Nuclear Operating Company
South Texas Project Electric Generating Station P.O. Box 289 Wadsworth, Texas 77483

April 14, 2010
U7-C-STP-NRC-100083

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
One White Flint North
11555 Rockville Pike
Rockville, MD 20852-2738

South Texas Project
Units 3 and 4
Docket Nos. 52-012 and 52-013
<u>Supplemental Response to Request for Additional Information</u>

References:
1. Letter, Mark McBurnett to Document Control Desk, "Response to Request for Additional Information," dated January 14, 2010. U7-C-STP-NRC-100018 (ML100191524)
2. Letter, Scott Head to Document Control Desk, "Response to Request for Additional Information," dated February 10, 2010. U7-C-STP-NRC-100036 (ML100550613)

Attachments 2 through 6 are revised or supplemental responses to NRC staff questions included in Request for Additional Information (RAI) letter numbers 297, 299 and 302 related to Combined License Application (COLA) Part 2, Tier 2, Sections 3.7 and 3.8. References 1 and 2 provided the original response to the following RAI questions:

RAI 03.07.01-24	RAI 03.08.05-2
RAI 03.08.01-8	RAI 03.08.05-3
RAI 03.08.04-25	

Additionally, supplemental information dates, as originally provided in Attachment 15 of Reference 2 for Sections 3.7 and 3.8, have been updated and are provided in Attachment 1 to this letter.

D091
NRO

STI 32655171

Where there are COLA markups, they will be made at the first routine COLA update following NRC acceptance of the RAI response.

There are no commitments in this letter.

If you have any questions regarding this response, please contact me at (361) 972-7136, or Bill Mookhoek at (361) 972-7274.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on 4/14/10

Scott Head
Manager, Regulatory Affairs
South Texas Project Units 3 & 4

jep

Attachments:

1. Supplemental Information Dates
2. RAI 03.07.01-24, Supplement 1
3. RAI 03.08.01-8, Supplement 1
4. RAI 03.08.04-25, Revision 1
5. RAI 03.08.05-2, Revision 1
6. RAI 03.08.05-3, Revision 1

cc: w/o attachment except*
(paper copy)

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SUPPLEMENTAL INFORMATION DATES (UPDATED)

RAI Number	INFORMATION DESCRIPTION	STPNOC LETTER NUMBER	SUPPLEMENTAL / REVISION DATE
03.07.01-18	Increase in Soil Pressure due to Structure to Structure interaction	U7-C-STP-NRC-100035	April 30, 2010
03.07.01-19	Details for Diesel Generator Fuel Oil Storage Vaults	U7-C-STP-NRC-100035	April 30, 2010
03.07.01-24	Effects of Crane Wall on the Reactor and Control Buildings	U7-C-STP-NRC-100036 U7-C-STP-NRC-100083	Submitted
03.07.02-13	Stability Evaluations for Seismic Category II Structures	U7-C-STP-NRC-100036	April 30, 2010
03.07.02-19	Seismic Input for II/I Evaluation for Radwaste Building	U7-C-STP-NRC-100035	April 30, 2010
03.07.03-3	Revise previous RAI response for Control Building Annex Input Motion based on DCD model – As previously discussed in the January 19-20 meeting	U7-C-STP-NRC-090225 U7-C-STP-NRC-100036	April 30, 2010
03.08.01-8	Effect of Increase in Pool Swell Height and Pressure	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036 U7-C-STP-NRC-100083	Submitted
03.08.04-18	Radwaste Building Analysis Results and Design Details	U7-C-STP-NRC-100036	May 31, 2010
03.08.04-19	Results of Sliding Evaluation	U7-C-STP-NRC-100036	April 30, 2010
03.08.04-25	Details of Interface Connections Between the RSW Piping Tunnel and Buildings	U7-C-STP-NRC-100036 U7-C-STP-NRC-100083	Submitted
03.08.05-2	Results of Time Rate of Settlement and Evaluation of Gaps for Site-Specific Structures	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036 U7-C-STP-NRC-100083	Submitted
03.08.05-3	Revise previous RAI response for Acceptance Criteria for Building Tilt due to Settlement – As previously discussed in the January 19-20 meeting	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036 U7-C-STP-NRC-100083	Submitted

RAI 03.07.01-24, Supplement 1

QUESTION:

(Follow-up Question to RAI 03.07.01-14)

With regard to Item c of the response to RAI 03.07.01-13, the applicant is requested to address the following:

1. In the response to RAI 03.07.01-14, Item 1, the applicant cited DCD Appendix 3A in concluding that "... *the potential effect of structure-to-structure interaction was relatively small.*" However, DCD Section 3A.9.7, "Effect of Adjacent Buildings" also concluded that seismic soil pressure in between the RB and CB increased due to structure-to-structure interaction (SSSI) effect. As such the applicant is requested to discuss how the potential effects of increase in the seismic soil pressure in between the Category 1 structures and the retaining wall due to the SSSI effect has been addressed and bounded by the certified design.

2. In the response to RAI 03.07.01-14, Item 2, the applicant stated in the second bullet that "*In comparison to the Reactor, Control and Turbine Buildings, the retaining wall is a light structure and a lighter structure will have less influence on the seismic behavior of the heavy adjacent structures.*" While the inertia of the RC retaining wall is not expected to affect the seismic response of the adjacent seismic Category I structures, the stiff retaining wall can act as a barrier to reflect the seismic waves due to kinematic interaction with surrounding soil and could affect the seismic input to the adjacent structures. As such, the applicant is requested to provide a quantitative assessment of the effect of RC retaining wall on the SSI analysis of adjacent Reactor and Control Buildings.

SUPPLEMENTAL RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100036, dated February 10, 2010. In a meeting with the NRC on January 19 and 20, 2010, STPNOC agreed to provide a quantitative assessment of the effect of reinforced concrete (RC) retaining wall (crane wall) on the soil structure interaction (SSI) analysis of adjacent Reactor and Control Buildings. This information is provided in this response.

In order to address the above two questions, a soil-structure interaction (SSI) analysis of the Reactor Building (RB) and Control Building (CB), with and without the crane wall, was performed for the site-specific conditions, including site-specific safe shutdown earthquake (SSE) and soil properties. These analyses were performed using two-dimensional (2D) models of the RB and CB. The SSI analyses were performed using the SASSI2000 program. Summaries of the SSI analyses results for the mean soil case are presented below. Similar results are obtained for lower and upper bound soil cases.

Summary of Results for RB:

- Table 03.07.01-24a compares the maximum forces and moments, at key locations of the RB, with and without the crane wall. As can be seen, the crane wall has a negligible effect on the resulting maximum forces and moments.
- Figures 03.07.01-24a through 03.07.01-24d provides comparisons of response spectra at several locations with and without the crane wall. As can be seen, the crane wall has a negligible effect on the resulting spectra.
- Figure 03.07.01-24e provides the comparison of the resulting seismic soil pressures from the SSI analyses with and without the crane wall. As expected, these seismic lateral soil pressures are significantly bounded by the design seismic soil pressure per DCD Tier 2, Figure 3H.1-11 and pressure obtained from the alternate modified Ostadan method described in COLA Part 2, Tier 2, Section 2.5S.4.10.5.2.

Summary of Results for CB:

- Table 03.07.01-24b compares the maximum force and moment at grade with and without the crane wall. As can be seen, the crane wall has a negligible effect on the resulting maximum force and moment.
- Figures 03.07.01-24f and 03.07.01-24g provide comparisons of response spectra at top of basemat and top of CB with and without the crane wall. As can be seen, the crane wall has a negligible effect on the resulting spectra.
- Figure 03.07.01-24h provides the comparison of the resulting seismic soil pressures from the SSI analyses with and without the crane wall. As expected, these seismic lateral soil pressures are significantly bounded by the design seismic soil pressure per DCD Tier 2, Figure 3H.2-14 and pressure obtained from the alternate modified Ostadan method described in COLA Part 2, Tier 2, Section 2.5S.4.10.5.2.

No COLA change is required for this response.

Table 03.07.01-24a
Reactor Building Force Comparison

Effect Of Crane Wall on Maximum Forces, Mean Soil				
Beam Element	Location	Response Type	Model in SSI Analysis	
			2-D Reactor Building (alone)	2-D Reactor Building + Crane Wall
28	Shroud Support	Shear	87	90
		Moment	1,845	1,851
69	RPV Skirt	Shear	374	383
		Moment	8,948	8,717
78	RSW Base	Shear	361	348
		Moment	5,630	5,405
86	Pedestal Base	Shear	1,949	1,977
		Moment	119,475	120,414
89	RCCV at Grade	Shear	5,995	6,107
		Moment	326,161	335,547
99	R/B at Grade	Shear	13,041	13,277
		Moment	884,819	911,607

Units: Shear in kip; Moment in kip-ft

Table 03.07.01-24b
Control Building Force Comparison

Effect of Crane Wall on Maximum Forces, Mean Soil				
			Model in SSI Analysis	
Beam Element	Location	Response Type	2-D Control Building (alone)	2-D Control Building + Crane Wall
6	C/B at Grade	Shear (kip)	3,068	3,124
		Moment (kip-ft)	111,181	110,472

Figure 03.07.01-24a: Comparison of Response Spectra, Reactor Building, Bottom of Basemat

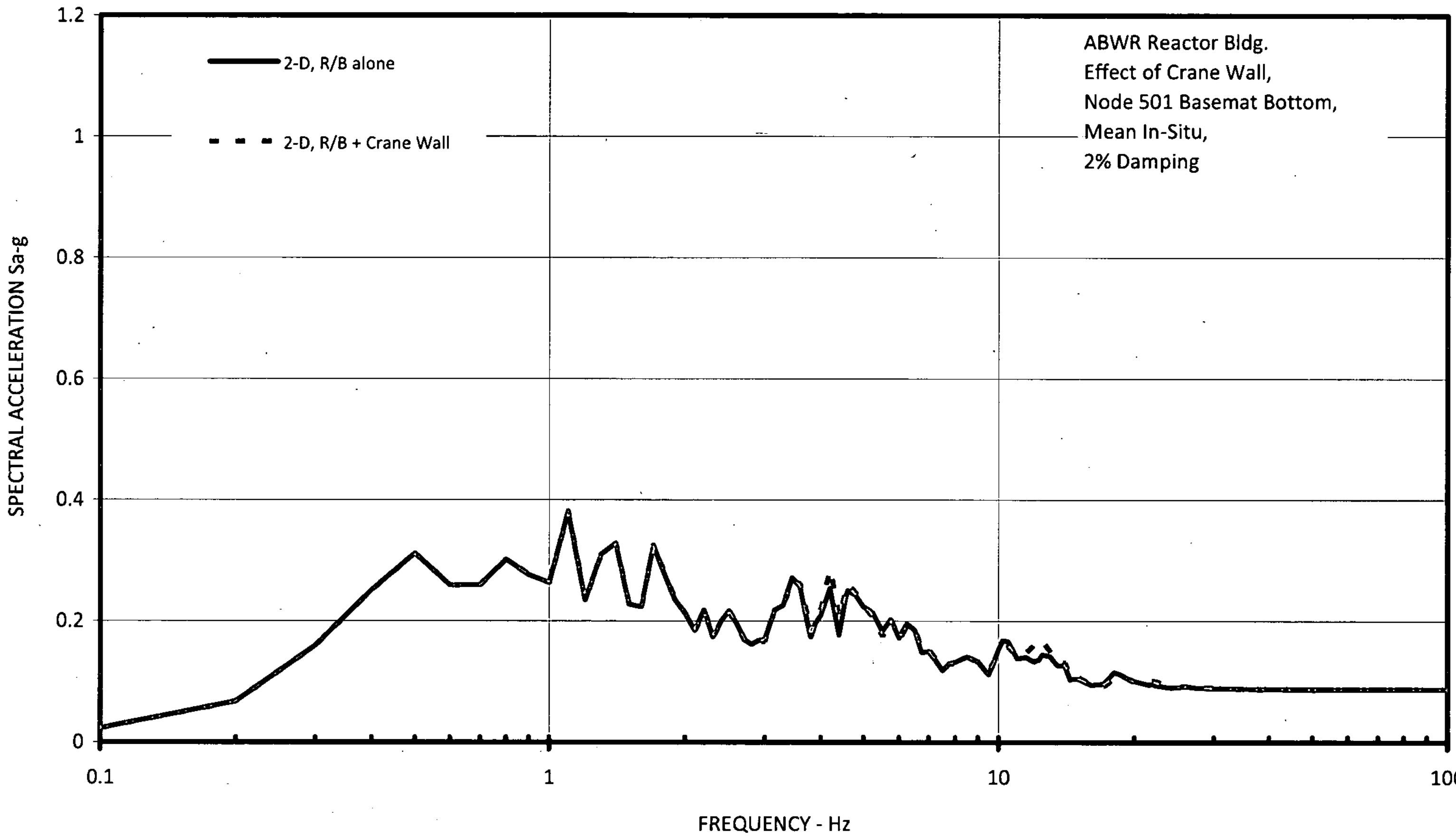


Figure 03.07.01-24b: Comparison of response Spectra, Reactor Building, RPV/MS Nozzle

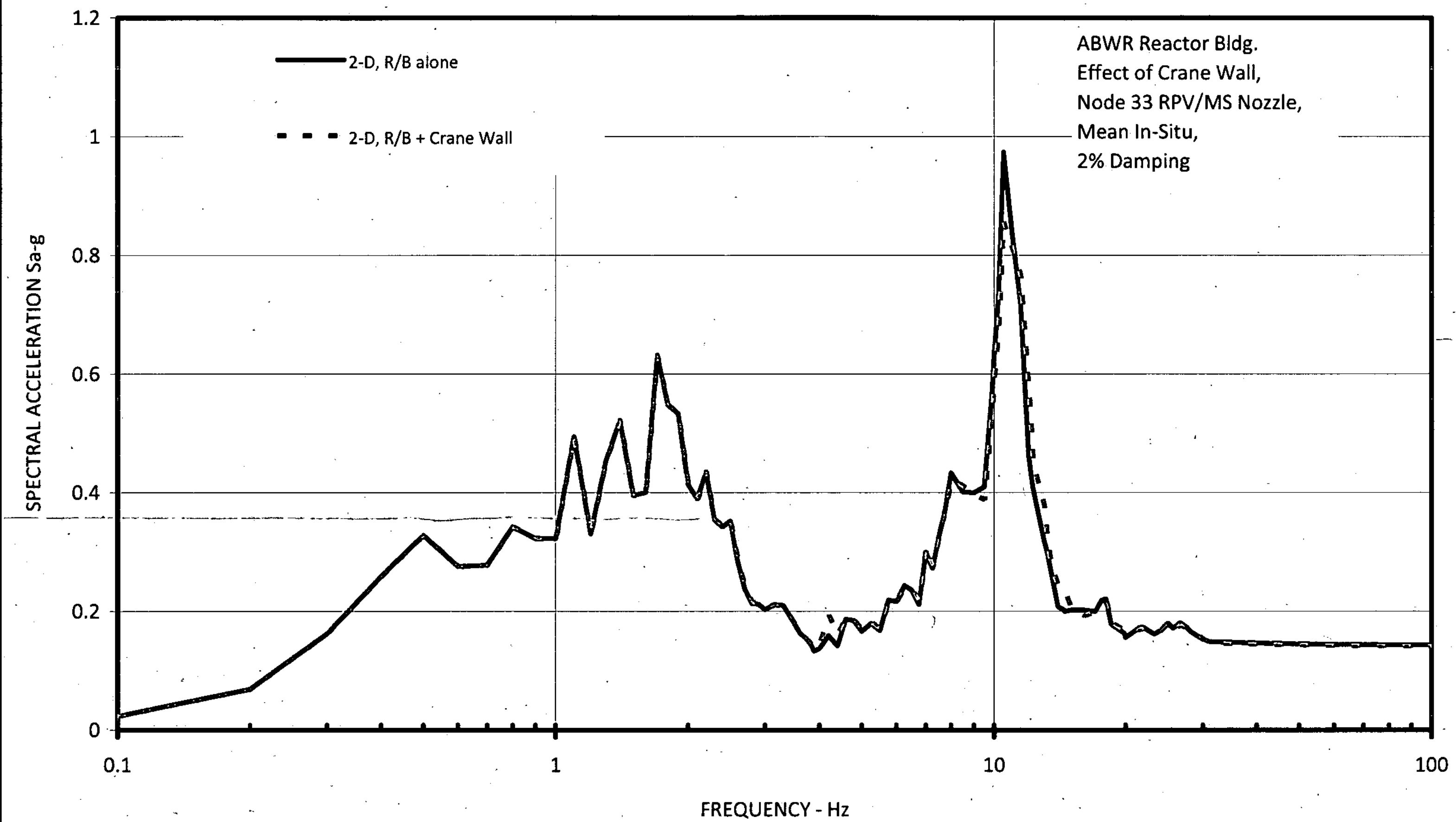


Figure 03.07.01-24c: Comparison of Response Spectra, Reactor Building, Top of RCCV

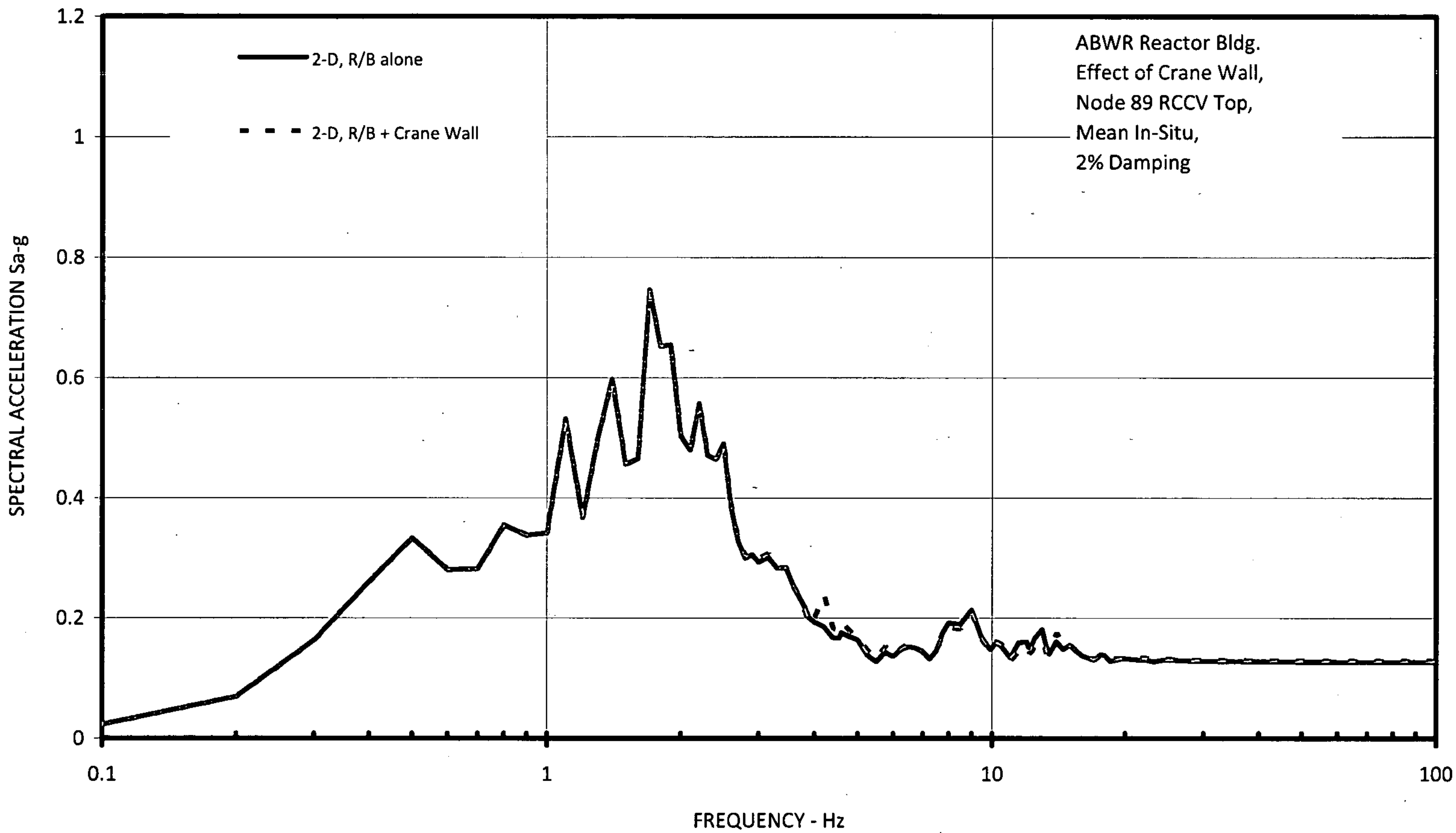


Figure 03.07.01-24d: Comparison of Response Spectra, Top of Reactor Building

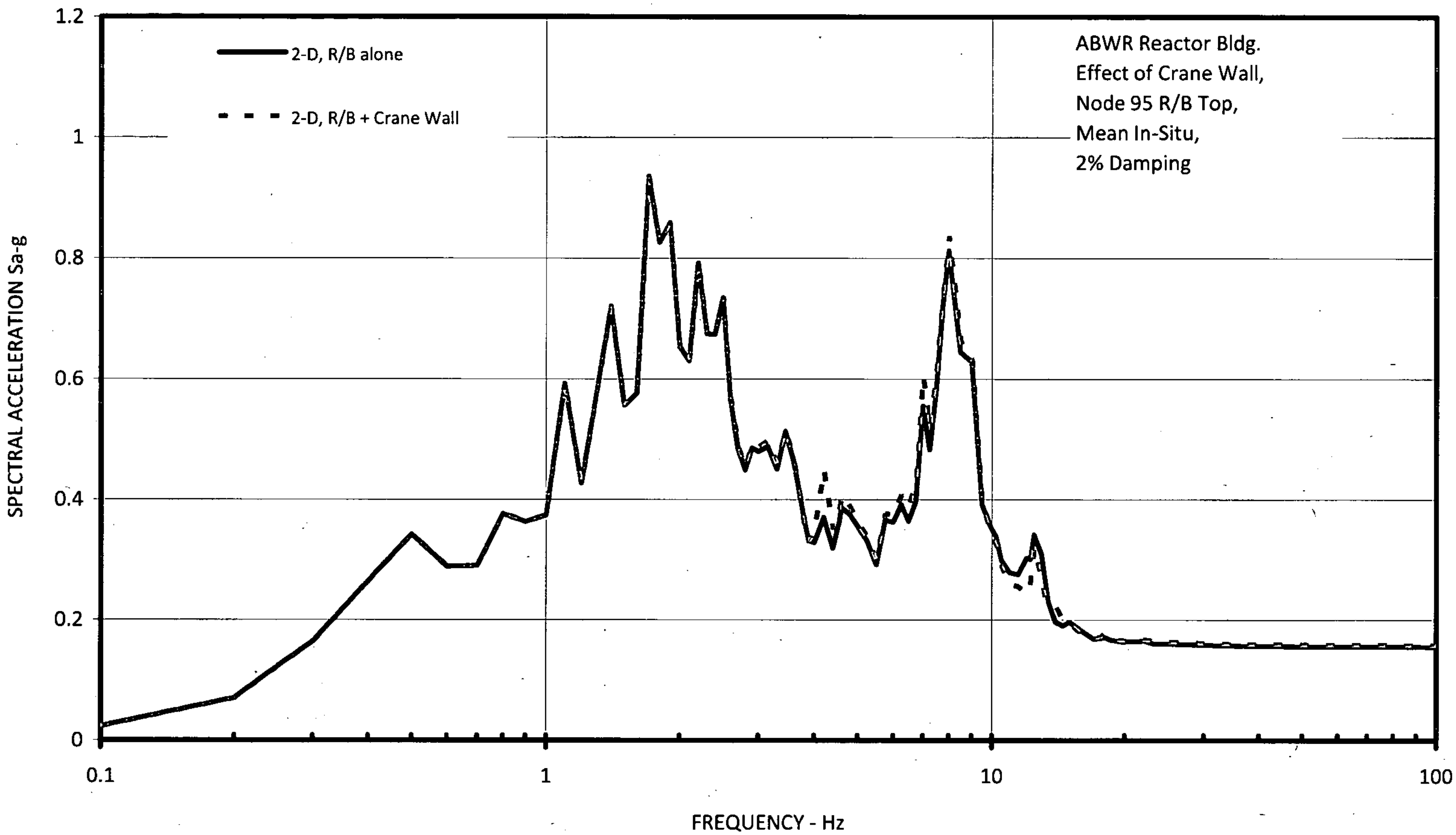


Figure 03.07.01-24e
Reactor Building Site Specific
At-Rest Seismic Lateral Earth Pressure (psf)
Multiple Methods Displayed

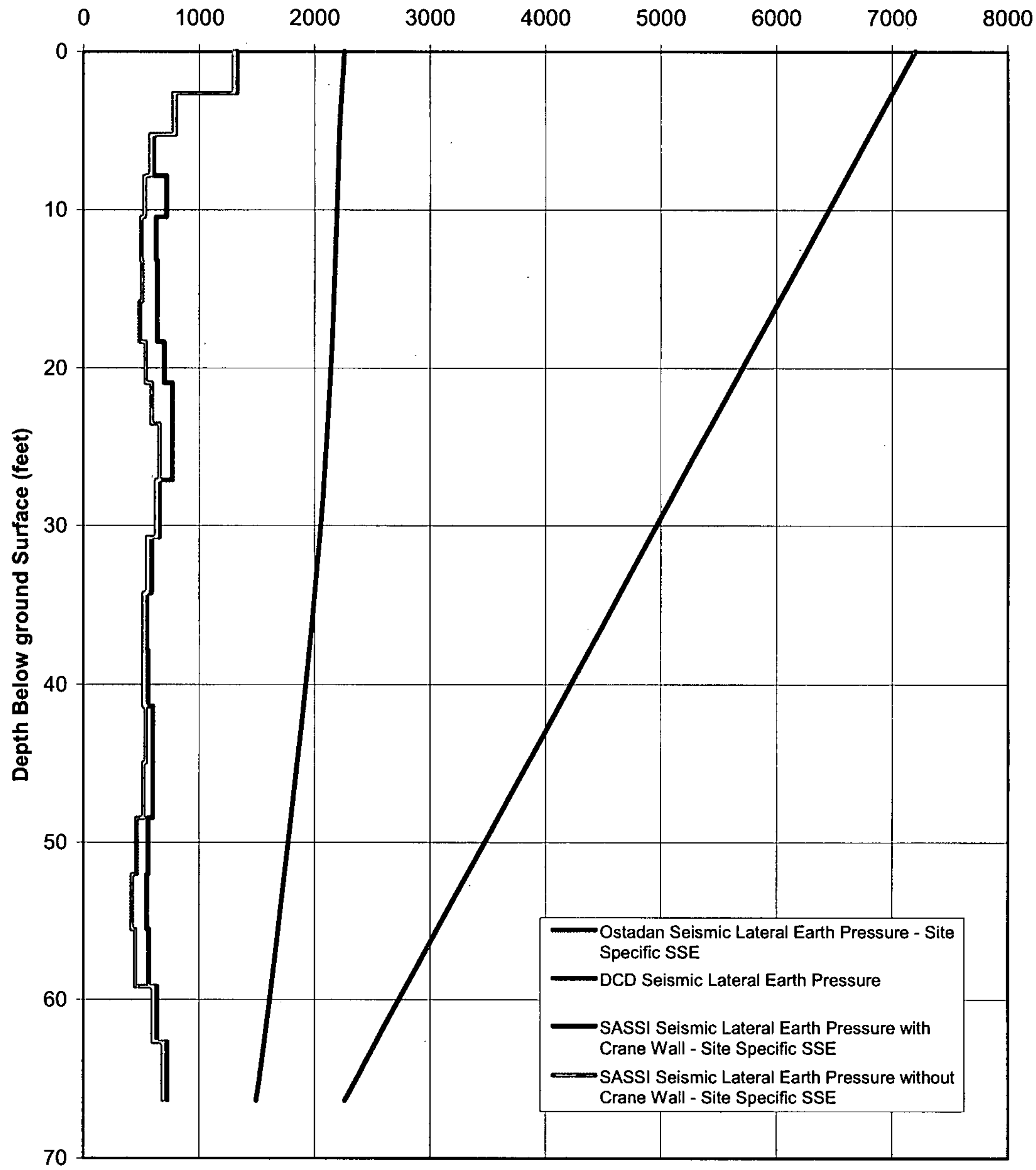


Figure 03.07.01-24f: Comparison of Response Spectra, Control Building, Top of Basemat

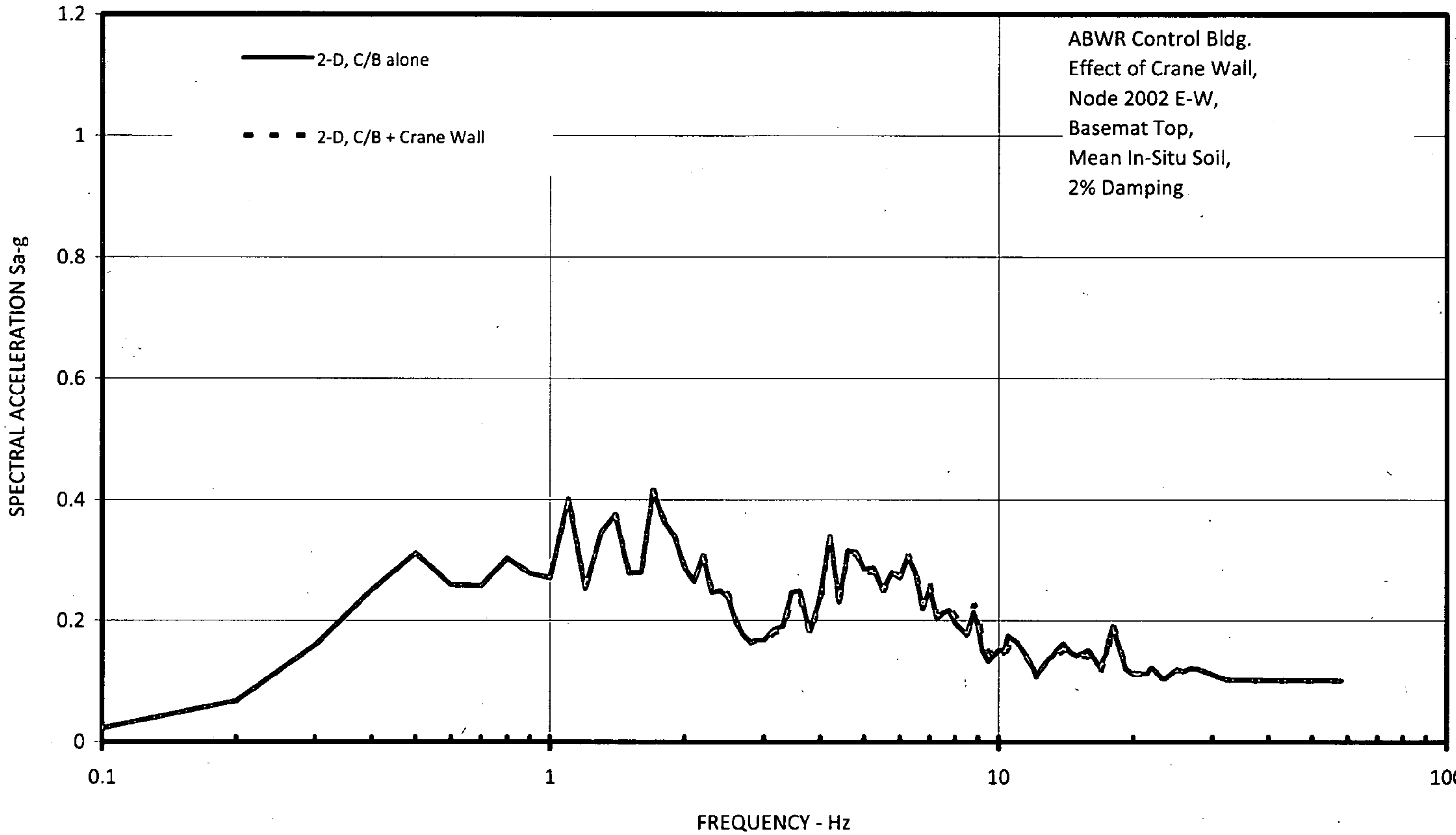


Figure 03.07.01-24g: Comparison of Response Spectra, Top of Control Building

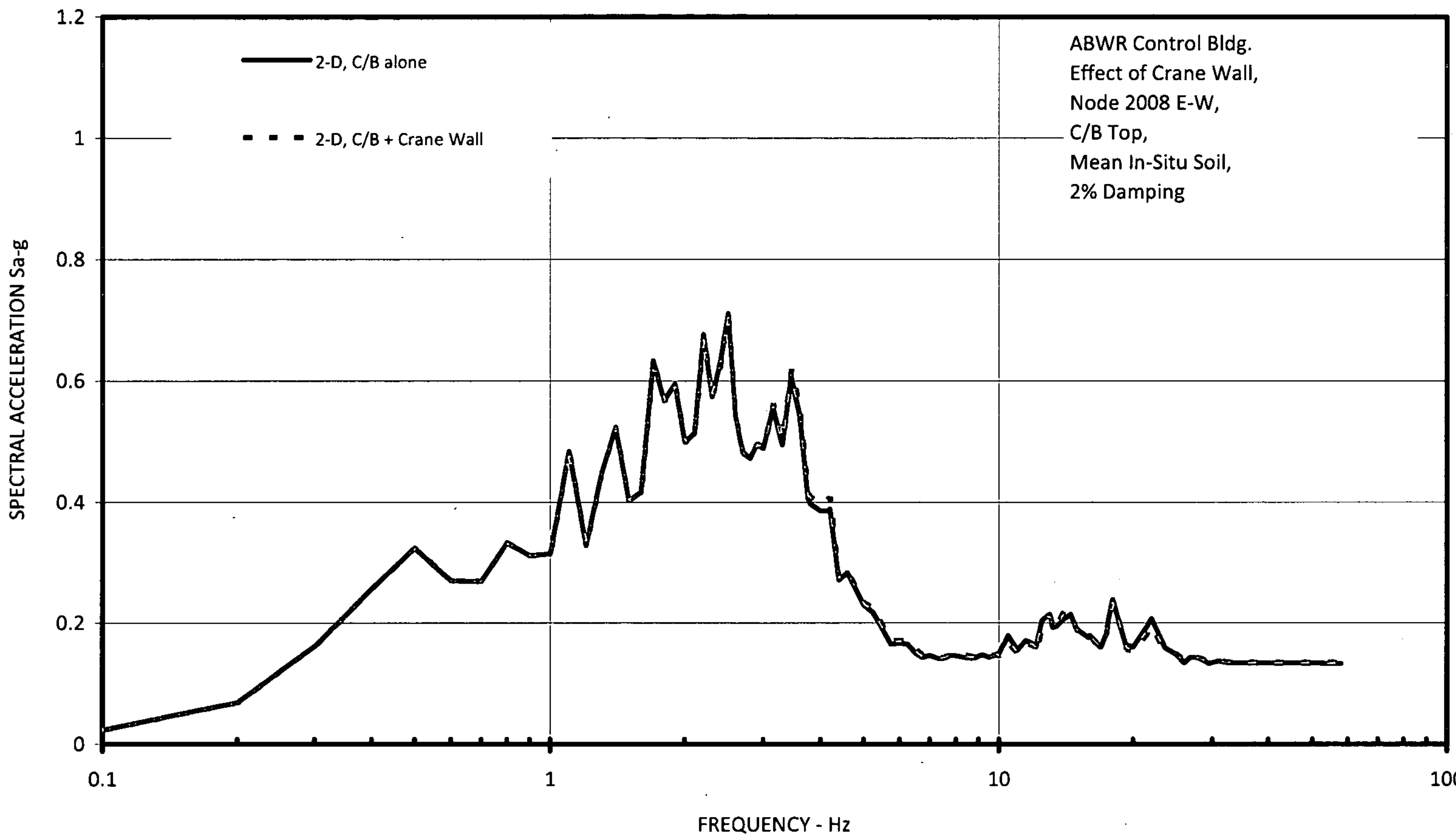
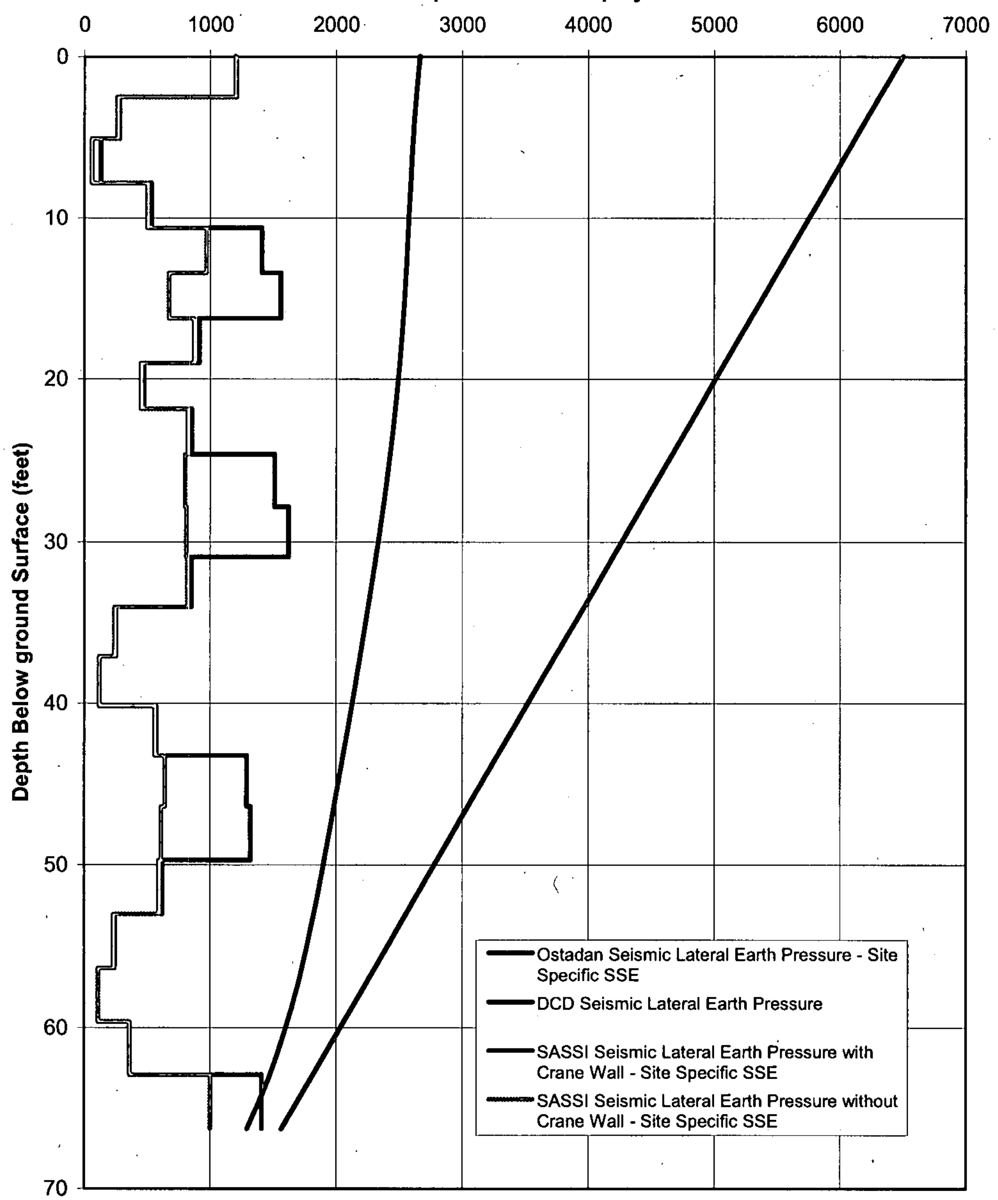


Figure 03.07.01-24h
Control Building Site Specific
At-Rest Seismic Lateral Earth Pressure (psf)
Multiple Methods Displayed



RAI 03.08.01-8, Supplement 1

QUESTION:

Follow-up question to Question 03.08.01-5 (RAI 2962)

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. These are incorporated by reference in FSAR Section 3H. Also, pool swell loads are used in loading combinations for design of the containment structure, and analysis and design results for the containment structure are reported in Appendix 3H. Since the changes in loads due to increases 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 that 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.

SUPLEMENTAL RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010. In a meeting with the NRC on January 19 and 20, 2010, STPNOC agreed to provide the evaluation of the effect of revised pool swell load. This evaluation is provided in this response.

The effect of increased pool swell (PS) height and pressure has been evaluated. The results of the evaluation 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 that 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 the ABWR DCD. The design of the Reinforced Concrete Containment Vessel (RCCV) and internal structures, as described in ABWR DCD Section 3H.1.5, considered selected load combinations as delineated in ABWR DCD Tables 3H.1-5a and 3H.1-5b. In these load combinations, Condensation Oscillation (CO) was considered to be the controlling loss-of-coolant accident (LOCA) (CO, Chugging (CH), or PS) load, as these three LOCA loads do not occur simultaneously. For the containment and containment internal structures described in ABWR DCD Sections 3H.1.5.5.1, 3H.1.5.5.2, and 3H.1.5.5.3, the load combinations including CO + Pa (Pa is the containment pressure associated with the LOCA) loads governed over the load combinations including PS + Pa loads, as shown in Figures 03.08.01-8A (which shows the magnitude and distribution of the DCD PS and Pa loads) and 03.08.01-8B (which shows the magnitude and distribution of the CO and Pa loads). It has been concluded that the load combinations including CO + Pa loads still govern over the load combinations including the revised PS + Pa loads, as shown in attached Figures 03.08.01-8B and 03.08.01-8C (which shows the magnitude and distribution of the revised PS and Pa loads). A summary of the comparison, based on the data from Figures 03.08.01-8A through 03.08.01-8C, is included in the table below:

(All units kPaG)	Based on DCD PS and CO Loads		Based on Revised PS Load
	PS + Pa	CO + Pa	PS + Pa
Diaphragm Floor differential pressure	69 (conservative)	69	36
Reinforced Concrete Containment Vessel (RCCV) wall pressure	108 to 175	241 to 520	146 to 195
Reactor Pressure Vessel Pedestal pressure	108 to 241	241 to 520	146 to 195
Basemat pressure	175 to 241	520	195

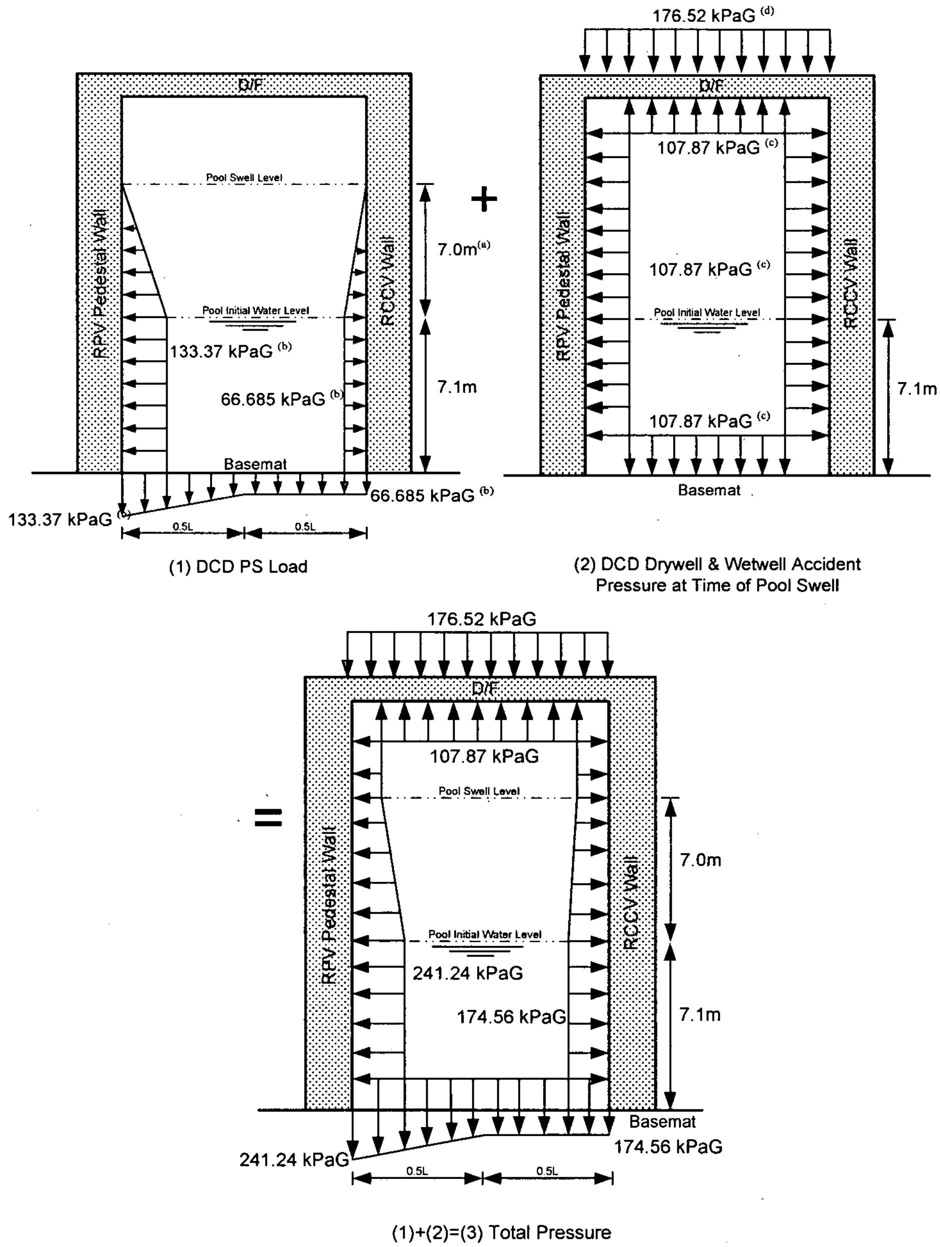


Figure 03.08.01-8A DCD Pool Swell Load

Notes:

(a) Maximum Pool Swell Height given in DCD Table 3B-1
(b) Maximum Air Bubble Pressure, during pool swell loading, included in DCD Table 3B-1 and Figure 3B-11
(c) Maximum Wetwell Airspace Pressure, during the pool swell, included in DCD Table 3B-1
(d) The drywell pressure at the time of pool swell is not given in the DCD. It is noted that the drywell and wetwell pressures during the pool swell are time-dependent. The maximum differential pressure (309.9 - 241.25 = 68.65 kPaG) during the Large Break LOCA included in DCD Table 3H.1-2 can be conservatively used, given that the differential pressure between drywell and wetwell pressure remained constant during and after the pool swell. For the purpose of this comparison, the drywell pressure is determined by adding the maximum differential pressure to the maximum wetwell pressure during the pool swell described in Note (c).

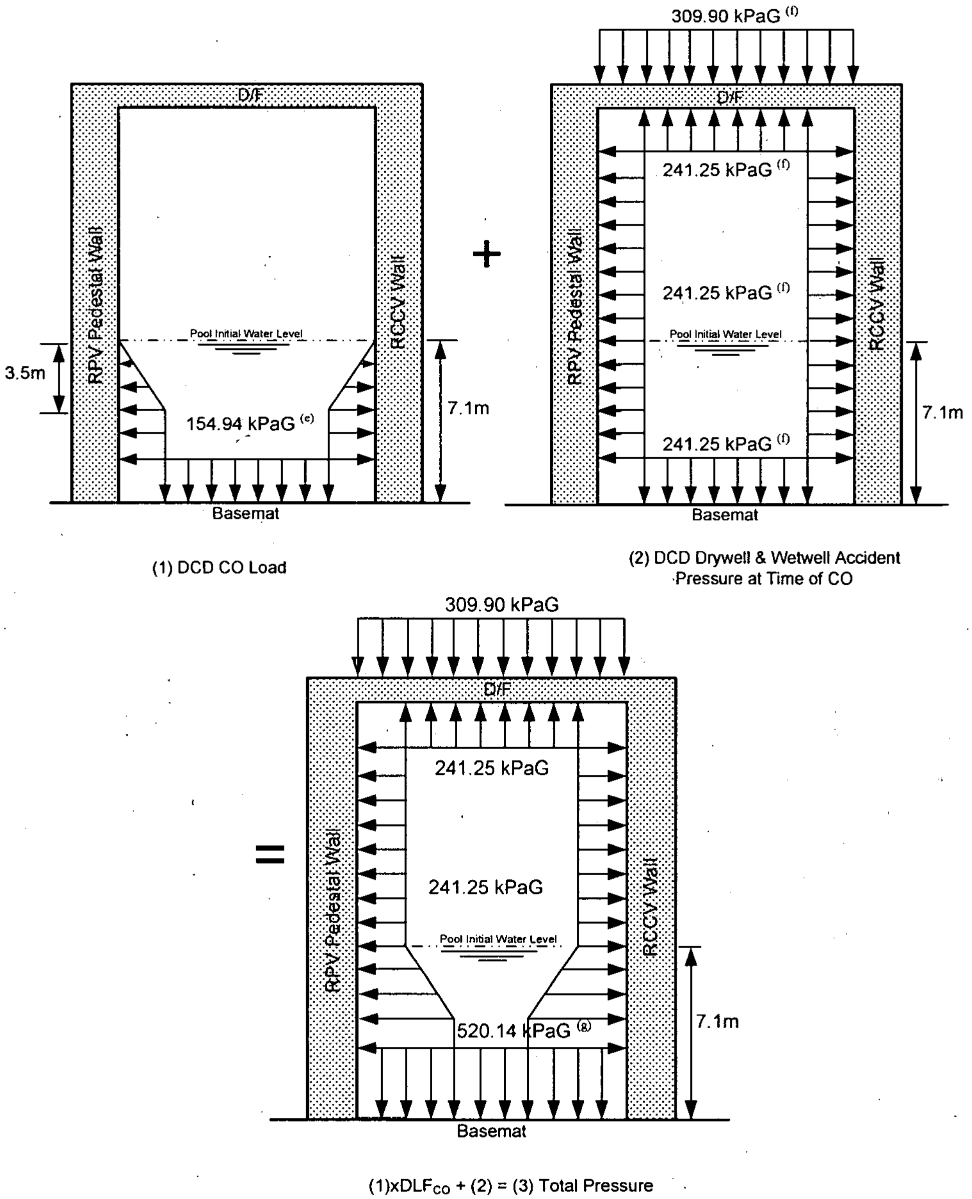


Figure 03.08.01-8B DCD CO load

Notes:
(e) CO Load given in DCD Table 3H.1-3
(f) Controlling Drywell and Wetwell Accident Pressures, at the time of CO load, are given in DCD Table 3H.1-2. It is noted that the drywell and wetwell pressures are time-dependent and increase to the maximum values during the condensation oscillation period of a postulated LOCA which follows the pool swell transient (DCD Section 3B.4.3.2)
(g) DCD CO load (e) x Dynamic Load Factor (given in DCD Table 3H.1-3) + Drywell & Wetwell pressures given in Note (f). Note: $DLF_{CO} = 1.8$

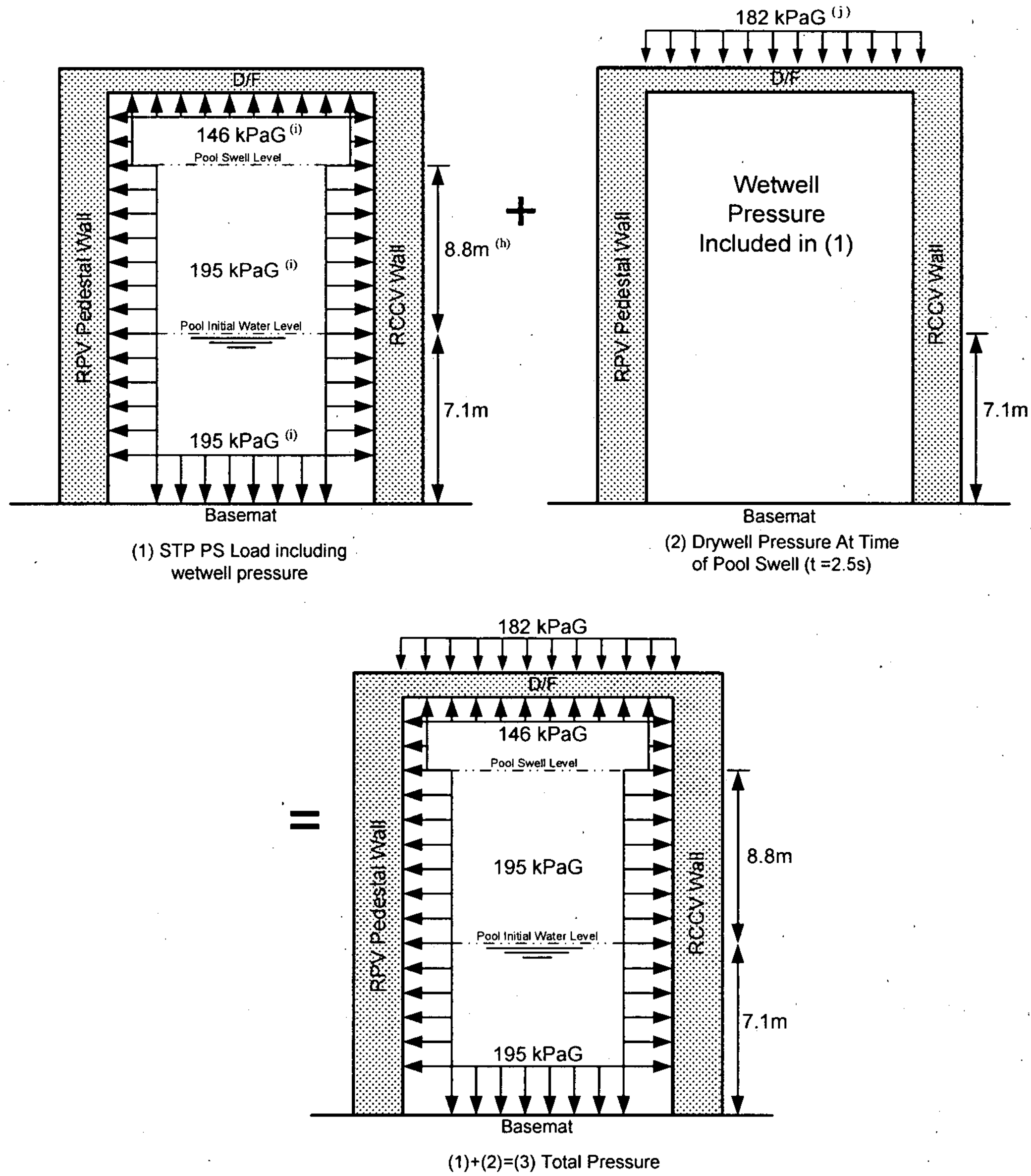


Figure 03.08.01-8C Revised Pool Swell Load

Notes:

(h) Maximum Pool Swell Height given in FSAR Table 3B-1

(i) Maximum Air Bubble Pressure, during STP Pool Swell loading, given in FSAR Table 3B-1and New Pool Swell Pressure Load Distribution

(j) Maximum Drywell Pressure at the maximum air bubble pressure (t = 2.5 second)

The RCCV penetrations that are now located within the revised pool swell impact zone will be located above the revised pool swell height (7700 mm) and COLA Part 2, Tier 2, Table 6.2-8 will be revised as shown below.

Table 6.2-8 Primary Containment Penetration List

Penetration Number	Name	Elevation (mm)	Azimuth (deg)	Offset (mm)	Diameter (mm)	Barrier Type	Testing†‡
X-102G	I & C	13500	180	-1175	300	O-ring	B
X-102H	I & C	13500	180	-5250	300	O-ring	B
X-102J	I & C	13500	55	1100	300	O-ring	B
X-103A	I & C	~~6000~~ See note 1	340.5	0	150	O-ring	B
X-103B	I & C	~~6000~~ See note 1	211	0	150	O-ring	B
X-103C	I & C	~~6000~~ See note 1	134	0	150	O-ring	B
X-103D	I & C	~~6000~~ See note 1	295	5600	150	O-ring	B
X-103E	I & C	~~6000~~ See note 1	211	1350	300	O-ring	B
X-104A	FMCRD Position Indicator	19000	81	0	300	O-ring	B
X-104B	FMCRD Position Indicator	19000	260.5	0	300	O-ring	B
X-104C	FMCRD Position Indicator	20100	99	1350	450	O-ring	B
X-322C	I & C	400	102	0	90	Valve	A
X-322D	I & C	400	282	0	90	Valve	A
X-322E	I & C	~~1400~~ See Note 1	106	0	90	Valve	A
X-322F	I & C	~~1400~~ See Note 1	282	0	90	Valve	A
X-323A	I & C	-5200	30	0	90	Valve	A
X-323B	I & C	-5200	210	0	90	Valve	A
X-323C	I & C	-5500	138	0	90	Valve	A

Note 1: Penetration will be located such that bottom of penetration sleeve is above revised pool swell impact zone (7700 mm).

COLA, Part 7, Section 2.3 will be revised as follows:

STD DEP 3B-2, Revised Pool Swell Analysis

Description

This departure updates the hydrodynamic loads analysis to incorporate a new analysis method for pool swell compared to the method described in the DCD. It is necessary to revise the pool swell analysis to address the effects of the changes to the containment pressure response for LOCA events as described in STD DEP 6.2-2. The COL applicant no longer has access to the analytical codes described in DCD Section 3B Reference 14, and an alternate method is used to perform the revised pool swell analysis. This alternate method utilizes a calculation approach that is similar to the DCD approach; however, it uses some different assumptions and different analytical software for implementation of the analysis. This change affects Tier 2 Appendix 3B Subsections 3B.4.2.1, 3B.7, and Table 6.2-8.

RAI 03.08.04-25, Revision 1

QUESTION:

Follow-up to Question 03.08.04-15 (RAI 3323)

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.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100036, dated February 10, 2010 and committed to provide gap size information. The following revised response provides the gap size information and completely supersedes the original response. The revised portion of the response is marked with a revision bar.

The joint is designed to accommodate the expected relative building movements without transmitting significant forces. The separation gap between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings, as well as the Diesel Generator Fuel Oil Storage Vaults and the Diesel Generator Fuel Oil Tunnels, will be at least 50% larger than the absolute sum of the calculated displacement 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 (based on 50% margin or a commensurate value if a margin larger than 50% is provided) without subjecting the building to more than a negligible force relative to the resistance capacity of the building.

The joint material will be a polyurethane foam impregnated with a waterproof sealing compound, or a similar material. Typical vendor data indicates that the material tensile strength is about 21 psi. Vendor testing for this material in a 5 inch joint compressed to 50% movement has a 7 psi compressive stress in the compressed condition. Considering the negligible strength and limited area of the sealing material compared to strength (minimum compressive strength (f'c) of

4000 psi) and massive size of the tunnels and abutting structures the effect on interaction between structures, if any, is negligible.

To minimize the movements due to settlement, the complete installation of the details will not occur until after the short term settlement is substantially complete.

The values for the required and provided separation gaps due to seismic movements plus long term settlement are provided in the attached COLA mark-up.

Because of the low rate with which groundwater can flow through the detail if it were to fail in any particular location, in-leakage of groundwater is a housekeeping issue and not a safety concern. Even a degraded flexible filler material acts as a sieve to slow the flow of groundwater into the building/tunnel. Constant exposure to groundwater may deteriorate the waterproofing material. However, the detail provided (Figure 03-08-04-15A) with the response to RAI 03.08.04-15 (see letter U7-C-STP-NRC-090160, dated October 5, 2009) allows the waterproofing material to be replaced if it becomes degraded or for inspections as required.

The COLA will be revised as shown on the following pages as a result of this response.

COLA Part 2, Tier 2, will be revised to add a new Section 3H.6.8 and new Table 3H.6-11, as shown below.

1. Add the following new subsection 3H.6.8 and revise the subsection number for References.

3H.6.8 Seismic Gaps at the Interface of Site-Specific Seismic Category I Structures and the Adjoining Structures

The joints (i.e. separation gaps) at the interface of site-specific seismic category I structures (Reactor Service Water Tunnels and Diesel Generator Fuel Oil Storage Vaults) with the adjoining structures (Control Buildings, Reactor Service Water Pump Houses, and Diesel Generator Fuel Oil Tunnels) are designed to accommodate the expected movements without transmitting significant forces. These separation gaps are sized at least 50% larger than the absolute sum of the maximum calculated displacements due to seismic movements and long term settlement. The joint material used as flexible filler will be polyurethane foam impregnated with a waterproofing sealing compound, or a similar material, capable of being compressed to 1/3 of its thickness without subjecting the structures to more than a negligible pressure of about seven psi.

Table 3H.6.11 provides summary of the required and provided gaps at the interface of site-specific seismic category I structures with adjoining structures.

~~3H.6.8~~3H.6.9 References

2. Add new Table 3H.6-11 shown below

Table 3H.6-11: Required and Provided Gaps at the Interface of Site-Specific Seismic Category I Structures and the Adjoining Structures

Interfacing Structures	Required and Provided Gaps (inches)	
	Required Gap	Provided Gap
RSW Piping Tunnels and Control Building	4.13	4.5
RSW Pump House and RSW Piping Tunnel A	3.51	4.5
RSW Pump House and RSW Piping Tunnel B	4.44	4.5
RSW Pump House and RSW Piping Tunnel C	2.59	4.5
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 1 and its Diesel Generator Fuel Oil Tunnel	1.49	2.0
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 2 and its Diesel Generator Fuel Oil Tunnel	1.67	2.0
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 3 and its Diesel Generator Fuel Oil Tunnel	1.5	2.0

RAI 03.08.05-2, Revision 1

QUESTION:

Follow-up to Question 03.08.05-1 (RAI 3324)

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 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.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010. In a meeting with the NRC on January 19 and 20, 2010, STPNOC agreed to provide additional information regarding differential movements for commodity design. The following revised response provides this information and completely supersedes the previous response. The revised portion of the response is marked with a revision bar.

There are three different effects of settlements which need to be considered in design of structures. Each of these effects is discussed in the following paragraphs.

a. Rigid Body Angular Distortions/Tilts

COLA Part 2, Tier 2, Section 2.5S.4.10 presents conservatively calculated angular distortions/tilts based on conservatively estimated differential settlements of each structure. The calculation assumed a perfectly flexible structure with no applied reduction due to buoyancy or structural rigidity. As explained in the response to RAI 03.08.05-3, the calculated tilt values are acceptable and no additional consideration is needed in the design of structures for these tilt values.

b. Differential Settlement due to Flexibility of Structure/Basemat and Supporting Soil

Settlements due to flexibility of structure/basemat and supporting soil induce stresses within the structure. In the analysis and design of the site-specific seismic category I structures, this effect is accounted for through the use of Finite Element Analysis (FEA) in conjunction with foundation soil springs. FEA representation of the structure accounts for the flexibility of structure/basemat, and the soil springs with their stiffness based on subgrade modulus, which is a function of the foundation settlement, account for the flexibility of the supporting soil medium. The information on the analysis and design of the site-specific structures provided in Supplement 2 to the response to RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090230, dated December 30, 2009) is based on the FEA that includes the foundation soil springs, and thus incorporates the effect of this differential settlement.

c. Differential Settlement between Buildings

Differential settlements due to structural backfill, loading of other structures and consolidation of clay layers result in differential settlements between the buildings and angular distortions/tilts. These differential settlements and angular distortions/tilts will impact the design of commodities and tunnels running between the buildings and the seismic gaps among the adjacent buildings. The magnitude of these impacts will be minimized by delaying final connections to a time when the majority of the differential settlements and angular distortions/tilts have already taken place. The timing for the final connection of such commodities and tunnels will be established based on time-rate of settlement analyses described in the response to RAI 02.05.04-30 (see STPNOC letter U7-C-STP-NRC-090146 dated September 21, 2009).

The total movement for design of commodities and tunnels running between buildings and seismic gaps of the adjacent buildings are determined considering the differential settlements and angular distortions/tilts from the time-rate of settlement analysis and any additional movement during a seismic event.

Based on the results of time-rate of settlement analyses, the differential movements for the design of commodities running between the site-specific seismic category I structures (RSW Piping Tunnels and Diesel Generator Fuel Oil Storage Vaults) and the adjoining structures (Control Buildings, RSW Pump Houses, and Diesel Generator Fuel Oil Tunnels) are as shown in the attached Table 03.08.05-2a. The gap information between the site-specific Seismic

Category I structures and the adjoining structures is provided in the revised response to RAI 03.08.04-25, being submitted concurrently with this letter.

The information on the analysis and design of the site-specific structures has been provided in Supplement 2 to the response to RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090230, dated December 30, 2009).

No COLA revision is required as a result of this response.

Table 03.08.05-02a:

Differential Movements for Design of Commodities Running between Site-Specific Seismic Category I Structures and Adjoining Structures

Location	Differential Movement (inches)		
	North-South	East-West	Vertical
RSW Piping Tunnels and Control Building (for RSW Piping)	1.99	1.40	1.44
RSW Pump House and RSW Piping Tunnel A (for RSW Piping)	1.35	1.06	1.33
RSW Pump House and RSW Piping Tunnel B (for RSW Piping)	1.35	1.06	1.08
RSW Pump House and RSW Piping Tunnel C (for RSW Piping)	1.35	1.06	1.52
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 1 and its Diesel Generator Fuel Oil Tunnel (for Fuel Oil Piping)	1.03	1.88	0.93
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 2 and its Diesel Generator Fuel Oil Tunnel (for Fuel Oil Piping)	1.16	1.66	1.08
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 3 and its Diesel Generator Fuel Oil Tunnel (for Fuel Oil Piping)	1.66	1.18	1.18

RAI 03.08.05-3, Revision 1

QUESTION:

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.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010. In a meeting with the NRC on January 19 and 20, 2010, STPNOC was requested to provide additional information on the acceptance criterion for angular distortion / tilt. The following revised response completely supersedes the previous response. The revised portion of the response is marked with a revision bar.

The ABWR DCD does not contain any criteria for settlement-related angular distortions/tilts.

The angular distortions/tilts determined for the structures for the STP site are compared in the COLA to commonly-stated industry empirical values in the literature.

The criteria presented in COLA Part 2 Tier 2, Section 2.5S.4.10 indicates acceptable angular distortions/tilts of less than 1/300 as presented by Bowles in Table 5-7 (Reference 5, which is the same as COLA Reference 2.5S.4-55) and by Das in Section 5.20 (Reference 6). The 1/300 criteria presented by Bowles and Das is based on work by MacDonald and Skempton (Reference 1). MacDonald and Skempton reported that an angular distortion of 1/300 will cause fractures and 1/150 will cause structural damage. Bjerrum (Reference 2) presented his limiting angular distortion values incorporating the MacDonald and Skempton work. Bjerrum indicates an angular distortion of 1/500 as a safe limit (including a factor of safety) for no cracking in buildings. Bjerrum indicates 1/300 as first cracking of panel walls and 1/150 as a limit for danger of structural damage to most buildings. The 1/300 angular distortion criteria of MacDonald and Skempton were also substantiated by Feld (Reference 3) and a study by Grant, Christian, and Vanmarcke (Reference 4). Thus, the generally accepted engineering standard of practice is to allow for an angular distortion of no greater than 1/300. Nevertheless, an angular distortion value of 1/500, as recommended by Bjerrum (Reference 2), is more conservative and will be used as an acceptance criterion for the Category I structures.

This 1/500 criterion is adopted as applying to the differential settlement between local points on the foundation (angular distortion) as well as to the structure tilts (the differential settlement from one side to the other side or one end to the other end).

The calculated maximum flexible angular distortions/tilts presented in COLA Part 2, Tier 2, Section 2.5S.4.10 range from 1/400 to 1/1750 for Seismic Category I structures. As noted in Section 2.5S.4.10.4, the structure/basemat rigidity will reduce the angular distortions values to at most half of those shown, or to 1/800 to 1/3500 which are well within the acceptance criterion of 1/500. Angular distortion of the Seismic Category I structures of the STP Units 3 & 4 is therefore acceptably low. In the structural analysis and design of these structures, the induced stresses within the structure due to structural and foundation flexibility are accounted for as explained in the response to RAI 03.08.05-2.

Note that the tilting of a structure from end to end or side to side would be influenced by the time sequence of application of settlement-producing loads from backfilling and other structural loads beside the tilting structure. This is considered qualitatively in ratios described in the COLA. Section 2.5S.4.10.4 notes that it is expected that by the time building superstructures are ready to receive equipment and/or piping, a significant amount (i.e., more than half) of foundation settlements are expected to have already taken place; resulting in the angular distortions/tilts values to be half of those presented in the COLA. Tilt angles will therefore be well within the criterion of the 1/500 limit in Reference 2. Tilt of the Seismic Category I structures of the STP Units 3 & 4 is therefore acceptably low.

For impact of the site-specific differential settlement between adjacent structures on the tunnels and other commodities between the buildings, please see the revised response to RAI 03.08.05-2, being submitted concurrently with this letter.

References (used herein for this RAI response):

1. MacDonald, D.H. and Skempton, A.W., 1955, "A Survey of Comparisons between Calculated and Observed Settlements of Structures on Clay", Conference on Correlation of Calculated and Observed Stresses and Displacements, ICE London.

2. Bjerrum, L., 1963, "Allowable Settlement of Structures", Proceedings European Conference on Soil Mechanics and Foundation Engineering, Vol. III.
3. Feld, J., 1965, "Tolerance of Structures to Settlement: J. Soil Mech. And Found. Div., 91 (SM3), ASCE", pp. 555-569.

4. Grant, R., Christian, J.T., Vanmarcke, E.H., 1994, "Differential Settlement of Buildings", Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT9, Proceedings Paper 10802.

5. FSAR Reference 2.5S.4-55. Bowles, J. E., 1996, "Foundation Analysis and Design, (5th edition)".

6. Das, Braja, M., 2007, "Principles of Foundation Engineering," 6th Edition.

The COLA will be revised as follows:

The first full paragraph on Page 2.5S.4-134 of Revision 3 to COLA Section 2.5S.4.10 will be revised as shown below:

> Total settlements such as calculated in Table 2.5S.4-42 can be accommodated when critical connections to adjacent structures, utilities, and pavements can be delayed. Differential settlements are usually more important in the context of structure performance than total settlements, with acceptable angular distortion/tilt of the order of 1/300, generally reported for frame buildings (Reference 2.5S.4-55), to as low as 1/750 for foundations supporting sensitive machinery (Reference 2.5S.4-59), having been suggested. Reference 2.5S.4-55A recommends an angular distortion /tilt criterion of 1/500. This value (1/500) includes additional safety factor and will be used as the acceptance criterion for assessing the Seismic Category I structures.

The last paragraph on Page 2.5S.4-134, continuing to Page 2.5S.4-135 of COLA Revision 3 will be revised as shown below:

> Foundations evaluated had estimated differential settlements in the range of approximately 0.4 inches to 2.3 inches (measured from center to edge of structure) for the flexible case. From the differential settlement values, angular distortions/tilts were estimated (based on average foundation plan dimension), and for all evaluated structures were within the acceptable limit of ~~1/300~~ 1/500. From the differential settlement values, calculated angular distortion/tilt values for the flexible case exceeded the 1/750 criterion for the special case of foundations supporting sensitive machinery for only the RSW Pump Houses and Diesel Generator Fuel Oil Storage Vaults (No.1). The calculated angular distortion/tilt values were less than the 1/750 criterion for the Reactor Buildings and Control Buildings, UHS Basins, RSW Tunnels, and Diesel Generator Fuel Oil Storage Vaults No. 2 and No. 3. However, it should be noted that despite the calculated total settlement for the referenced foundations, and the angular distortion/tilt values, actual angular distortion/tilt values are much less even for the flexible case, given that a significant amount (i.e., more than half) of foundation settlements are expected to have taken place by the time building superstructures are ready to receive equipment and/or piping. In this case, estimated angular distortion/tilt would similarly be one-half of those calculated above, or approximately 1/1200 to 1/1500 for the Reactor Buildings, and 1/800 to 1/900 for the Control Buildings, 1/1300 to 1/1400 for the UHS Basins, 1/3400 to 1/3500 for the RSW Pump Houses, 1/1400 for the RSW Tunnels, 1/2000 to 1/2100 for the Diesel Generator Fuel Oil Storage Vault No. 1, 1/1000 to 1/1100 for the Diesel Generator Fuel Oil Storage Vault No. 2, and 1/1300 to 1/1500 for the Diesel Generator Fuel Oil Storage Vault No. 3. These are well within the stricter criterion for the special case of foundations supporting sensitive machinery and the 1/500 limit of Reference 2.5S.4-55A. Note, more significantly, that settlement estimates were based on the assumption of flexible mat foundations, not including the effects that thick, highly-reinforced concrete mat foundations have in mitigating differential settlements. To verify

that foundations perform according to estimates, and to provide an ability to make corrections if needed, major structure foundations are monitored for movement during and after construction.

The third paragraph on Page 2.5S.4-135 of COLA Revision 3 will be revised as shown below:

Construction sequencing will be necessary to address the time-rate of settlement for the Category 1 structures. The structural and mechanical considerations (addressed during design) will influence differential settlement tolerances between structures. Experience during settlement monitoring of STP Units 1 & 2 (Reference 2.5S.4-3) will be used to assist with the time-rate of settlement projections. The acceptance criteria for differential settlement ~~of~~ between Category 1 structures will be developed during design of these structures and will be consistent with the DCD.

The third paragraph on Page 2.5S.4-141 of COLA Revision 3 to Section 2.5S.4.11 will be revised as shown below:

Subsection 2.5S.4.10 specifies and discusses allowable bearing capacity and settlement values for site soils and for planned Seismic Category I structures. Mat foundations will be used for all Seismic Category I structures. Table 2.5S.4-42B provides ultimate bearing capacity for Seismic Category I structures. Generally, a minimum FOS=3.0 was used when applying ultimate bearing capacity equations when static loading conditions apply. This FOS can also be applied against breakout failure due to uplift forces on buried piping. This FOS can be reduced to 1.5 when dynamic or transient loading conditions apply (Reference 2.5S.4-69). Table 2.5S.4-47 shows estimated structure total settlements under the stated foundation loads. As a guideline, if total and differential settlements are limited to 3 inches (up to 5 inches) and 1.5 inches, respectively, for mat foundations (and angular distortions/tilts do not exceed ~~1/300~~ 1/500, or 1/750 for foundations supporting sensitive machinery), then settlements do not impact foundation performance. Higher total settlements such as calculated for the STP 3 & 4 Reactor Buildings can be accommodated when critical connections to adjacent structures, utilities, and paving are delayed.

A new reference will be added on page 2.5S.4-148 of COLA Revision 3 to Section 2.5S.4.13, as shown below:

2.5S.4-55A "Allowable Settlement of Structures," Proceedings European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Germany, Vol. III, pp. 135-137., Bjerrum, L., 1963.