 3324, Rev.2) The applicant's response to RAI 03.08.05-1 states that "the differential settlements will be determined based on detailed settlement calculations considering the time rate of settlements and construction sequence. Additional information on settlements is provided in the response to RAI 02.05.04-30 (see letter U7-C-STP-NRC-090146 dated September 21, 2009)." Although the applicant's response to RAI 02.05.04-30 (letter U7-C-STP-NRC-090146) provides general information on the settlement study, the applicant did not provide any information regarding magnitudes of the differential settlements considered for design of site-specific seismic category I structures, and how the differential settlements were included in the analysis of these structures. Therefore, the applicant is requested to clearly describe the magnitudes of differential settlements considered for design of site-specific seismic category I structures, and also explain how differential settlements were accounted for in the analysis of these structures. This information is needed so the staff can conclude that the design of site-specific seismic category I structures has appropriately considered the differential settlements. Also, the applicant stated in its response that information pertaining to analysis and design results including the coefficient of friction used for sliding evaluation, calculated factors of safety for static and dynamic bearing pressures, lateral pressure on foundation walls, and design details of foundation walls and mat will be provided in a supplemental response to RAI 03.07.01-13 by December 31, 2009. The applicant is requested to either include the above information in its response, or include the information in the December supplemental response, and update the FSAR with relevant information, as appropriate. **Status** Open.

RAI Number	Question Summary	Full RAI Text / Application Response / Staff Assessment
03.08.05-? New Question	COL License Information Item 3.24 - Foundation settlement.	**RAI Text** In FSAR Section 3.8.6.2, "Site Specific Physical Properties and Foundation Settlement," the applicant referred to FSAR Sections 3H.6.4.2 and 2.5S.4 to address COL License Information Item 3.24, which required that the physical properties of the site-specific subgrade medium be determined, and the settlement of foundations and structures, including seismic category I, be evaluated. In FSAR Section 2.5S.4.10.4, the applicant provided a settlement evaluation of the structures and stated that "from the differential settlement value, angular distortions/tilts were estimated (based on average foundation plan dimension), and for all evaluated structures were within the acceptable limit of 1/300." It is not clear if the applicant implied that the ABWR DCD standard plant structures were designed using the above acceptable limit. Therefore, the applicant is requested 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, provide justification for acceptability of angular distortions determined for these structures for the STP site. Please also explain how the site-specific differential settlements between adjacent buildings are considered acceptable in relation to their impact on tunnels and other commodities between these buildings for the standard plant structures. Please include pertinent references to the sources of any information used in the response. This information is needed so the staff can conclude that the applicant has completed all actions required by COL License Information Item 3.24. **Application Response** **Reference File< >** **COLA Impact** **Staff Assessment**

		Follow-up Question to the Partial Response Status

Request for Additional Information No. 4834 Revision 3

South Texas Project Units 3 and 4
South Texas Project Nuclear Operating Co
Docket No. 52-012 and 52-013
SRP Section: 03.08.05 - Foundations
Application Section: FSAR 3.8

QUESTIONS for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

03.08.05-***

Follow-up to Question 03.08.05-2

In its response to Question 03.08.05-2 (letter U7-C-STP-NRC-100108, dated May 13, 2010) regarding how differential settlements were considered for site-specific seismic category I structures, the applicant provided some information. However, in order for the staff to clearly understand the amount of differential settlement values accounted for in the design of site-specific seismic category I structures, and how these values reconcile with the estimated differential settlements at the site, the applicant is requested to provide the following additional information:

1. In Part (a) of its revised response to Question 03.08.05-2, the applicant referred to COLA Part 2, Tier 2, Section 2.5S.4.10 for conservatively calculated angular distortion/tilts. The applicant provided an explanation in its response to Question 03.08.05-3 about why the calculated angular distortions/tilts may be considered acceptable. In its justification of an acceptable tilt value of 1/500 for the seismic category I structures at STP, the applicant referenced several published materials that appear to be based on observations of cracking and structural damage of commercial structures. The applicant did not provide any justification for using this information for seismic category I structures. The information included in the response does not provide any estimate of the amount of additional stresses that may be imposed on these structures as a result of the tilt. Therefore, in order for the staff to conclude that the acceptable tilt of 1/500 for the seismic category I structures at STP will not adversely impact the calculated stresses in these structures in critical areas, the applicant is requested to provide a quantitative evaluation that explicitly considers the tilt for these structures.

2. In Part (b) of its revised response to Question 03.08.05-2 on Differential Settlement due to Flexibility of Structure/Basemat and Supporting Soil, the applicant stated that the effect of settlement due to the flexibility of the structure/basemat and supporting soil is accounted for through the use of finite element analysis (FEA) in conjunction with foundation soil springs. However, the foundation subgrade modulus may vary over a wide range across the foundation footprint. It is not clear from the response if the applicant considered in the analysis the horizontal variation of foundation subgrade modulus over the entire area of the foundation. Also, it is not clear from the response how the differential settlements accounted for in the design through the FEA modeling reconcile with the calculated differential settlements in Section 2.5S.10.4 of the FSAR and the

values of maximum differential settlements that the structures are designed for. Therefore, in order for the staff to complete its review of how differential settlements are accounted for in the design of site-specific seismic category I structures, the applicant is requested to provide the following additional information:

- Describe how the variation of the subgrade modulus over the foundation footprint has been considered in the analysis, and
- List in the FSAR the values of maximum differential foundation settlements for which each seismic category I structure is designed.

03.08.05-***

Follow-up to Question 03.08.05-3

In its response to Question 03.08.05-3 (letter U7-C-STP-NRC-100083, dated April 14, 2010), the applicant stated that the ABWR DCD does not contain any criteria for settlement-related angular distortions/tilts. The applicant explained that its use of an acceptable tilt value of 1/500 for Category I structures is based on information from several published literature. However, the applicant did not provide any information about the amount of additional stress that may be imposed on the standard plant structures as a result of the acceptable tilt of 1/500. The applicant further stated that structural analysis and design of the structures account for the induced stresses due to structural and foundation flexibility. However, it is not clear from the response if the expected differential settlements for the standard plant structures at the STP site would be within the values of differential settlements that were accounted for in the analysis of ABWR standard plant structures. Therefore, to address COL 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, be evaluated, the applicant is requested to:

1. Provide a quantitative evaluation of the proposed acceptance criteria for foundation tilt to demonstrate that the ABWR standard plant structures would not be adversely stressed as a result of the tilt.
2. 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.

Please also update the FSAR to clearly state how this COL information item is addressed. The staff needs this information to conclude that the ABWR standard plant structures are adequate to accommodate site-specific differential settlements.

Request for Additional Information No. 4832 Revision 3

South Texas Project Units 3 and 4
South Texas Project Nuclear Operating Co
Docket No. 52-012 and 52-013
SRP Section: 03.08.01 - Concrete Containment
Application Section: FSAR 3.8

QUESTIONS for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

03.08.01-***

Follow-up to Question 03.08.01-6

In its response to Question 03.08.01-6, the applicant addressed some of the issues regarding the watertight doors. However, additional information is needed to completely address all of the issues pertaining to the design of the watertight doors. In order for the staff to complete its review, the applicant is requested to provide the following additional information:

1. In Section 2 of the response, the applicant provided a sketch that shows the location of the watertight door between the Control building and the Radwaste Building Access Corridor. However, the applicant did not include the sketch in the FSAR mark-up provided with the response. Therefore, the applicant is requested to include the sketch in the FSAR to clearly identify locations of all seismic category I watertight doors.

2. In Section 3(a) of the response, the applicant provided loadings and loading combinations for design of watertight doors considering flooding. The staff needs the following clarifications for the loads and load combinations provided in the response:

    a. Since ANSI/AISC N690 and ACI 349 do not specifically address flood loads, please explain how the flood loads and the loading combinations, including the load factors used in loading combinations involving flood load, were determined with reference to applicable industry codes and standards. Please include in FSAR Section 3H.6.4.3.3.4, “Extreme Environmental Flood (FL),” a description of the various components of flood load, e.g., hydrostatic load, hydrodynamic load, impact load from debris transported by flood water, etc., and the corresponding design values used.

    b. The applicant defined pressure load ‘P’ as hydrostatic or differential pressure, and used it in several loading combinations. Please explain why only pressure load ‘P’ need to be considered for design of watertight doors, and not the other components of FL, e.g., hydrodynamic load and load from debris transported by flood.

3. In Section 3(b) of the response, the applicant stated that the doors will be designed in accordance with AISC N690. Since it is not clear which version of ANSI/AISC N690 was used by the applicant, please confirm that the version of the specification used is the same as that referenced in SRP 3.8.4 and update FSAR accordingly, or provide justification for using a different version.

4. In response to the staff's question regarding design and analysis procedure used for the watertight doors, the applicant stated in Section 3(c) of the response that "the design of the door will be performed in accordance with the requirements of SRP Section 3.8.4." SRP 3.8.4 provides general guidance and acceptance criteria for analysis and design procedure of concrete and steel category I structure. Merely referencing the SRP does not provide any information about the analysis and design procedure used by the applicant. Therefore, the applicant is requested to include in the FSAR a description of the analysis and design procedure including how seismic loads are determined for the watertight doors.

5. In response to the staff's question regarding testing and in-service inspection of the watertight doors, the applicant stated in Section 3(f) of the response, and the FSAR mark-up included in the response, that the watertight doors will allow slight seepage during an external flooding in accordance with criteria for Type 2 closures in U.S. Army Corps of Engineers (COE) EP 1165-2-314. The applicant also stated that this criterion will be met under hydrostatic loading of 12 inches of water above the design basis flood level. The applicant further stated that the water retaining capability of the doors will be demonstrated by qualification tests that shall not allow leakage more than 1/10 gallon per linear foot of gasket when subjected to the specified head pressure plus a 25% margin for one hour. The applicant did not provide in the response any information regarding in-service inspections of the watertight doors. In order for the staff to assess adequacy of the watertight doors and their availability when needed, please provide the following additional information:

    a. The allowable leakage of 1/10 gallon per linear foot of gasket per hour may potentially allow ingress of significant amount of water over time. Please provide justification why this leakage is considered to meet criterion for Type 2 closure, which is defined to form essentially dry barriers or seals, and the basis for the underlying assumption that such leakage will not compromise functionality of any safety related commodity or any other design basis.
    b. Since hydrostatic pressure on the door may help in providing a seal for the door, please explain why testing these doors against the maximum water pressure only is adequate, and will envelope performance of the seals during lower hydrostatic pressure.
    c. Since the applicant did not include in its response any information about the in-service surveillance programs for the watertight doors, and corresponding FSAR update, please explain how availability of the normally open watertight doors during a flooding event is ensured considering that these doors will need to be closed upon indication of an imminent flood.

6. In Section 6 of the response, the applicant states that the access doors between the Reactor Building (RB) and Control building (CB) are not required to be watertight since both buildings are separately protected from design basis flood, and the gap between the two buildings will be sealed using the detail shown in Figure 03.08-04-15A, which is attached to the response to RAI 03.08.04-15 (see STPNOC letter U7-C-STP-NRC-090160 dated October 5, 2009). The above referenced Figure provides only a conceptual detail of a joint seal between the buried Reactor Service Water (RSW) tunnels, and the RSW Pump House and the Control Buildings. In its response to a subsequent follow-up question 03.08.04-25 for the above referenced joint seal, the applicant provided additional design criteria for the seals to accommodate differential movements across the seal, and explained that because of the low rate with which groundwater can flow through the seal if it were to fail in any particular location, the in-leakage of groundwater is a housekeeping issue and not a safety concern. Since the seals for the gaps between the RB and the CB are credited to prevent ingress of flood water into these buildings and provide protection to safety related commodities against flooding, reference to the joint seals used for the RSW tunnels does not adequately address the issue of ingress of flood water and potential damage to safety related components. Therefore, the applicant is requested to include in the FSAR a description of the seal between the RB and the CB including information about seismic classification, performance demand, qualification, and in-service inspection of the seal to demonstrate that the seals will be capable of preventing flood water from entering these buildings under all postulated design basis loading conditions.

The staff needs the above information to conclude that the watertight doors are designed for appropriate loads and load combinations, pertinent design information per guidance provided in SRP 3.8.4 are included in the FSAR, and there is reasonable assurance that the normally open watertight doors will be available during a flooding event.

03.08.01-***

Follow-up to Question 03.08.01-7

In response to Question 03.08.01-7, Section (1), the applicant provided details of how the out-of-plane shear and moment demands for flood and seismic loads were determined. The staff notes that the applicant in its response did not consider loading due to floating debris for computing shear and moment demands for flood. Also, the applicant implicitly used the loading combination for flood load as shown in FSAR Section 3H.6.4.3.4.3. This loading combination is not included in ACI 349, “Code Requirements for Nuclear Safety Related Concrete Structures,” as referenced in SRP 3.8.4. Further, computations of shear and moment demands due to flood loading for the RB and CB walls appear to be incorrect for the assumed boundary conditions for the wall sections. Therefore, in order for the staff to be able to conclude that the ABWR standard plant structures are capable of withstanding the site-specific flood load, the applicant is requested to provide the following additional information:

1. Please include the effect of debris in flood water in the evaluation of representative wall elements of the Reactor Building (RB) and the Control Building (CB) for design basis flood. The staff notes that in its response to Question 03.08.04-22, the applicant had considered loading due to debris in flood water by considering

the unit weight of flood water to be 80 pounds per cubic foot (pcf). Please provide justification for assumed debris loading with reference to industry standards and codes, as applicable.

2. Please provide the basis for the loading combination used for flood loading with reference to applicable industry codes and standards.
3. Please review the computations for shear and moment demands due to flood for RB and CB wall sections included in the response, and correct them, as needed.

Request for Additional Information No. 4833 Revision 3

South Texas Project Units 3 and 4
South Texas Project Nuclear Operating Co
Docket No. 52-012 and 52-013
SRP Section: 03.08.04 - Other Seismic Category I Structures
Application Section: FSAR 3.8

QUESTIONS for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

03.08.04-***

Follow-up to Question 03.08.04-19

In its response to Question 03.08.04-19 (letter no. U7-C-STP-NRC-100093 dated April 29, 2010), the applicant provided some information about the foundation waterproofing material. However, some of the information provided needs further clarification. In order for the staff to conclude that the foundation waterproofing used is adequate for providing waterproofing, and will not compromise sliding stability of structures, the applicant is requested to provide the following additional information:

1. The applicant stated in its response that a two-coat elastomeric spray-on membrane will be used for waterproofing, and the physical properties of the membrane have been specifically designed to cope with the rigorous requirements of below grade conditions. However, the applicant did not provide any information regarding the meaning of “rigorous requirements of below grade conditions,” and how the physical properties of the membrane meet these requirements. The applicant is requested to describe the rigor of the requirements of the below grade conditions, and how the physical properties of the membrane meet these requirements. Please also include in the in the FSAR description and thickness of the material used for the waterproof membrane.

2. The applicant stated in the response that the waterproofing membrane will be 120 mils thick, and a qualification program, which will include testing, will be developed to demonstrate that the selected material will meet the waterproofing requirements. However, the applicant did not provide any information about what the waterproofing requirements are, and the criteria to be used for the testing. Therefore, the applicant is requested to describe these waterproofing requirements to be tested including how these requirements are established, and how they will be tested to demonstrate that the selected membrane is adequate to meet the waterproofing requirements considering long term behavior of the membrane. The applicant is also requested to update the FSAR as appropriate.

3. In response to the staff’s question regarding the coefficient of friction for the waterproofing membrane, the applicant has proposed an ITAAC that states that “Type testing will be performed to determine the minimum coefficient of friction of the type of material used in the mudmat-waterproofing-mudmat interface beneath the basemats of the Category I structures.” It is not clear from the description if the thickness of the specimen tested will be the same as that used for the

membrane. The applicant is requested to clarify this and revise the ITAAC. Also, the acceptance criteria for the ITAAC states 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 applicant stated in the response that the minimum coefficient of friction needed for maintaining the minimum factor of safety against sliding for the Reactor Building (RB) and the Control Building (CB) is 0.47. In its response, the applicant also presented in Table RAI 03.08.04-19a the minimum coefficient of friction provided at the structural concrete fill and waterproofing membrane interface as 0.6. The applicant is requested to clarify which value of coefficient of friction will be used for the acceptance criteria of the ITAAC, and include in the FSAR the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface. Please also revise the ITAAC acceptance criteria accordingly.

4. The applicant stated in its response (Table RAI 03.08.04-19a) 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. Elsewhere in the response, the applicant stated that the soil capacity exceeds the value of 0.47 needed for maintaining minimum factor of safety against sliding of RB and CB. The applicant is requested to clarify the minimum coefficient of friction available at the bottom of gravel and soil interface based on site-specific soil properties and explain how it is determined.

03.08.04-***

Follow-up to Question 03.08.04-22

In its response to Question 03.08.04-22 (letter no. U7-C-STP-NRC-100036 dated February 10, 2010), the applicant provided marked-up FSAR pages with information about loadings to be used for design of site-specific seismic category I structures. To assist staff in understanding the information provided, the applicant is requested to provide the following additional information/clarifications:

1. FSAR mark-up for Section 3H.6.4.3.1.5 includes a statement "This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors" for thermal conditions described in sub item (3) and sub item (6). Please clarify why the statement is applicable for only the above two thermal conditions, and not for all 6 thermal conditions.

2. FSAR mark-up for Section 3H.6.4.3.4.3 included in the response provides loading combinations to be used for site-specific seismic category I structures. Please explain the following loading combinations:

   - $D + F + L + H + T_a + E'$ – Provide justification for using only lateral soil pressure H, and not H', which includes seismic effects.
   - $D + F + L_0 + H' + T_0 + R_0 + E'$ – Provide justification for using L_0, which is only 25% of design live load, and not L, the full design live load.

03.08.04-***

Follow-up to Question 03.08.04-23

In response to staff question requesting additional information (letter U7-C-STP-NRC-100036, dated February 10, 2010) about how various steel and concrete elements of site-specific structures are designed, and the design results, the applicant provided some analysis and design information. The applicant also referred to the Supplement 2 response to Question 03.07.01-13 (letter U7-C-STP-NRC-090230, dated 12/30/09) for pertinent design summary information. In order for the staff to conclude that the design of site-specific structures meet the requirements of GDC 2 by meeting the guidance provided in SRP 3.8.4 and 3.8.5, or otherwise, the applicant is requested to provide the following additional information:

1. The applicant states in the response that a three dimensional finite element analysis (FEA) is used for structural analysis and design of the UHS/RSW Pump House. FSAR Section 3H.6.6.1 states that analysis for the seismic loads was performed using equivalent static loads and the induced forces due to X, Y, and Z seismic excitations were combined using the SRSS method of combination. However, the applicant did not describe how the equivalent static loads due to seismic excitation were determined and applied to the static FEA model from the results of soil structure interaction (SSI) analysis used for determination of seismic response. Therefore, the applicant is requested to provide details of how seismic response analysis results from dynamic SSI analysis were transferred to the static FEA model, including how the effects of accidental torsion were included in the analysis and design of UHS/RSW Pump house. Please also update FSAR with the information, as appropriate.

2. The applicant stated in its response that the modulus of subgrade reaction for static loading was calculated as the average of the local values at nine locations under the foundation. The applicant is requested to provide these nine values, and explain why it is considered appropriate to use the average value. Please also explain how the foundation subgrade modulus was used for calculating nodal springs for the FEA model, and how the effect due to coupling of soil springs was considered in the analysis.

3. For seismic loading, the applicant has outlined a hand-calculated procedure that utilizes published formulas and charts to estimate the foundation spring constants. According to this procedure, the equivalent modulus and Poisson's ratio of a layered soil system are first estimated using the cumulative strain energy method. The resulting values are then used in the equations for computation of the spring constants for a rigid foundation of an arbitrary shape embedded in a uniform half-space. The shear moduli used for individual layers are strain-compatible values, and include the mean, upper bound, and lower bound soil cases. The approximate procedure outlined above for developing the foundation spring constants does not take into account the pressure distribution under the base slab. Furthermore, this procedure does not account for the frequency dependence of these springs. As such, the applicant is requested to provide a justification for not considering the effects of pressure distribution and

system frequency in developing the foundation dynamic springs including describing the impact on the calculated results.

4. The applicant's response does not provide details as to how the soil springs calculated under static and seismic loadings are inputted to the 3-D static FEA model to calculate the design stresses. Therefore, the applicant is requested to describe in detail how the static and seismic soil springs are inputted into the FEA model, and how the results are obtained for stress evaluations. Specifically, the applicant is requested to explain if the two sets of springs were used in a single model, and how the two sets were combined to a single set of springs. Otherwise, if the two sets of springs were applied to separate FEA models, describe how the load combinations were performed. The applicant is also requested to provide sufficient detail to assist staff in understanding how static and seismic soil springs are used in the FEA model and results combined for stress evaluations.

5. In the FSAR mark-up of Sections 3H.6.6.3.1 and 3H.6.6.3.2 provided with the response, the applicant identifies the method used by the applicant for combining forces and moments. In this method, for each reinforcing zone, the maximum force or moment is coupled with the corresponding moment or force for design for the same load combination. It is not clear if this method of combining forces and moments for design will envelop the worst combination of forces and moments for all elements in a reinforcing zone. Therefore, the applicant is requested to describe the method of combining forces and moments used by the applicant with a typical example of a reinforcing zone, and demonstrate that this method of combination will yield the worst combination of forces and moments that should be considered for design.

6. The staff notes that in the FSAR mark-up of Section 3H.6.6.3.1 provided with the response, the reported values of soil springs for the RSW Pump House are significantly larger than those for the UHS basin. The applicant is requested to confirm these values, and explain the reason for the large difference.

7. The response did not include any information about the maximum static and dynamic bearing pressures under the foundations of UHS/RSW Pump House. The applicant is requested to provide the maximum static and dynamic bearing pressure under the foundations of UHS/RSW Pump House, compare these values with the maximum allowable static and dynamic bearing pressures, and include this information in the FSAR.

8. In its response to Question 03.07.01-19 (letter U7-C-STP-NRC-100129, dated June 7, 2010), the applicant provided analysis and design information for the seismic category I Diesel Generator Fuel Oil Storage Vault (DGFOSV) a which was not previously included in the FSAR. The information included in the response does not describe how structural analysis and design of the structure was performed. Also, reference is made to FSAR Section 3H.6.4 for design loads. FSAR Section 3H.6.4 has been updated several times in various responses, and it is not clear where this information can be found. Therefore, the applicant is requested to provide complete structural analysis and design information for the DGFOSV to ensure it meets acceptance criteria 1 through 7 of SRP 3.8.4 and 3.8.5. The staff needs this information to conclude that the

DGFOSV is designed to withstand seismic loads and meet GDC 2. Include in the response an updated version of Appendix 3H where structural analysis and design information for all seismic category I structures can be found.

9. While reviewing this response, and other responses referenced in this response, the staff noted that the applicant has used different values of coefficient of friction for sliding stability evaluation; e.g., the value 0.3 was used for the RSW Pump House, 0.4 was used for UHS basin, 0.58 was used DGFOSV, and for the Reactor Building (RB) and the Control Building (CB), it was stated to be more than 0.47. It is not clear if these values are the required coefficient of friction, or the minimum coefficient of friction available. The applicant is requested to clearly specify the minimum coefficient of friction at various locations of the site, if they are different, and explain how these values were determined. Please also clarify this information in the FSAR.

10. The staff noted references to Diesel Generator Fuel Oil Tunnel (DGFOT) in several RAI responses. Please confirm that DGFOT is not a seismic category I structure, and if it is seismic category I, include the analysis and design information to show how the design of the DGFOT meets the acceptance criteria 1 through 7 in the SRP 3.8.4 and 3.8.5 in the FSAR.

03.08.04-***

Follow-up to Question 03.08.04-25

The staff reviewed the applicant's response to Question 03.08.04-25 (letter U7-C-STP-NRC-10018, dated May 13, 2010). In order for the staff to conclude that the interface between seismic category I buildings and tunnels will not result in any unacceptable interaction, the applicant is requested to provide the following additional information:

1. The applicant stated in its response that the separation gap between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump House and the Control Building (CB), as well as between the Diesel Generator Fuel Oil Storage Vaults (DGFOSV) and the Diesel Generator Fuel Oil Tunnels (DGFOT), will be at least 50% larger than the absolute sum of the calculated displacements due to seismic movements and long term settlement. The material used as flexible filler will be able to be compressed to approximately 1/3 of its thickness without subjecting the building to more than a negligible force. However, the applicant provided vendor test result where 7 psi compressive stress was observed when 5 inch joint was compressed to 50% movement. This does not provide any estimate of how much compressive stress may be developed when the material is compressed to 1/3 thickness of the material. Therefore, the applicant is requested to justify that no significant stress will be imparted to the building when the joint is compressed to 1/3 thickness.

2. The DGFOT is connected to the DGFOSV at one end. It is not clear from the response where the DGFOT is connected at the other end, and what are the anticipated movements at that connection. Please include this information in Table 3H.6-15.

3. Please provide an ITAAC with key parameters for as-built verification of the connections, or provide justification for not doing so.

03.08.04-***

Follow-up to Question 03.08.04-27

The applicant stated in its response (letter U7-C-STP-NRC-100036, dated February 10, 2010) to Question 03.08.04-27 regarding COL License Information Item 3.25 that the details of the Structural Integrity Test (SIT) and the instrumentation required for the test will be provided in the ASME Construction Specification. The applicant referred to RG 1.206, Section CIII.4.3, situation 4 for resolving the COL information item six months before performance of the test. According to RG 1.206, Section CIII.4.3, the applicant should justify why the item is not resolved before the issuance of license. However, the applicant did not provide any justification. Therefore, the applicant is requested to provide a detailed justification for why any part or all of the information pertaining to the COL information item cannot be provided at this time and clearly addressing all parts of COL license information item. Also, the applicant is requested to identify in Chapter 1 of the FSAR if the COL information item cannot be resolved completely before the COL is issued. The staff needs this information to conclude that deferral of the COL information item meets the guidance provided in RG 1.206.

03.08.04-***

In FSAR Section 3.8, page 3.8-1, the applicant references the departure STD DEP 1.8-1, “Tier 2* Codes, Standards, and Regulatory Guide Edition Changes.” One of the changes included in this departure updates Tier 2 to refer to the 1997 edition of ACI 349 in place of the 1980 edition of the same building code for concrete structures. In the ABWR design certification (NUREG-1503, page 3-53), the staff had evaluated only the use of 1980 edition of ACI 349. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two editions of the code as they apply to the ABWR standard design, and provide justifications for any differences in order for the staff to evaluate the acceptability of the 1997 edition of ACI 349.

FSAR Section 3H.6.4.1 references ANSI/AISC N690 specification for design, fabrication, and erection of site-specific seismic category I steel structures. The applicant did not specify in this section which version of the specification is used. It appears that the applicant uses the 1984 edition of the specification referenced in ABWR DCD Table 1.8-21, which the applicant incorporated by reference. However, according to SRP acceptance criteria 3.8.4.II.5, ANSI/AISC N690-1994 including Supplement 2 (2004) has been accepted by the staff for design, fabrication, and erection of safety-related steel structures. According to the guidance provided in RG 1.206, Section C.I.1.9.2, the applicant should use the current SRP for structures outside the scope of the ABWR DCD, or provide justification for not doing so. Therefore, the applicant is requested to provide a detailed comparison of the differences between the 1984 (or whatever edition is used by the applicant) and the 1994 editions of the specification as they apply to the site-specific seismic category I structures at STP site. Also, provide the justification(s) for any differences in order for the staff to evaluate the acceptability of the 1984 edition of the specification.

Furthermore, the staff observed that Table 1.8-21 in FSAR Tier 2, Section 1.8, references ASME Code, Section III, Division 2, Edition 2001 with 2003 addenda, and identifies certain limitations. The ABWR DCD specifies the use of ASME code version 1989. In the ABWR FSER, p. 3-49, the NRC has accepted the 1989 Edition of the ASME Code, Section III, Division 2. Therefore, the applicant is requested 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 provide justification(s) for any differences in order for the staff to evaluate the acceptability of the ASME Code, Section III, Division 2 Edition 2001 with 2003 addenda. The applicant is also requested to explain how use of the Edition of the ASME Code proposed by the applicant meets the provisions of NCA-1140, "Use of Code Editions, Addenda, and Cases."

The staff needs the above information to conclude that the applicant used acceptable codes and standards for all seismic category I structures, and any deviations are appropriately addressed.