Nuclear Operating Company

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

November 29, 2010
U7-C-STP-NRC-100253

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
Revised Response to Request for Additional Information

Attached are supplemental and revised responses to NRC staff questions included in Request for Additional Information (RAI) letter numbers 345, 349, 350, and 358 related to Combined License Application (COLA) Part 2, Tier 2, Sections 3.4, 3.7, and 3.8. The attachments address the responses to the RAI questions listed below:

03.04.02-6	03.08.01-4
03.04.02-11	03.08.01-7
03.07.01-25	03.08.01-9
03.07.02-24	03.08.01-10

There are no commitments in this response.

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

If you have any questions regarding these responses, please contact Scott Head at (361) 972-7136, or Bill Mookhoek at (361) 972-7274.

D091
NRO

STI 32789128

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

Executed on 11/29/2010

MA McBurnett

Mark McBurnett
Vice President, Oversight & Regulatory Affairs
South Texas Project Units 3 & 4

jep

Attachments:

1. RAI 03.04.02-6, Revision 2
2. RAI 03.04.02-11, Revision 1
3. RAI 03.07.01-25, Supplement 1
4. RAI 03.07.02-24, Supplement 1
5. RAI 03.08.01-4, Revision 2
6. RAI 03.08.01-7, Revision 2
7. RAI 03.08.01-9, Revision 1
8. RAI 03.08.01-10, Revision 1

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

Director, Office of New Reactors
U. S. Nuclear Regulatory Commission
One White Flint North
11555 Rockville Pike
Rockville, MD 20852-2738

Regional Administrator, Region IV
U. S. Nuclear Regulatory Commission
611 Ryan Plaza Drive, Suite 400
Arlington, Texas 76011-8064

Kathy C. Perkins, RN, MBA
Assistant Commissioner
Division for Regulatory Services
Texas Department of State Health Services
P. O. Box 149347
Austin, Texas 78714-9347

Alice Hamilton Rogers, P.E.
Inspection Unit Manager
Texas Department of State Health Services
P. O. Box 149347
Austin, Texas 78714-9347

*Steven P. Frantz, Esquire
A. H. Gutterman, Esquire
Morgan, Lewis & Bockius LLP
1111 Pennsylvania Ave. NW
Washington D.C. 20004

*Tom Tai
Two White Flint North
11545 Rockville Pike
Rockville, MD 20852

(electronic copy)

*George F. Wunder
*Tom Tai
Loren R. Plisco
U. S. Nuclear Regulatory Commission

Steve Winn
Joseph Kiwak
Eli Smith
Nuclear Innovation North America

Peter G. Nemeth
Crain, Caton & James, P.C.

Richard Peña
Kevin Pollo
L. D. Blaylock
CPS Energy

RAI 03.04.02-6, Revision 2

QUESTION:

In its evaluation of **RAI 03.04.02-2 (ID 3322 Question 13162)**, the staff accepts in general the applicant's physical description of watertight door locations and the proposed measures and procedures to accomplish water tightness of any below DBFL openings and penetrations of seismic category I, in-and out-of-scope SSC, as reflected in the proposed revision to COLA FSAR. The staff considers that since watertight doors are seismic category I SSC, each exterior door under DBFL located in any category I structure should be given a unique component ID, a set of specific design parameters, other conditions (e.g., controls measures) and be keyed into the corresponding plans to show each door's location. Such information should be reflected in the ITAAC tables conveying the design requirements, the proposed inspections, tests, analyses and the acceptance criteria including the need for as-built reconciliation which is required for category I SSC. All certified and plant-specific category I SSC should be considered, including the underground diesel tanks and vaults if applicable. Compliance with RG1.102 Flood Protection for Nuclear Power Plants should also be indicated for the underground diesel tank access openings if applicable. The staff needs this information to be able to conclude that the seismic category I doors are designed and installed to withstand the design basis flood during an accident.

REVISED RESPONSE:

The Revision 1 response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. This revision completely supersedes the previously submitted Revision 1 response for this RAI. The revisions are marked with revision bars in the margin. This revision updates the value used for the density of the flood water to include the weight of sediment and clarifies testing requirements for water stops and joint seals as discussed during the NRC audit held during the week of October 18, 2010.

Each of the exterior watertight doors used for protection against the Design Basis Flood (DBF) will be given a unique component ID. The specific design parameters and other conditions will be contained in the purchase specification for the doors, and are included in the COLA markups included with this response. The design commitments, as-built reconciliation requirements, required inspections, tests, analyses and acceptance criteria for penetrations in exterior walls below design basis flood level are included in ITAAC Tables 2.15.10 and 2.15.12. ITAAC Table 2.15.10 also applies to the watertight doors in the Diesel Generator Fuel Oil Storage Vaults. The ITAACs for both the Reactor Building (Table 2.15.10) and the Control Building (Table 2.15.12) state that "Penetrations in the external walls below flood level are provided with flood protection features." The ITAACs for both buildings state that they are protected from external flooding events and require a Flood Analysis Report that includes the results of inspections of the as-built flood protection features.

This RAI response will impact previously submitted responses to the following RAIs and COLA Sections.

- COLA Part 2, Tier 2 Sections 3.4.3.1 and 3.4.3.3
- COLA Part 2, Tier 2 Section 2.4S.10
- RAI 03.04.02-2
- RAI 03.08.01-3
- RAI 03.08.01-6
- RAI 14.03.02-9
- RAI 19-30

The markup to the COLA Revision 4 sections is presented in Enclosure 1. The revised responses to the above RAIs were submitted with the Revision 1 response of this RAI, which are not affected by the changes made in this revision. The COLA Revision 4 markups include the description of loads, load combinations, and acceptance criteria for the watertight doors. Please note that Section 3H.6.7, which is referenced in the revision to Section 3.4.3.3, was submitted with response to RAI 03.07.01-19 Revision 2, as submitted in STPNOC letter U7-C-STP-NRC-100129, dated June 7, 2010.

Enclosure 2 has been removed from this RAI response because it has been incorporated into COLA Revision 4 in its entirety.

RAI 03.04.02-6 Revision 2
Enclosure 1
Revisions to COLA Part 2, Tier 2, Sections 3.4.3.1 and 3.4.3.3

3.4.3.1 Flood Elevation

The following site specific supplement addresses COL License Information Item 3.5.

The site specific design basis flood elevation is defined as 182.9 cm above grade. The design basis flood is described in Subsection 2.4S.2.

As described in Table 3.4-1 note 3 and 5, all penetrations and doors that penetrate the exterior walls of Seismic Category I Buildings that are located below the design basis flood level are watertight. Therefore all safety-related equipment in these buildings are protected from postulated external floods and satisfy the requirements of GDC 2.

Watertight doors or barriers are provided on the Reactor Building and Control Building to protect the buildings from the external design basis flood. These watertight doors or barriers are considered Seismic Category I components. In order to ensure that the watertight doors and barriers can withstand the ABWR Standard Plant loading requirements, the watertight doors and barriers of the Reactor Building and Control Building will be designed for the more severe of the standard plant and site-specific loading. Watertight doors shall be designed to meet the Incorporated Barrier requirements of Regulatory Guide 1.102.

The watertight doors or barriers for the Reactor Building consist of the six exterior doors and the exterior Large Equipment Access indicated in Tier 1 Figures 2.15.10h and 2.15.10j. The watertight doors for the Control Building consist of the access doors between the Control Building and the Service Building shown in Tier 1 Figures 2.15.12d, e, and f, the exterior equipment access door shown in Tier 1 Figure 2.15.12g, and an access door between the Control Building and the Service Building shown in Tier 1 Figure 2.15.12g. Each door will be given a unique component ID in the construction drawings.

The locations for watertight doors in the Reactor Building and Control Building include:

Exterior Watertight Door or Barriers

Structure	Door or Barrier Description	Elevation
Reactor Building	Clean Access Area Corridor Entrance	B1F (4800 mm)
	Diesel Generator A Access	1F (12300 mm)
	Diesel Generator B Access	1F (12300 mm)
	Diesel Generator C Access	1F (12300 mm)
	East Equipment Hatch Access	1F (12300 mm)
	West Equipment Hatch Access	1F (12300 mm)
	Large Equipment Access	1F (12300 mm)
Control Building	HX Area Access at Service Building	B3F (-2150 mm)
	Electrical Area Access at Service Building	B2F (3500 mm)
	Control Building Access at Service Building	B1F (7900 mm)
	Entrance to Reactor Building Controlled Access	1F (12300 mm)
	Equipment Access	1F (12300 mm)

~~Exterior openings of the Reactor Building and Control Building that could make safety-related SSCs vulnerable to tornado missiles are protected by separate barriers or doors designed to resist tornado missiles.~~

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. Watertight doors will be individually engineered assemblies designed by the supplier to satisfy the design basis performance requirements for external flooding. Watertight doors and water stops will allow only slight seepage during an external flooding event in accordance with criteria for Type 2 closures in U.S. Army Corps of Engineers (COE) EP 1165-2-314, "Flood-Proofing Regulations". This criterion will be met under hydrostatic loading of 12 inches of water above the design basis flood elevation per Table 3.4-1, plus ~~the height of wave run-up and~~ drag effects, as required. Water retaining capability of the doors shall be demonstrated by qualification tests for the water head levels. These tests will be completed prior to shipment of the doors. For this purpose a test fixture may be used, with gasket material and cross section, its retainers, and the anvil configuration being identical to that of the full size doors. The test fixture shall have the necessary valving, pressure gages, flow meters, and instruments for measuring gasket compression. To validate that the door satisfies a Type 2 closure per (COE) EP 1165-2-314, the leakage shall not exceed 0.10 gallon/hour/linear foot of gasket when subjected to 125% of the specified head pressure. The hydrostatic head shall be raised at a rate not more than 1 ft/min. If leaks occur during the rising of the hydrostatic head and the leakage rate begins to diminish as the hydrostatic head increases, the assembly shall be tested at the hydrostatic head where the more substantial leakage was observed.

The joint seals between the Reactor Building and the Control Building below the design basis flood level shall be made using a polyurethane foam impregnated with a waterproof sealing compound. The joint shall also include a redundant water stop at the interior side of the joint.

The seals shall be tested to be watertight when subjected to the maximum anticipated hydrostatic head. The testing program will demonstrate that the seal material can withstand movement ±25% of the gap size in any resultant direction and still be watertight. This will demonstrate that the material is capable of being watertight after the effects of long term settlement or tilt, as well as during normal operating vibratory loading such as SRV actuation. Although this will provide margin to accommodate differential displacements from the majority of the movements from short duration extreme environmental loading such as SSE, the seals need not be designed to be watertight during the maximum differential displacements from these extreme environmental loadings. Testing will also demonstrate that the seal material will function as a watertight barrier after being subjected to the maximum displacements due to a SSE and that the redundant water stop on the interior side of the joint can withstand the SSE maximum displacements without degradation.

The joint seal and interior water stops used to protect the safety-related buildings against external water entry are classified as Seismic Category I with respect to their ability to remain in-place to stop significant water leakage into the safety-related buildings during and after a seismic event. An in-service inspection program will ensure that the joint seal materials do not significantly degrade.

The watertight doors or barriers that are utilized for protection against external flooding are normally closed and are used for egress, as required.

The watertight doors, frames, and all components are designed to the requirements of AISC N690 and SRP Section 3.8.4. The structural steel used for the watertight doors conforms to either ASTM A36, ASTM A992 or ASTM A500 Grade B. The faceplate conforms to ASTM A606, type 4 and the rubber gasket conforms to ~~ASTM D2000, Grade BC~~ ASTM D1056 Type 2 Class D. Fabrication of the doors shall meet the requirements of AISC N690. The welding shall meet the requirements of nondestructive testing, personnel qualifications and acceptance criteria contained in AWS D1.1.

The watertight doors shall be designed for the following loads and load combinations:

$S = D + W + P_o$

$1.6S = D + E' + P_o$

$1.6S = D + W_t + P_o$

$1.6S = D + FL + P_o$

$\cancel{S = D + W}$

$\cancel{S = D + P}$

$\cancel{1.6S = D + P + E'}$

~~$1.6S = D + W_t$ (See definition below)~~

Where:

S = Normal allowable stresses as defined in AISC N690

D = Dead Loads

E' = Loads generated by SSE, per Sections 3H.1 and 3H.2.

P_o = Loads due to normal operating differential pressure.

FL = Design basis extreme flood loads, including the hydrostatic load due to flood elevation at 40 ft MSL, the associated drag effects of 44 psf, hydrodynamic load due to wind-generated wave action per Figure 3.4-1, and impact due to floating debris per Section 3.4.2. The weight of the water (above ground) due to the flood loads shall be 63.85 pcf in order to include the effects of suspended sediments in the water.

~~P = Pressure Loads, which may be due to hydrostatic pressure, P_h, or differential pressure, P_d.~~

~~P_h = Loads due to hydrostatic pressure, determined based on the flood elevation per Table 3.4-1, plus the height of wave run-up and drag effects, as required.~~

~~P_d = Loads due to differential pressure~~

W = Normal Wind Loads, per Sections 3H.1 and 3H.2.

W_t = Tornado Loads, per Sections 3H.1 and 3H.2, including wind velocity pressure W_w, differential pressure W_p, and tornado-generated missiles (if not protected) W_m

The value used for W_t shall be computed to satisfy the following possible combinations:

$W_t = W_w$

$W_t = W_p$

$W_t = W_m$

$W_t = W_w + W_m$

$W_t = W_w + 0.5\ W_p$

$W_t = W_w + 0.5\ W_p + W_m$

3.4.3.3 Flood Protection Requirements for Other Structures

The following site specific supplement addresses COL License Information Item 3.7.

The Ultimate Heat Sink and Reactor Service Water Piping Tunnel have the same flood protection features as other Seismic Category I structures within the scope of the certified design. These design features are addressed in Subsection 3.4.1.1. As described in that

Subsection, they are protected from postulated flooding and satisfy the requirements of GDC 2 and the guidance of RG 1.102.

The Ultimate Heat Sink and Reactor Service Water Piping Tunnel are divisionally separated in accordance with Section 3.13 and 3.12. Penetrations that are located below design flood level are watertight thereby preventing an internal flood event from propagating from one division to another.

Watertight doors or barriers are provided on the site-specific Diesel Generator Fuel Oil Storage Vaults to protect the vaults from the external design basis flood. These watertight doors or barriers are considered site-specific Seismic Category I components. Each door will be given a unique component ID in the construction drawings.

The locations of watertight doors for the Diesel Generator Fuel Oil Storage Vaults include:

Exterior Watertight Door or Barrier ~~Component IDs~~

Structure	Door Description
Diesel Generator Fuel Oil Storage Vaults	Access to Vault A
	Access to Vault B
	Access to Vault C

The design requirements for Diesel Generator Fuel Oil Storage Vault watertight doors are similar to the requirements described in Section 3.4.3.1, except that only the site-specific loads are considered, as described in Section 3H.6.~~7~~4.

RAI 03.04.02-11, Revision 1

QUESTION:

With STP letter U7-C-STP_NRC-100165, dated July 12, 2010, Attachment 1, the applicant responded to **RAI 03.04.02-9,** stating that:

"Waves generated based on the provisions of the reference given in Standard Review Plan (SRP) Section 3.4.2.11(3) are discussed in FSAR Section 2.4S.3.6, which refers to FSAR Section 2.4S.4.3.1,which concludes that the maximum flood level, including the maximum wave run-up, would be El. 34.4 ft MSL. Table 2.4S.4-8 presents the water levels due to dam break, wind set-up and wave run-up at STP 3 & 4 for the critical fetch. The dynamic load effects due to wave run-up splash of 0.4 ft above plant grade level would be negligible in comparison to out-of-plane design basis loads such as tornado wind pressure for seismic Category I structures. The methodology given by the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, was adopted to estimate the wave height and wave run-up at STP 3 & 4 power block. The procedures outlined in the CEM use the wind speed, wind duration, water depth, and over-water fetch distance, and the run-up slope surface characteristics as input. Reference 2.4S.4-13 is the "Coastal Engineering Manual," U.S. Army Corps of Engineers, June 2006, which is a later version of the reference given in SRP Section 3.4.11 (3). As discussed in COLA Section 2.4S.4.2.2.4.3 and in response to RAI 03.04.02-1, the 44 pounds per square foot hydrodynamic drag force is due to velocity of the Main Cooling Reservoir breach flood flow."

During its evaluation the staff noted that the applicant's response refers to the wave action associated with the postulated river dam breaks located upstream of the Units 3 & 4-site. These events are calculated to result in a maximum flood elevation (including wave action) of 34.4ft MSL, thus only 0.40ft above nominal finished plant grade set at 34.0 ft MSL. The staff agrees that the resulting hydrodynamic and wave loads from those events are not significant. The governing flood event is however the assumed breach of the Main Cooling Reservoir which leads to a calculated flood elevation of 38.8ft MSL or nominal DBFL of 40.0ft MSL. As stated in its response, the fluid analysis has determined a flow velocity of 4.72 fps with an associated hydrodynamic surcharge fluid pressure of 44 psf. For DBFL above finished grade, SRP Section 3.4.2.II(3) requires consideration of wave load effects in the design of Seismic Category I SSC.

In its response the applicant has not evaluated the effect of water waves that may propagate on the water surface of the governing flood event. In its response to RAI 03.04.02-1 (RAI 3322 Question 13161), the applicant also referred to responses to four other RAIs (RAI 03.08.01-4, RAI 03.04.02-2, RAI 03.04.02-4, and RAI 03.04.02-5) for the resolution of RAI 03.04.02-1. The applicant is therefore requested to evaluate the effect of water waves that may propagate on the water surface of the governing flood event, and to track the closure status of the above noted four RAIs. The staff needs this information in order to be able to conclude that the above defined DBF effects are adequately accounted for in the design of Seismic Category I SSC pursuant to SRP Section 3.4.2.II(3).

REVISED RESPONSE:

The original response of RAI 03.04.02-11 was submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010. This revised response completely supersedes that response. Changes to the original RAI response are marked with a revision bar in the margin. This revision changes the flood water density from 62.4 pcf to 63.85 pcf to include the weight of sediment, based on discussions in the NRC audit held during the week of October 18, 2010.

Coincidental hydrodynamic wind wave forces were not considered with the conservative Main Cooling Reservoir (MCR) breach flood level because of the short duration of this flood. In addition, the relevant NRC and industry guidance provide for the consideration of wind-generated waves for design flood level and their effects on safety-related structures only for potential flooding due to hydrologic causes, such as Probable Maximum Precipitation, and does not provide for consideration of wind-generated waves coincident with a non-hydrologic failure, such as the postulated breach of the MCR dike breach.

To respond to this RAI, however, a 2-year fastest mile wind speed of 50 mph, based on COLA Reference 2.4S.4-7, is conservatively applied coincident with the MCR breach flood level to determine the hydrodynamic load due to the wind generated waves. The methodology given in the Coastal Engineering Manual (CEM), COLA Reference 2.4S.4-13 is used to estimate the wave height and wave forces on the vertical walls of the STP 3 & 4 power block buildings.

1. Hydrodynamic Wind Wave Forces on the Safety-Related Structures:

Based on the site layout and considering the sheltering effect of other buildings or structures on the site, the controlling fetch length will be due to the westerly winds. Therefore, the longest fetch on the west facing Unit 4 safety-related structures is determined. For this governing condition, the wave height is calculated for the above wind speed, fetch and the depth of water along the fetch. Based on this, a significant non-breaking wave with a wave height (H_s) of 1.25 feet and a period (T) of 1.7 seconds would be generated. Considering a 1% wave height (H_1= 1.67 H_s) of 2.1 feet, per COLA Reference 2.4S.4-7, the wave force due to the wind generated waves is calculated and conservatively applied to all the safety-related structures including those for Unit 3.

The resultant hydrodynamic wave force is calculated to be 603 pounds (0.6 kips) per foot length of the vertical wall corresponding to the maximum breach flood level of 38.8 feet. The wave force diagram is shown in Figure 3.4-1, included with the COLA mark-up at the end of this response.

As seen from Figure 3.4-1, the total hydrostatic and hydrodynamic pressure at grade elevation 34'-0" is 339 psf (i.e. considering a sediment-laden water density of 63.85 lb/ft^3, 306.5 psf + 32.5 psf = 339 psf). This pressure is less than the hydrostatic pressure due to conservatively established design basis flood level of 40'-0" (i.e. 6 x 63.85 = 383 psf). Therefore, inclusion of wind generated waves does not affect design of below grade walls.

2. Maximum Water Level due to Wind-Generated Waves near the Safety-Related Structures:

Due to the waves generated by the postulated wind the water level near the safety-related structures will fluctuate above and below the still water level caused by the MCR dike breach flood. As stated in Item 1 above, the water levels near the Unit 4 safety-related structures are affected more than the water levels near the Unit 3 structures due to the controlling westerly winds. Therefore, the rise in water level due to wind wave effect near Unit 4 safety-related structures is considered as the upper bound water level fluctuation for the Unit 3 structures also.

Following are the maximum water levels near Unit 4 safety-related structures due to MCR dike breach flood and the fluctuation of the water level due to the wind waves.

- Maximum water level due to MCR breach flood near the Unit-4 Ultimate Heat Sink (UHS) = 38.8 feet
- Maximum water level due to MCR breach flood near the Unit-4 power block structures = 38.2 feet.
- Maximum periodic rise in water level due to wind wave action = 3.1 feet (see Figure 3.4-1)

Including the fluctuation in water level due to wind wave effect;

- The maximum water level near the Unit-4 UHS = 38.8 + 3.1 = 41.9 feet.
- The maximum water level near the Unit-4 power block structures = 38.2 + 3.1 = 41.3 feet.

The UHS and Reactor Service Water (RSW) Pump Houses are designed to be watertight below 50 feet MSL. All the power block safety-related structures are watertight below elevation 41.0 feet MSL due to one foot threshold provided above the design basis flood level of 40 feet MSL. Any periodic splash flooding above the 41-foot elevation up to the wave run-up elevation of 41.3 feet MSL will be minor and would be taken care of with normal housekeeping and will not affect the safety-related function of the structures.

Consistent with Standard Review Plan Section 3.4.2 requirements, and considering the above, the following criteria will be applied for the design of the safety-related structures:

a) Flotation stability evaluations shall be based on the buoyancy calculations using the conservatively established design basis flood level of 40'-0" MSL.

b) The lateral loads on the structural walls and overturning moment on the structure will include the effect of the wave-generated hydrodynamic forces, as discussed in Item 1 above and floating debris (see the Revision 1 response to RAI 03.08.01-10 which is being submitted concurrently with this response). As such, external walls of the structures shall be capable of resisting the following loads:

- Hydrostatic force considering a conservatively established design basis flood level of 40'-0" MSL.
- Hydrodynamic drag force of 44 psf due to flood water flow, applicable to above grade portion.
- Wind generated wave forces as shown in Figure 3.4-1, applicable to above grade portion.
- Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec.

c) Watertight seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL.

Application of the above criteria will impact previously submitted responses to the following RAIs and COLA sections:

- COLA Section 2.4S
- COLA Section 3.4
- RAI 03.04.02-1
- RAI 03.04.02-2
- RAI 03.04.02-4
- RAI 03.04.02-5
- RAI 03.08.01-4
- RAI 03.08.01-7
- RAI 03.08.04-18
- RAI 03.08.04-22

Impact on COLA Sections 2.4S and 3.4:

See COLA Revision 4 changes provided at the end of this response.

Impact on RAI 03.04.02-1:

See revised response to RAI 03.04.02-1 Revision 1 submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010.

Impact on RAI 03.04.02-2:

See revised response to follow up RAI 03.04.02-6 Revision 2 being provided concurrently with this response.

Impact on RAI 03.04.02-4:

See response to RAI 03.04.02-10, submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010.

Impact on RAI 03.04.02-5:

As noted in the criteria provided under Item 2C above, the seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL. This criterion is same as that previously used in the response to RAI 03.04.02-5. Therefore, there is no change in the response to RAI 03.04.02-5.

Impact on RAI 03.08.01-4

See Revision 2 response to RAI 03.08.01-4 being provided concurrently with this response.

Impact on RAI 03.08.01-7

See Revision 2 response to RAI 03.08.01-7 being provided concurrently with this response.

Impact on RAI 03.08.04-18

The flood loading including the hydrodynamic forces due to flood water flow and wind generated waves is bounded by the seismic loading considered in the design of the Radwaste Building. The exterior, above grade walls of the Radwaste Building are 3 ft thick, spanning nearly 60 ft (i.e. from elevation 35 ft to roof elevation of approximately 95 ft) which have been qualified/designed for seismic II/I requirement, considering an earthquake input that envelops 0.3g Regulatory Guide 1.60 response spectrum and the induced acceleration response spectrum due to site-specific Safe Shutdown Earthquake.

For COLA changes due to this response, see Supplement 1 for RAI 03.08.04-18 Revision 1, submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010.

Impact on RAI 03.08.04-22

The design of the RSW Piping Tunnels and UHS/RSW Pump House for flood loading, including the hydrodynamic forces due to flood water flow and wind generated waves, is bounded by the existing design for the following reasons:

- The only portions of the RSW Piping Tunnels which are located above grade are the access shafts which have 3 ft thick walls with minimum reinforcement of #10 at 12 inch spacing (except where wall reinforcement is #8 at 12 inch spacing for a 4 ft clear span). The maximum span for these access shaft walls is 30 ft. Assuming a maximum uniform load of 0.4 k/ft (which exceeds the maximum flood load at grade level) and a maximum span of 30 ft, the maximum induced shear and moment will be 6 k/ft and 45 k-ft/ft, respectively. The shear and moment capacity of a 3 ft thick wall with #10 bars at 12 inch spacing will far exceed the shear and moment due to these loads.

- Exterior walls of the UHS/RSW Pump House subject to flooding loads are 6-foot-thick reinforced concrete walls with a minimum reinforcement of #11 at 12 inch spacing. Design of these walls is governed by loadings other than flood loading because the induced shears and moments due to flood loading will be far less than the minimum shear and moment capacity of these walls. It should also be noted that the flood forces acting on the 6-foot-thick UHS basin exterior walls will oppose the hydrostatic load due to water within the basin.

For COLA changes due to this response, see RAI 03.08.04-22 Revision 1, submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010.

The COLA Revision 4 will be revised as follows as a result of this response.

1. Revise Section 2.4S as follows:

2.4S.4.2.2.4.3 Hydrodynamic Forces

The maximum water levels and velocities obtained near Units 3 and 4 were used to assess the hydrodynamic loadings on the plant buildings. Figures 2.4S.4-21(g) and 2.4S.4-21(h) show the time-dependent plots of the velocities at this location during the east and west breach scenarios, respectively. The peak velocities observed were 4.72 and 4.68 feet per second for the east and west breach scenarios, respectively. Figures 2.4S.4-21(g) and 2.4S.4-21(h) also show the sediment concentrations predicted by the SED2D model. The sediment-laden water density was used for hydrodynamic load calculations. The figures show that the sediment concentrations at the time and location of peak velocities would be 16.5 kg/m^3 and 15 kg/m^3 for the east and west breach scenarios, respectively. However, Figure 2.4S.4-21(g) shows a maximum concentration of 23 kg/m^3 occurring at approximately T = 1.3 hours. Conservatively, the maximum sediment concentration was used in conjunction with the maximum velocity to determine the hydrodynamic loads on the STP 3 and 4 plant facilities. Selecting a 23 kg/m^3 sediment concentration, a water density of 1023 kg/m^3 or 63.85 lb/ft^3 was used for load calculations. The maximum hydrostatic force on any plant building would be due to the depth of floodwater at the maximum water level. Hydrodynamic loads were calculated using the drag force formula with a drag coefficient conservatively set to 2.0, as presented below:

$$\text{Force (lb/ft}^2\text{)} = 2.0 \times \text{Density (lb/ft}^3\text{)} \times \text{Velocity}^2\ (\text{ft}^2/\text{sec}^2) / 2g$$

The maximum drag force due to the maximum velocity of flow near the plant buildings is estimated as 44 pounds per square foot of the projected submerged area of the buildings.

The hydrodynamic loads due to wind-generated waves have also been calculated. A two year fastest mile wind speed of 50 mph, based on Reference 2.4S.4-7, is conservatively applied coincident with the Main Cooling Reservoir (MCR) breach flood level. The methodology given in the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, is used to estimate the wave height and wave forces on the vertical walls of the power block buildings.

Based on the site layout and considering the sheltering effect of other buildings or structures on the site, the controlling fetch length will be due to the westerly winds. Therefore, the longest fetch on the west facing Unit 4 safety-related structures is determined. For this governing condition, the wave height is calculated for the above wind speed, fetch and the depth of water along the fetch. Based on this, a significant non-breaking wave with a wave height (H_s) of 1.25 feet and a period (T) of 1.7 seconds would be generated. Considering a 1% wave height (H_1 = 1.67 H_s) of 2.1 feet, per Reference 2.4S.4-7, the wave force due to the wind generated waves is calculated and conservatively applied to all the safety-related structures including those for Unit 3.

The resultant hydrodynamic wave force is calculated to be 603 pounds (0.6 kips) per foot length of the vertical wall corresponding to the maximum breach flood level of 38.8 feet. The wave force diagram is shown in Figure 3.4-1.

Due to the waves generated by the postulated wind the water level near the safety-related structures will fluctuate above and below the still water level caused by the MCR dike breach flood. As stated above, the water levels near the Unit 4 safety-related structures are affected more than the water levels near the Unit 3 structures due to the controlling westerly winds. Therefore, the rise in water level due to wind wave effect near Unit 4 safety-related structures is considered as the upper bound water level fluctuation for the Unit 3 structures also.

Following are the maximum water levels near Unit 4 safety-related structures due to MCR dike breach flood and the fluctuation of the water level due to the wind waves. The MCR dike breach flood levels are described in Section 2.4S.4.

- Maximum water level due to MCR breach flood near the Unit-4 Ultimate Heat Sink (UHS) = 38.8 feet
- Maximum water level due to MCR breach flood near the Unit-4 power block structures = 38.2 feet.
- Maximum periodic rise in water level due to wind wave action = 3.1 feet (see Figure 3.4-1)

Including the fluctuation in water level due to wind wave effect:

- The maximum water level near the Unit-4 UHS = 38.8 + 3.1 = 41.9 feet.
- The maximum water level near the Unit-4 power block structures = 38.2 + 3.1 = 41.3 feet.

The UHS and Reactor Service Water (RSW) Pump Houses are designed to be watertight below 50 feet MSL. All the power block safety-related structures are watertight below elevation 41.0 feet MSL due to one foot threshold provided above the design basis flood level of 40 feet MSL. Any periodic splash flooding above the 41-foot elevation up to the wave run-up elevation of 41.3 feet MSL will be minor and would be taken care of with normal housekeeping and will not affect the safety-related function of the structures.

2. Revise Section 3.4 as follows:

3.4.2 Analytical and Test Procedures

STP DEP T1 5.0-1

Since the design basis flood elevation is at El. 40.0 ft (see Subsection 2.4S.2.2), 182.9 cm above the finished plant grade, the lateral hydrostatic and hydrodynamic pressure on the structures due to the design flood water level, as well as ground and soil pressures, are calculated.

As discussed in Section 2.4S.4.2.2.4.3, the hydrodynamic force due to the wind-generated wave action on building walls has been calculated as shown in Figure 3.4-1.

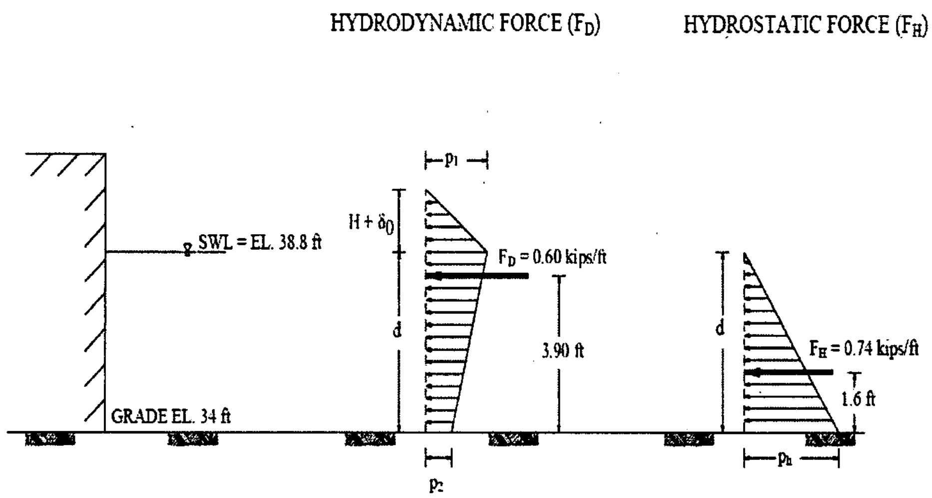
Consistent with Standard Review Plan Section 3.4.2 requirements, and the discussion provided in Section 2.4S.4.2.2.4.3, the following criteria will be applied for the design of the safety-related structures:

a) Flotation stability evaluations shall be based on the buoyancy calculations using the conservatively established design basis flood level of 40'-0" MSL.

b) The lateral loads on the structural walls and overturning moment on the structure will include the effect of the wave-generated hydrodynamic forces. As such, external walls of the structures shall be capable of resisting the following loads:

- Hydrostatic force considering a conservatively established design basis flood level of 40'-0" MSL.
- Hydrodynamic drag force of 44 psf due to flood water flow, applicable to above grade portion.
- Wind generated wave forces as shown in Figure 3.4-1, applicable to above grade portion.
- Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec.

c) Watertight seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL.



F_D = Resultant hydrodynamic force (in kips per linear foot) equivalent to the hydrodynamic wave pressure (p_1 and p_2) acting on the structure
F_H = Resultant hydrostatic force (in kips per linear foot) equivalent to the hydrostatic pressure (p_h) acting on the structure
H = Non-Breaking Wave Height(1%) = 2.1 ft
T = Non-Breaking Wave Period = 1.7 sec
d = depth of water at the structure = 4.8 ft
δ_0 = vertical shift of the wave crest and trough at the structure = 1.0 ft

p_1 = hydrodynamic wave pressure at the still water level = 132.8 lb/ft^2
p_2 = hydrodynamic wave pressure at the base of the structure = 32.5 lb/ft^2

p_h = hydrostatic pressure at the base of the structure = 306.5 lb/ft^2

Figure 3.4-1
Non-Breaking Wave Force on Vertical Wall

RAI 03.07.01-25, Supplement 1

QUESTION:

Follow-up Question to RAI 03.07.01-17 (STP-NRC-100035)

10CFR50 Appendix S requires that seismic evaluation must take into account soil-structure interaction (SSI) effects. STP has performed a site-specific SSI analysis to confirm that the ABWR DCD results envelop the results of the site-specific SSI analysis of the RB and CB. Regarding this reconciliation analysis the staff needs the following additional information to determine that site-specific SSI analysis adequately predicts the RB and CB seismic response:

1. In response to Item 1b of RAI 03.07.01-17, the applicant has provided comparison of the strain compatible shear wave velocity profiles for the backfill with those of the in-situ and DCD UB1D150 soil columns in Figure 3A-230a. Based on this comparison, the applicant has concluded that a separate confirmatory SSI analysis of the RB and CB incorporating backfill is not necessary because the lower and upper bound shear wave velocities of the backfill are enveloped by those of the in-situ soils and those used in DCD. Although this assertion is acceptable for the lower bound backfill properties, it has not been shown in Figure 3A-230a that the strain compatible DCD shear wave velocity profile envelop the upper bound backfill properties where the velocities exceed those of the in-situ upper bound profile and DCD UB1D150 at depths of approximately 12 to 52 feet below grade (see Figure 3A-230a). While the UB1D150 may be the lowest shear wave velocity case in the DCD, the applicant is requested to provide in the same Figure (3A-230a) comparison of the DCD upper bound strain compatible soil case that envelops the upper bound backfill properties.

2. In response to Item 1b of RAI 03.07.01-17 with respect to the strain-compatible damping properties, the applicant has provided comparison of the soil damping profiles for the backfill with those of the in-situ soil columns in Figure 3A-230b. Based on this comparison, the applicant has concluded that the backfill damping is generally higher than those of the in-situ soils, and thus bounded by the in-situ soil properties. A review of the results presented in Figure 3A-230b shows the lower-bound damping profile for the backfill to be significantly higher than that of the in-situ soils. Because the SSE design motion is specified at the free-field ground surface, a higher damping in the backfill material may result in a higher motion at the foundation level as compared with that obtained from the in-situ soil column with lower damping to compensate for the higher attenuation of the motion in the backfill soils. As such, the applicant is requested to provide further justification that the higher damping in the backfill material for the lower bound case will not result in foundation motions that exceed those of DCD.

3. In the response to Item 2 of RAI 03.07.01-17, the applicant has stated that the Poisson's ratio has been capped at 0.48 for saturated soils in calculating the compression wave velocity. This results in calculated compression wave velocities lower than 5000 ft/sec in saturated soils when the shear wave velocities drop below approximately 980 ft/sec. For example, as shown in Tables 3H.6-1b through 3H.6-2c (see the enclosure to STP's response to

RAI 03.07.02-17), approximately 57, 75 and 240 feet of the respective soil column of the in-situ upper bound, lower bound and mean soil cases have calculated P-wave velocities less than 5000 ft/sec. The use of compression wave velocities in saturated soils less than 5000 ft/sec will not allow the higher frequency components of the vertical motion to be transmitted into the structure and may result in less conservative response. As such, the applicant is requested to assess the impact of using P-wave velocities lower than 5000 ft/sec in saturated soils on the response of the structure including in-structure response spectra by performing a sensitivity study and comparing the results for two cases: Case 1 will cap Poisson's ratio at 0.48 for saturated soils and let P-wave velocity drop below 5000 ft/sec (similar to the procedure stated by the applicant) and Case 2 will set P-wave velocity to 5000 ft/sec in saturated soils and allow Poisson's ratio to rise above 0.48 depending on the strain-compatible shear wave velocities.

SUPPLEMENTAL RESPONSE:

The response to Parts 1 and 2 of this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. The response to Part 3 is provided in this supplemental response.

For saturated soil, the Poisson's ratio is capped at 0.48 to avoid any potential numerical instability that might be caused if a larger value is used in the soil-structure interaction (SSI) analysis using SASSI2000 computer program. To avoid any such instability, capping of Poisson's ratio (at 0.48) is a general industry practice. The following points should also be considered.

a. The vertical Ground Motion Response Spectrum at the STP site is calculated using the applicable vertical to horizontal (V/H) acceleration response spectral ratios, and not through site response analysis using P-wave velocity. Also, the free field deconvolved vertical motion at the foundation outcrop of the Reactor Building, Control Building and UHS/RSW Pump House , with the site specific Safe Shutdown Earthquake (SSE) specified at the ground surface, envelopes the Foundation Input Response Spectra (FIRS) by a wide margin (Refer to COLA Part 2, Tier 2 Figures 3A-235 and 3A-244).

   Therefore, the P-wave velocities calculated with Poisson's ratio capped at 0.48 do not adversely affect the calculated vertical SSE motion at the STP site. The only effect, if any, would occur in the SSI results.

b. The values of P-wave velocities below the water table were examined in Reference 1. In this study, which was part of the Lotung SSI Experiment, co-sponsored by the US NRC, the ground motion recordings at two downhole arrays were utilized. The study examined different cases where P-wave velocities used for vertical wave propagation are the ones calculated from geophysical measurements, or inferred from Fourier spectral ratio analyses. The study compared the results in each case to the recorded vertical motions.

Some of the conclusions of the study, pertinent to the P-wave velocity calculation are summarized below:

- P-wave velocities from geophysical measurements below the ground water table were lower than that of water, especially in the upper soil layers. This is attributed to the upper soil layers not being fully saturated. It is also indicated that the P-wave velocities of soils are strongly affected by the compressibility of soil and fluid components of the soil-fluid system.

- The study supports the use of decreased P-wave velocities for site response analysis of vertical excitation.

In light of the referenced study (Reference 1), the capping of Poison's ratio at 0.48, with resulting P-wave velocity of less than 5000 ft/sec in the upper soil layers is considered to be reasonable, and it represents the condition of soil layers that are not fully saturated below the ground water table. For SSI analysis, the requirement for variation of the soil properties stipulates a large variation in the shear modulus which in effect results in a large variation of shear and P-wave velocity values. The wide variation of the soil data, as stipulated by the SRP 3.7.2 requirement (minimum C_v of 0.5 on soil shear modulus), is intended to capture the variability and uncertainty associated with geotechnical data in SSI results. For example, P-wave velocity at the foundation elevation of the reactor building is 4150 ft/sec for the lower bound profile, and 5000 ft/sec for the mean and upper bound profiles. The results obtained from SSI analysis of the three soil profiles are enveloped for the design.

Sensitivity Study for Poisson's ratio Effect

Sensitivity study is performed to assess the effect of the Poisson's ratio limitation in the seismic SSI results. Control Building (CB) SSI model is used to perform this sensitivity study. SSI analysis results using Poisson's ratio limit of 0.495 are compared with the analyses results which used the Poisson's ratio limit of 0.48 (SSI analysis using Poisson's ratio limit of 0.48 is referred to as "original analysis" throughout this response). The CB structure model corresponds to the CB model described in Section 3A.19, and Figures 3A-265 and 3A-266 of the FSAR, with the exception that a two feet thick concrete mudmat is added under the base slab and the mesh size is refined such that the model is capable of passing frequencies up to 33 Hz in both the vertical and horizontal directions. Passing frequency is based on one-fifth wave length of soil layer shear wave velocity. Figures 03.07.01-25.1 and 03.07.01-25.2 schematically show the SSI models for the horizontal and vertical directions excitation, respectively.

The following provides the details of this sensitivity study.

In the original SSI analyses three site-specific strain compatible soil profiles were used, i.e. lower bound (LB), mean and upper bound (UB); and the response results from the three soil profiles were enveloped. For this sensitivity study two soil profiles were used. First, the LB soil profile was used, because the lower bound properties would be most affected by the increase in the Poisson's ratio limit. The other used the UB soil profile since the UB profile results, in

general, envelop the results of the LB and Mean soil profiles. In these analyses, the previous limit of Poisson's ratio was revised from 0.48 to 0.495. The modified LB and UB strain-compatible soil properties are shown in Tables 03.07.01-25.1 and 03.07.01-25.2, respectively. The original LB and UB soil properties, corresponding to the 0.48 Poisson's ratio limit, are shown in Tables 03.07.01-25.3 and 03.07.01-25.4, respectively.

The seismic responses from these analyses, in which the Poisson's ratio was capped at 0.495, were compared with the original seismic responses, in which the Poisson's ratio was capped at 0.48. The responses compared are (a) transfer functions, (b) total seismic forces, (c) maximum nodal accelerations and (d) response spectra. The comparisons are performed for the lower bound soil and the upper bound soil. The following summarizes the comparison of responses:

(a) Comparison of Transfer Functions

The sensitivity analyses used the same frequencies to calculate transfer functions as the original corresponding analyses, plus additional frequencies to obtain smoother transfer functions and to capture calculated peaks. Nodes 102, 105 and 108 (shown in Figure 03.07.01-25.1) were selected for the comparisons of the transfer functions. Figures 03.07.01-25.3, 4 and 5 show some of the transfer function comparisons for the LB soil profile analysis. Figures 03.07.01-25.6, 7 and 8 show some of the transfer function comparisons for UB soil profile analysis. The comparisons show that the number of peaks and the number of narrow peaks increases as compared to the transfer functions from the original analyses (with Poisson's ratio limit of 0.48). These transfer functions are intermediate results. The seismic responses of the CB, such as the in-structure response spectra, maximum accelerations, and element forces, as discussed in the following paragraphs, indicate that the analyses using a Poisson's limit of 0.495 produce relatively similar results to those using a Poisson's limit of 0.480. The largest increases in seismic responses are seen in results from the modified lower bound soil with vertical input motion, which is to be expected due to the larger increases in compression wave velocity. This shows that the use of Poisson's ratio limit of 0.495 did not cause an instability problem in the SSI analysis.

(b) Comparison of Seismic Forces and Moments

The total shear force, moment and axial force are compared in Tables 03.07.01-25.5, 6 and 7. The total forces and moments represent total story forces and moments (including the forces and moments in below grade walls).

Table 03.07.01-25.5 shows the total shear forces from the original analyses and modified analyses for the LB and UB soil cases. The envelope total shear forces are from the original analyses (envelope of LB, Mean and UB soil cases). The differences between the modified and original results for LB vary from about -2.5 % to 1.2 % (a positive difference represents modified result greater than the original result) and for UB from about -0.5% to 2.4 %. These differences are insignificant.

Table 03.07.01-25.6 shows the total moments from the original analyses and modified analyses for the LB and UB soil cases. The envelope total moments are from the original analyses (envelope of LB, Mean and UB soil cases). The differences between the modified and original results for LB vary from about -2.6 % to 10.8 %, and for UB from about -1.1% to 7.2 %. When the modified LB and modified UB results are compared to the original envelope results the differences reduce to a maximum of 7.7% and 5.1%, respectively. These differences are small.

Table 03.07.01-25.7 shows the total axial forces from the original analyses and modified analyses for the LB and UB soil cases. The envelope total axial forces are from the original analyses (envelope of LB, Mean and UB soil cases). The differences between the modified and original results for LB vary from about 19.5 % to 29.2 %. When the LB results are compared with the enveloped results, the maximum difference is 16.7 % for the top beam element and within 10% for other beam elements. The differences between the modified and original results for UB vary from about 1.8 % to 4.6 %. As expected, the LB vertical response is most affected by the Poisson's ratio limit.

(c) Comparison of Maximum Nodal Accelerations

Table 03.07.01-25.8 shows the maximum horizontal accelerations from the original analyses and modified analyses for the LB and UB soil cases. The envelope accelerations are from the original analyses (envelope of LB, Mean and UB soil cases). The differences between the modified and original results for LB vary from about -2.8 % to 1.7 % and for UB from about -2.9 % to 1.1 %. These differences are insignificant.

Table 03.07.01-25.9 shows the vertical accelerations from the original analyses and modified analyses for the LB and UB soil cases. The envelope accelerations are from the original analyses (envelope of LB, Mean and UB soil cases). The differences between the modified and original results for LB vary from about 2.0 % to 53.7 %. When the LB results are compared with the enveloped results, the maximum difference is 17.5 %. The differences between the modified and original results for UB vary from about -0.7 % to 14.4 %. Similar differences remain between the UB modified and envelope values.

(d) Comparison of In-Structure Response Spectra

Nodes 102, 105, 108, and 111 are selected as representatives for response spectra comparison. Node 102 is located at the base of the beam stick, Node 105 is approximately mid-height, node 108 is at the top of beam stick, and Node 111 is the slab oscillator at mid-height. 5% damped spectra calculated in the original analyses and in the modified analyses are compared.

Horizontal Response Spectra:

Horizontal spectra in the X (north-south), and Y (east-west) directions are compared in Figures 03.07.01-25.9 through 03.07.01-25.14. These comparisons show the following results:

- The LB modified spectra are about the same as the corresponding LB original spectra for frequencies lower than 6 Hz with small difference at higher frequencies. In Figures 03.07.01-25.9 through 03.07.01-25.14, these spectra are also compared with the corresponding original envelope spectra. The comparison shows that the envelope spectra, in general, envelop the LB modified spectra. The UB modified spectra are about the same as the corresponding UB original spectra in the entire frequency range, with only insignificant differences.

- In summary, the horizontal response spectra calculated using modified soil properties are enveloped by the original envelope spectra with the exception that some are greater than the envelope by less than about 0.02g at some frequencies, such as around 12.8 Hz in Figure 03.07.01-25.9, around 10.5 Hz in Figure 03.07.01-25.10 and around 13 Hz in Figure 03.07.01-25.14.

Vertical Response Spectra:

Vertical spectra (Z direction) for wall Nodes 102, 105 and 108 are compared in Figures 03.07.01-25.15 through 03.07.01-25.17. Figure 03.07.01-25.18 shows the comparison of the vertical spectra for floor Node 111. These comparisons show the following results:

- The LB modified vertical spectra, at wall Nodes 102, 105, and 108, are slightly higher than those from the original envelope spectra with a maximum increase of about 0.03g at some frequencies.

- The UB modified spectra at wall Nodes 102, 105, and 108, in general, are about the same as the corresponding UB original spectra with a maximum increase of about 0.07g above the UB original and envelope spectra at frequency of 22.5 Hz, at Node 108 (Figure 03.07.01-25.17).

- The modified vertical floor spectra are significantly higher than the envelope spectra at isolated frequencies. For example, at Node 111 the increase is about 0.16g at frequency of about 13 Hz (Figure 03.07.01-25.18).

(e) Comparison of Responses from the Modified Analysis with DCD Responses

Table 03.07.01-25.10 shows the comparison between the maximum floor accelerations from the modified analyses and the maximum floor accelerations provided in Table 3A-24 of the DCD. Table 03.07.01-25.11 shows the comparison between the maximum element shear and moments from the modified analyses and the maximum element shear and moments provided in Table 3A-20 of the DCD. Figures 03.07.01-25.19 through 03.07.01-25.25 show comparisons between broadened 5% damped response spectra from the modified analyses and the corresponding spectra from the DCD for Nodes 102, 105, 108 and 111 (for DCD spectra see DCD Figures 3A-210, 213, 216, 217, 220, 223, and 226). These comparisons show that similar margins remain between the DCD responses and the responses from the

modified analyses, with 0.495 as the Poisson's ratio cut-off, as they were when 0.48 was used as the Poisson's ratio cut-off in the original analysis.

Conclusion

Seismic responses from the modified analyses, in which the Poisson's ratio was capped at 0.495 for determining the P-wave velocities of soil layers below groundwater level, were compared with the original seismic responses, in which the Poisson's ratio was capped at 0.48. Based on the comparison of seismic responses provided above, it is concluded that the results obtained from Poisson's ratio capped at 0.495 are in general close to the corresponding enveloped responses obtained from the Poisson's ratio capped at 0.48, except for some of the responses in the vertical direction, especially for the vertical responses of the floor slabs. The following considerations apply to these exceedances.

- For the Control and Reactor Buildings, where the original site-specific SSI analyses used 0.48 as the Poisson's ratio cut-off, as described in Appendix 3A, it was shown that the DCD responses were higher than the site-specific responses. As discussed in Paragraph (e) above, even the modified responses, with 0.495 as the Poisson's ratio cut-off, show similar margins in comparison to the DCD responses. Therefore, the increases in vertical responses shown in this sensitivity study, as discussed above, are not significant to the conclusion that the DCD responses significantly envelop the site-specific responses for the Reactor and Control Buildings.

- For the new SSI analyses of the site-specific structures, a Poisson's ratio of 0.495 has been used. Therefore, the conclusions derived from the new analyses include the effect of higher Poisson's ratio cut-off.

Reference cited in this response:

1. Mok, C. M. Chang, C.-Y., and Legaspi, D. E. (1998) "Site Response Analyses of Vertical Excitation," Geotechnical Earthquake Engineering and Soil Dynamics III, Geotechnical Special Publication No 75, Proceedings of a Specialty Conference, Vol. 1, pp. 739-753, University of Washington, Seattle Washington, August 3-6, 1998.

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

Table 03.07.01-25.1: Modified Lower Bound Soil Properties

Layer No.	Layer Thickness [ft]	Top of Layer Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Poisson Ratio	Damping [%]
1	2.53	0.00	0.124	419.1	1128.4	0.420	1.7
2	2.62	-2.53	0.124	441.7	1189.3	0.420	1.8
3	2.70	-5.15	0.124	474.4	1277.4	0.420	1.9
4	2.80	-7.85	0.124	472.1	4744.5	0.495	2.1
5	2.80	-10.65	0.124	470.6	4729.5	0.495	2.2
6	2.80	-13.45	0.124	470.1	4724.4	0.495	2.3
7	2.80	-16.25	0.124	470.0	4723.4	0.495	2.3
8	2.80	-19.05	0.124	466.9	4692.3	0.495	2.4
9	2.80	-21.85	0.123	488.7	4911.4	0.495	2.5
10	3.24	-24.65	0.121	578.1	5000.0	0.493	2.9
11	3.08	-27.89	0.121	580.2	5000.0	0.493	3.0
12	3.08	-30.97	0.121	581.7	5000.0	0.493	3.0
13	3.08	-34.05	0.122	606.6	5000.0	0.493	2.2
14	3.08	-37.13	0.122	604.9	5000.0	0.493	2.2
15	3.08	-40.21	0.122	602.2	5000.0	0.493	2.3
16	3.13	-43.29	0.122	599.0	5000.0	0.493	2.4
17	3.30	-46.42	0.122	598.5	5000.0	0.493	2.5
18	3.30	-49.72	0.122	600.0	5000.0	0.493	2.5
19	3.30	-53.02	0.122	679.5	5000.0	0.491	2.3
20	3.30	-56.32	0.122	720.0	5000.0	0.489	2.3
21	3.30	-59.62	0.122	720.6	5000.0	0.489	2.3
22	3.35	-62.92	0.122	720.1	5000.0	0.489	2.3
23	2.91	-66.27	0.122	719.8	5000.0	0.489	2.3
24	2.91	-69.18	0.122	719.1	5000.0	0.489	2.3
25	2.91	-72.09	0.122	752.9	5000.0	0.488	2.2
26	1.12	-75.00	0.123	827.3	5000.0	0.486	2.1
27	2.00	-76.12	0.123	827.3	5000.0	0.486	2.1
28	2.94	-78.12	0.123	826.2	5000.0	0.486	2.1

Table 03.07.01-25.1: Modified Lower Bound Soil Properties (continued)

29	2.94	-81.06	0.123	825.7	5000.0	0.486	2.1
30	4.95	-84.00	0.123	824.2	5000.0	0.486	2.2
31	4.95	-88.95	0.123	822.8	5000.0	0.486	2.2
32	5.10	-93.90	0.125	849.7	5000.0	0.485	2.3
33	5.00	-99.00	0.125	849.9	5000.0	0.485	2.3
34	5.00	-104.00	0.125	849.5	5000.0	0.485	2.3
35	5.00	-109.00	0.125	874.5	5000.0	0.484	2.0
36	5.00	-114.00	0.125	873.3	5000.0	0.484	2.0
37	5.00	-119.00	0.125	872.1	5000.0	0.484	2.1
38	5.00	-124.00	0.125	914.5	5000.0	0.483	2.3
39	5.00	-129.00	0.125	914.2	5000.0	0.483	2.3
40	5.00	-134.00	0.125	913.5	5000.0	0.483	2.3
41	5.00	-139.00	0.125	911.5	5000.0	0.483	2.3
42	5.00	-144.00	0.125	911.1	5000.0	0.483	2.4
43	5.00	-149.00	0.125	910.7	5000.0	0.483	2.4
44	5.00	-154.00	0.125	910.4	5000.0	0.483	2.4
45	5.00	-159.00	0.125	883.7	5000.0	0.484	2.2
46	5.00	-164.00	0.125	881.5	5000.0	0.484	2.2
47	5.00	-169.00	0.125	880.6	5000.0	0.484	2.3
48	5.25	-174.00	0.125	919.6	5000.0	0.482	2.4
49	5.25	-179.25	0.125	919.5	5000.0	0.482	2.4
50	5.25	-184.50	0.125	919.1	5000.0	0.483	2.4
51	5.25	-189.75	0.125	921.1	5000.0	0.482	2.4
52	5.25	-195.00	0.125	922.5	5000.0	0.482	2.4
53	5.25	-200.25	0.125	922.8	5000.0	0.482	2.4
54	5.25	-205.50	0.125	922.3	5000.0	0.482	2.4
55	5.25	-210.75	0.125	919.2	5000.0	0.483	2.4
56	5.25	-216.00	0.125	920.2	5000.0	0.482	2.4
57	5.25	-221.25	0.124	921.5	5000.0	0.482	2.4
58	5.25	-226.50	0.124	928.5	5000.0	0.482	2.4
59	5.25	-231.75	0.124	931.4	5000.0	0.482	2.4
60	5.00	-237.00	0.127	986.2	5000.0	0.480	2.3
61	5.00	-242.00	0.127	985.7	5000.0	0.480	2.3
62	5.00	-247.00	0.127	985.1	5000.0	0.480	2.3
63	5.00	-252.00	0.127	984.6	5000.0	0.480	2.3
64	5.00	-257.00	0.127	984.0	5000.0	0.480	2.3
65	5.00	-262.00	0.125	1025.7	5000.0	0.478	2.3

Table 03.07.01-25.1: Modified Lower Bound Soil Properties (continued)

66	5.95	-267.00	0.127	1010.5	5000.0	0.479	2.1
67	5.95	-272.95	0.127	1010.5	5000.0	0.479	2.1
68	5.95	-278.90	0.125	1021.8	5000.0	0.478	2.2
69	5.95	-284.85	0.123	1034.4	5000.0	0.478	2.4
70	6.10	-290.80	0.123	1034.2	5000.0	0.478	2.4
71	6.10	-296.90	0.123	1034.0	5000.0	0.478	2.4
72	6.10	-303.00	0.123	1033.8	5000.0	0.478	2.4
73	6.10	-309.10	0.123	1033.7	5000.0	0.478	2.4
74	6.10	-315.20	0.123	1035.9	5000.0	0.478	2.4
75	6.10	-321.30	0.123	1037.2	5000.0	0.478	2.4
76	6.10	-327.40	0.123	1037.0	5000.0	0.478	2.4
77	6.10	-333.50	0.123	1036.9	5000.0	0.478	2.4
78	6.10	-339.60	0.127	1195.4	5201.0	0.472	2.0
79	6.10	-345.70	0.128	1252.4	5264.0	0.470	1.8
80	6.20	-351.80	0.128	1252.4	5264.0	0.470	1.8
81	7.59	-358.00	0.123	1301.7	5471.3	0.470	2.1
82	7.59	-365.59	0.128	1309.8	5505.3	0.470	1.8
83	7.69	-373.18	0.128	1310.3	5507.2	0.470	1.8
84	7.69	-380.87	0.128	1309.6	5504.6	0.470	1.8
85	7.69	-388.56	0.128	1309.5	5503.9	0.470	1.8
86	7.69	-396.25	0.125	1297.2	5452.4	0.470	2.1
87	7.30	-403.94	0.126	1208.9	5169.4	0.471	2.1
88	6.85	-411.24	0.128	1156.1	5000.0	0.472	2.1
89	6.85	-418.09	0.128	1156.1	5000.0	0.472	2.1
90	5.85	-424.94	0.124	1021.2	5000.0	0.478	2.4
91	5.85	-430.79	0.123	995.4	5000.0	0.479	2.5
92	5.85	-436.64	0.123	995.4	5000.0	0.479	2.5
93	5.85	-442.49	0.123	995.2	5000.0	0.479	2.5
94	5.85	-448.34	0.123	995.2	5000.0	0.479	2.5
95	5.85	-454.19	0.126	977.7	5000.0	0.480	2.6
96	5.85	-460.04	0.126	976.7	5000.0	0.480	2.6
97	5.85	-465.89	0.123	990.9	5000.0	0.480	2.5
98	5.85	-471.74	0.123	990.9	5000.0	0.480	2.5
99	5.85	-477.59	0.123	990.9	5000.0	0.480	2.5
Halfspace			0.123	990.6	5000.0	0.480	2.5

Table 03.07.01-25.2: Modified Upper Bound Soil Properties

Layer No.	Layer Thickness [ft]	Top of Layer Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Poisson Ratio	Damping [%]
1	2.53	0.00	0.124	677.2	1823.4	0.420	0.8
2	2.62	-2.53	0.124	701.4	1888.6	0.420	0.8
3	2.70	-5.15	0.124	735.0	1979.0	0.420	0.9
4	2.80	-7.85	0.124	732.4	5000.0	0.489	0.9
5	2.80	-10.65	0.124	730.5	5000.0	0.489	1.0
6	2.80	-13.45	0.124	733.2	5000.0	0.489	1.0
7	2.80	-16.25	0.124	733.8	5000.0	0.489	1.0
8	2.80	-19.05	0.124	732.8	5000.0	0.489	1.1
9	2.80	-21.85	0.123	764.8	5000.0	0.488	1.1
10	3.24	-24.65	0.121	894.1	5000.0	0.483	1.2
11	3.08	-27.89	0.121	896.7	5000.0	0.483	1.3
12	3.08	-30.97	0.121	899.0	5000.0	0.483	1.3
13	3.08	-34.05	0.122	953.1	5000.0	0.481	0.8
14	3.08	-37.13	0.122	951.4	5000.0	0.481	0.8
15	3.08	-40.21	0.122	948.7	5000.0	0.481	0.9
16	3.13	-43.29	0.122	945.4	5000.0	0.481	0.9
17	3.30	-46.42	0.122	944.7	5000.0	0.481	0.9
18	3.30	-49.72	0.122	945.4	5000.0	0.481	0.9
19	3.30	-53.02	0.122	1069.5	5000.0	0.476	1.0
20	3.30	-56.32	0.122	1132.4	5000.0	0.473	1.1
21	3.30	-59.62	0.122	1132.9	5000.0	0.473	1.1
22	3.35	-62.92	0.122	1132.3	5000.0	0.473	1.1
23	2.91	-66.27	0.122	1132.0	5000.0	0.473	1.1
24	2.91	-69.18	0.122	1131.2	5000.0	0.473	1.1
25	2.91	-72.09	0.122	1166.7	5072.2	0.472	1.0
26	1.12	-75.00	0.123	1241.0	5215.9	0.470	0.8
27	2.00	-76.12	0.123	1241.0	5215.9	0.470	0.8
28	2.94	-78.12	0.123	1239.3	5209.1	0.470	0.8
29	2.94	-81.06	0.123	1238.6	5206.1	0.470	0.8
30	4.95	-84.00	0.123	1236.3	5196.6	0.470	0.8
31	4.95	-88.95	0.123	1234.2	5187.4	0.470	0.9

Table 03.07.01-25.2: Modified Upper Bound Soil Properties (continued)

32	5.10	-93.90	0.125	1274.6	5357.3	0.470	1.1
33	5.00	-99.00	0.125	1274.8	5358.3	0.470	1.1
34	5.00	-104.00	0.125	1274.2	5355.8	0.470	1.1
35	5.00	-109.00	0.125	1329.1	5586.6	0.470	0.8
36	5.00	-114.00	0.125	1327.9	5581.2	0.470	0.8
37	5.00	-119.00	0.125	1326.6	5576.1	0.470	0.8
38	5.00	-124.00	0.125	1371.7	5765.6	0.470	1.1
39	5.00	-129.00	0.125	1371.3	5763.9	0.470	1.1
40	5.00	-134.00	0.125	1370.7	5761.2	0.470	1.1
41	5.00	-139.00	0.125	1369.1	5754.5	0.470	1.1
42	5.00	-144.00	0.125	1368.6	5752.4	0.470	1.1
43	5.00	-149.00	0.125	1368.2	5750.9	0.470	1.1
44	5.00	-154.00	0.125	1367.9	5749.4	0.470	1.1
45	5.00	-159.00	0.125	1350.1	5674.8	0.470	0.9
46	5.00	-164.00	0.125	1348.4	5667.5	0.470	0.9
47	5.00	-169.00	0.125	1347.4	5663.6	0.470	0.9
48	5.25	-174.00	0.125	1379.4	5797.7	0.470	1.1
49	5.25	-179.25	0.125	1379.2	5797.0	0.470	1.1
50	5.25	-184.50	0.125	1378.7	5795.0	0.470	1.1
51	5.25	-189.75	0.125	1381.6	5807.0	0.470	1.1
52	5.25	-195.00	0.125	1383.7	5816.1	0.470	1.1
53	5.25	-200.25	0.125	1384.2	5817.9	0.470	1.1
54	5.25	-205.50	0.125	1383.5	5815.0	0.470	1.1
55	5.25	-210.75	0.125	1378.8	5795.4	0.470	1.1
56	5.25	-216.00	0.125	1389.5	5840.3	0.470	1.2
57	5.25	-221.25	0.124	1404.0	5901.3	0.470	1.2
58	5.25	-226.50	0.124	1413.8	5942.6	0.470	1.2
59	5.25	-231.75	0.124	1417.8	5959.3	0.470	1.2
60	5.00	-237.00	0.127	1497.4	6293.7	0.470	0.8
61	5.00	-242.00	0.127	1496.7	6291.0	0.470	0.8
62	5.00	-247.00	0.127	1496.1	6288.4	0.470	0.8
63	5.00	-252.00	0.127	1495.5	6285.9	0.470	0.8
64	5.00	-257.00	0.127	1494.9	6283.4	0.470	0.8
65	5.00	-262.00	0.125	1538.6	6467.1	0.470	1.1
66	5.95	-267.00	0.127	1515.8	6371.2	0.470	0.9
67	5.95	-272.95	0.127	1515.8	6371.2	0.470	0.9
68	5.95	-278.90	0.125	1532.7	6442.4	0.470	1.0

Table 03.07.01-25.2: Modified Upper Bound Soil Properties (continued)

69	5.95	-284.85	0.123	1551.6	6521.6	0.470	1.1
70	6.10	-290.80	0.123	1551.3	6520.4	0.470	1.1
71	6.10	-296.90	0.123	1551.0	6519.3	0.470	1.1
72	6.10	-303.00	0.123	1550.7	6518.0	0.470	1.1
73	6.10	-309.10	0.123	1550.5	6517.1	0.470	1.2
74	6.10	-315.20	0.123	1553.9	6531.2	0.470	1.2
75	6.10	-321.30	0.123	1555.8	6539.1	0.470	1.2
76	6.10	-327.40	0.123	1555.4	6537.8	0.470	1.2
77	6.10	-333.50	0.123	1555.3	6537.2	0.470	1.2
78	6.10	-339.60	0.127	1828.3	7684.6	0.470	0.8
79	6.10	-345.70	0.128	1929.1	8108.5	0.470	0.7
80	6.20	-351.80	0.128	1929.1	8108.5	0.470	0.7
81	7.59	-358.00	0.123	1968.4	8273.7	0.470	1.1
82	7.59	-365.59	0.128	1965.5	8261.5	0.470	0.8
83	7.69	-373.18	0.128	1965.4	8260.8	0.470	0.7
84	7.69	-380.87	0.128	1964.4	8256.9	0.470	0.7
85	7.69	-388.56	0.128	1964.2	8255.8	0.470	0.7
86	7.69	-396.25	0.125	1945.8	8178.6	0.470	1.0
87	7.30	-403.94	0.126	1814.3	7625.8	0.470	1.0
88	6.85	-411.24	0.128	1735.7	7295.4	0.470	0.9
89	6.85	-418.09	0.128	1735.7	7295.4	0.470	0.9
90	5.85	-424.94	0.124	1531.9	6439.0	0.470	1.1
91	5.85	-430.79	0.123	1493.2	6276.0	0.470	1.2
92	5.85	-436.64	0.123	1493.1	6275.7	0.470	1.2
93	5.85	-442.49	0.123	1492.8	6274.7	0.470	1.2
94	5.85	-448.34	0.123	1492.8	6274.7	0.470	1.2
95	5.85	-454.19	0.126	1466.5	6164.1	0.470	1.1
96	5.85	-460.04	0.126	1465.0	6157.8	0.470	1.1
97	5.85	-465.89	0.123	1486.4	6247.5	0.470	1.2
98	5.85	-471.74	0.123	1486.4	6247.5	0.470	1.2
99	5.85	-477.59	0.123	1486.3	6247.1	0.470	1.2
Halfspace			0.123	1486.0	6245.8	0.470	1.2

Table 03.07.01-25.3: Original Lower Bound Soil Properties

Layer No.	Layer Thickness [ft]	Top of Layer Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Poisson Ratio	Damping [%]
1	2.53	0.00	0.124	419.1	1128.4	0.420	1.700
2	2.62	-2.53	0.124	441.7	1189.3	0.420	1.800
3	2.70	-5.15	0.124	474.4	1277.4	0.420	1.900
4	2.80	-7.85	0.124	472.1	1763.3	0.461	2.100
5	2.80	-10.65	0.124	470.6	2399.5	0.480	2.200
6	2.80	-13.45	0.124	470.1	2397.3	0.480	2.300
7	2.80	-16.25	0.124	470.0	2396.4	0.480	2.300
8	2.80	-19.05	0.124	466.9	2380.6	0.480	2.400
9	2.80	-21.85	0.123	488.7	2491.9	0.480	2.500
10	3.24	-24.65	0.121	578.1	2947.9	0.480	2.900
11	3.08	-27.89	0.121	580.2	2958.3	0.480	3.000
12	3.08	-30.97	0.121	581.7	2966.2	0.480	3.000
13	3.08	-34.05	0.122	606.6	3093.0	0.480	2.200
14	3.08	-37.13	0.122	604.9	3084.2	0.480	2.200
15	3.08	-40.21	0.122	602.2	3070.6	0.480	2.300
16	3.13	-43.29	0.122	599.0	3054.4	0.480	2.400
17	3.30	-46.42	0.122	598.5	3051.7	0.480	2.500
18	3.30	-49.72	0.122	600.0	3059.2	0.480	2.500
19	3.30	-53.02	0.122	679.5	3464.9	0.480	2.300
20	3.30	-56.32	0.122	720.0	3671.2	0.480	2.300
21	3.30	-59.62	0.122	720.6	3674.4	0.480	2.300
22	3.35	-62.92	0.122	720.1	3671.7	0.480	2.300
23	2.91	-66.27	0.122	719.8	3670.2	0.480	2.300
24	2.91	-69.18	0.122	719.1	3666.7	0.480	2.300
25	2.91	-72.09	0.122	752.9	3839.3	0.480	2.200
26	1.12	-75.00	0.123	827.3	4218.4	0.480	2.100
27	2.00	-76.12	0.123	827.3	4218.4	0.480	2.100
28	2.94	-78.12	0.123	826.2	4212.9	0.480	2.100
29	2.94	-81.06	0.123	825.7	4210.5	0.480	2.100
30	4.95	-84.00	0.123	824.2	4202.7	0.480	2.200
31	4.95	-88.95	0.123	822.8	4195.3	0.480	2.200
32	5.10	-93.90	0.125	849.7	4332.7	0.480	2.300
33	5.00	-99.00	0.125	849.9	4333.5	0.480	2.300

Table 03.07.01-25.3: Original Lower Bound Soil Properties (continued)

34	5.00	-104.00	0.125	849.5	4331.5	0.480	2.300
35	5.00	-109.00	0.125	874.5	4459.3	0.480	2.000
36	5.00	-114.00	0.125	873.3	4452.8	0.480	2.000
37	5.00	-119.00	0.125	872.1	4446.7	0.480	2.100
38	5.00	-124.00	0.125	914.5	4663.0	0.480	2.300
39	5.00	-129.00	0.125	914.2	4661.6	0.480	2.300
40	5.00	-134.00	0.125	913.5	4658.2	0.480	2.300
41	5.00	-139.00	0.125	911.5	4647.8	0.480	2.300
42	5.00	-144.00	0.125	911.1	4645.5	0.480	2.400
43	5.00	-149.00	0.125	910.7	4643.8	0.480	2.400
44	5.00	-154.00	0.125	910.4	4642.2	0.480	2.400
45	5.00	-159.00	0.125	883.7	4506.2	0.480	2.200
46	5.00	-164.00	0.125	881.5	4494.7	0.480	2.200
47	5.00	-169.00	0.125	880.6	4490.3	0.480	2.300
48	5.25	-174.00	0.125	919.6	4689.0	0.480	2.400
49	5.25	-179.25	0.125	919.5	4688.3	0.480	2.400
50	5.25	-184.50	0.125	919.1	4686.8	0.480	2.400
51	5.25	-189.75	0.125	921.1	4696.5	0.480	2.400
52	5.25	-195.00	0.125	922.5	4703.8	0.480	2.400
53	5.25	-200.25	0.125	922.8	4705.3	0.480	2.400
54	5.25	-205.50	0.125	922.3	4702.9	0.480	2.400
55	5.25	-210.75	0.125	919.2	4687.1	0.480	2.400
56	5.25	-216.00	0.125	920.2	4692.0	0.480	2.400
57	5.25	-221.25	0.124	921.5	4698.6	0.480	2.400
58	5.25	-226.50	0.124	928.5	4734.5	0.480	2.400
59	5.25	-231.75	0.124	931.4	4749.0	0.480	2.400
60	5.00	-237.00	0.127	986.2	5000.0	0.480	2.300
61	5.00	-242.00	0.127	985.7	5000.0	0.480	2.300
62	5.00	-247.00	0.127	985.1	5000.0	0.480	2.300
63	5.00	-252.00	0.127	984.6	5000.0	0.480	2.300
64	5.00	-257.00	0.127	984.0	5000.0	0.480	2.300
65	5.00	-262.00	0.125	1025.7	5000.0	0.478	2.300
66	5.95	-267.00	0.127	1010.5	5000.0	0.479	2.100

Table 03.07.01-25.3: Original Lower Bound Soil Properties (continued)

67	5.95	-272.95	0.127	1010.5	5000.0	0.479	2.100
68	5.95	-278.90	0.125	1021.8	5000.0	0.478	2.200
69	5.95	-284.85	0.123	1034.4	5000.0	0.478	2.400
70	6.10	-290.80	0.123	1034.2	5000.0	0.478	2.400
71	6.10	-296.90	0.123	1034.0	5000.0	0.478	2.400
72	6.10	-303.00	0.123	1033.8	5000.0	0.478	2.400
73	6.10	-309.10	0.123	1033.7	5000.0	0.478	2.400
74	6.10	-315.20	0.123	1035.9	5000.0	0.478	2.400
75	6.10	-321.30	0.123	1037.2	5000.0	0.478	2.400
76	6.10	-327.40	0.123	1037.0	5000.0	0.478	2.400
77	6.10	-333.50	0.123	1036.9	5000.0	0.478	2.400
78	6.10	-339.60	0.127	1195.4	5201.0	0.472	2.000
79	6.10	-345.70	0.128	1252.4	5264.0	0.470	1.800
80	6.20	-351.80	0.128	1252.4	5264.0	0.470	1.800
81	7.59	-358.00	0.123	1301.7	5471.3	0.470	2.100
82	7.59	-365.59	0.128	1309.8	5505.3	0.470	1.800
83	7.69	-373.18	0.128	1310.3	5507.2	0.470	1.800
84	7.69	-380.87	0.128	1309.6	5504.6	0.470	1.800
85	7.69	-388.56	0.128	1309.5	5503.9	0.470	1.800
86	7.69	-396.25	0.125	1297.2	5452.4	0.470	2.100
87	7.30	-403.94	0.126	1208.9	5169.4	0.471	2.100
88	6.85	-411.24	0.128	1156.1	5000.0	0.472	2.100
89	6.85	-418.09	0.128	1156.1	5000.0	0.472	2.100
90	5.85	-424.94	0.124	1021.2	5000.0	0.478	2.400
91	5.85	-430.79	0.123	995.4	5000.0	0.479	2.500
92	5.85	-436.64	0.123	995.4	5000.0	0.479	2.500
93	5.85	-442.49	0.123	995.2	5000.0	0.479	2.500
94	5.85	-448.34	0.123	995.2	5000.0	0.479	2.500
95	5.85	-454.19	0.126	977.7	4962.8	0.480	2.600
96	5.85	-460.04	0.126	976.7	4963.5	0.480	2.600
97	5.85	-465.89	0.123	990.9	5000.0	0.480	2.500
98	5.85	-471.74	0.123	990.9	5000.0	0.480	2.500
99	5.85	-477.59	0.123	990.9	5000.0	0.480	2.500
Halfspace			0.123	990.6	5000.0	0.480	2.500

Table 03.07.01-25.4: Original Upper Bound Soil Properties

Layer No.	Layer Thickness [ft]	Top of Layer Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Poisson Ratio	Damping [%]
1	2.53	0.00	0.124	677.2	1823.4	0.420	0.800
2	2.62	-2.53	0.124	701.4	1888.6	0.420	0.800
3	2.70	-5.15	0.124	735.0	1979.0	0.420	0.900
4	2.80	-7.85	0.124	732.4	2734.2	0.461	0.900
5	2.80	-10.65	0.124	730.5	3725.1	0.480	1.000
6	2.80	-13.45	0.124	733.2	3738.6	0.480	1.000
7	2.80	-16.25	0.124	733.8	3741.8	0.480	1.000
8	2.80	-19.05	0.124	732.8	3736.6	0.480	1.100
9	2.80	-21.85	0.123	764.8	3899.9	0.480	1.100
10	3.24	-24.65	0.121	894.1	4559.1	0.480	1.200
11	3.08	-27.89	0.121	896.7	4572.3	0.480	1.300
12	3.08	-30.97	0.121	899.0	4584.1	0.480	1.300
13	3.08	-34.05	0.122	953.1	4859.9	0.480	0.800
14	3.08	-37.13	0.122	951.4	4851.0	0.480	0.800
15	3.08	-40.21	0.122	948.7	4837.3	0.480	0.900
16	3.13	-43.29	0.122	945.4	4820.8	0.480	0.900
17	3.30	-46.42	0.122	944.7	4817.0	0.480	0.900
18	3.30	-49.72	0.122	945.4	4820.4	0.480	0.900
19	3.30	-53.02	0.122	1069.5	4945.3	0.475	1.000
20	3.30	-56.32	0.122	1132.4	5000.0	0.473	1.100
21	3.30	-59.62	0.122	1132.9	5000.0	0.473	1.100
22	3.35	-62.92	0.122	1132.3	5000.0	0.473	1.100
23	2.91	-66.27	0.122	1132.0	5000.0	0.473	1.100
24	2.91	-69.18	0.122	1131.2	5000.0	0.473	1.100
25	2.91	-72.09	0.122	1166.7	5072.2	0.472	1.000
26	1.12	-75.00	0.123	1241.0	5215.9	0.470	0.800
27	2.00	-76.12	0.123	1241.0	5215.9	0.470	0.800
28	2.94	-78.12	0.123	1239.3	5209.1	0.470	0.800
29	2.94	-81.06	0.123	1238.6	5206.1	0.470	0.800
30	4.95	-84.00	0.123	1236.3	5196.6	0.470	0.800
31	4.95	-88.95	0.123	1234.2	5187.4	0.470	0.900
32	5.10	-93.90	0.125	1274.6	5357.3	0.470	1.100
33	5.00	-99.00	0.125	1274.8	5358.3	0.470	1.100

Table 03.07.01-25.4: Original Upper Bound Soil Properties (continued)

34	5.00	-104.00	0.125	1274.2	5355.8	0.470	1.100
35	5.00	-109.00	0.125	1329.1	5586.6	0.470	0.800
36	5.00	-114.00	0.125	1327.9	5581.2	0.470	0.800
37	5.00	-119.00	0.125	1326.6	5576.1	0.470	0.800
38	5.00	-124.00	0.125	1371.7	5765.6	0.470	1.100
39	5.00	-129.00	0.125	1371.3	5763.9	0.470	1.100
40	5.00	-134.00	0.125	1370.7	5761.2	0.470	1.100
41	5.00	-139.00	0.125	1369.1	5754.5	0.470	1.100
42	5.00	-144.00	0.125	1368.6	5752.4	0.470	1.100
43	5.00	-149.00	0.125	1368.2	5750.9	0.470	1.100
44	5.00	-154.00	0.125	1367.9	5749.4	0.470	1.100
45	5.00	-159.00	0.125	1350.1	5674.8	0.470	0.900
46	5.00	-164.00	0.125	1348.4	5667.5	0.470	0.900
47	5.00	-169.00	0.125	1347.4	5663.6	0.470	0.900
48	5.25	-174.00	0.125	1379.4	5797.7	0.470	1.100
49	5.25	-179.25	0.125	1379.2	5797.0	0.470	1.100
50	5.25	-184.50	0.125	1378.7	5795.0	0.470	1.100
51	5.25	-189.75	0.125	1381.6	5807.0	0.470	1.100
52	5.25	-195.00	0.125	1383.7	5816.1	0.470	1.100
53	5.25	-200.25	0.125	1384.2	5817.9	0.470	1.100
54	5.25	-205.50	0.125	1383.5	5815.0	0.470	1.100
55	5.25	-210.75	0.125	1378.8	5795.4	0.470	1.100
56	5.25	-216.00	0.125	1389.5	5840.3	0.470	1.200
57	5.25	-221.25	0.124	1404.0	5901.3	0.470	1.200
58	5.25	-226.50	0.124	1413.8	5942.6	0.470	1.200
59	5.25	-231.75	0.124	1417.8	5959.3	0.470	1.200
60	5.00	-237.00	0.127	1497.4	6293.7	0.470	0.800
61	5.00	-242.00	0.127	1496.7	6291.0	0.470	0.800
62	5.00	-247.00	0.127	1496.1	6288.4	0.470	0.800
63	5.00	-252.00	0.127	1495.5	6285.9	0.470	0.800
64	5.00	-257.00	0.127	1494.9	6283.4	0.470	0.800
65	5.00	-262.00	0.125	1538.6	6467.1	0.470	1.100
66	5.95	-267.00	0.127	1515.8	6371.2	0.470	0.900

Table 03.07.01-25.4: Original Upper Bound Soil Properties (continued)

67	5.95	-272.95	0.127	1515.8	6371.2	0.470	0.900
68	5.95	-278.90	0.125	1532.7	6442.4	0.470	1.000
69	5.95	-284.85	0.123	1551.6	6521.6	0.470	1.100
70	6.10	-290.80	0.123	1551.3	6520.4	0.470	1.100
71	6.10	-296.90	0.123	1551.0	6519.3	0.470	1.100
72	6.10	-303.00	0.123	1550.7	6518.0	0.470	1.100
73	6.10	-309.10	0.123	1550.5	6517.1	0.470	1.200
74	6.10	-315.20	0.123	1553.9	6531.2	0.470	1.200
75	6.10	-321.30	0.123	1555.8	6539.1	0.470	1.200
76	6.10	-327.40	0.123	1555.4	6537.8	0.470	1.200
77	6.10	-333.50	0.123	1555.3	6537.2	0.470	1.200
78	6.10	-339.60	0.127	1828.3	7684.6	0.470	0.800
79	6.10	-345.70	0.128	1929.1	8108.5	0.470	0.700
80	6.20	-351.80	0.128	1929.1	8108.5	0.470	0.700
81	7.59	-358.00	0.123	1968.4	8273.7	0.470	1.100
82	7.59	-365.59	0.128	1965.5	8261.5	0.470	0.800
83	7.69	-373.18	0.128	1965.4	8260.8	0.470	0.700
84	7.69	-380.87	0.128	1964.4	8256.9	0.470	0.700
85	7.69	-388.56	0.128	1964.2	8255.8	0.470	0.700
86	7.69	-396.25	0.125	1945.8	8178.6	0.470	1.000
87	7.30	-403.94	0.126	1814.3	7625.8	0.470	1.000
88	6.85	-411.24	0.128	1735.7	7295.4	0.470	0.900
89	6.85	-418.09	0.128	1735.7	7295.4	0.470	0.900
90	5.85	-424.94	0.124	1531.9	6439.0	0.470	1.100
91	5.85	-430.79	0.123	1493.2	6276.0	0.470	1.200
92	5.85	-436.64	0.123	1493.1	6275.7	0.470	1.200
93	5.85	-442.49	0.123	1492.8	6274.7	0.470	1.200
94	5.85	-448.34	0.123	1492.8	6274.7	0.470	1.200
95	5.85	-454.19	0.126	1466.5	6164.1	0.470	1.100
96	5.85	-460.04	0.126	1465.0	6157.8	0.470	1.100
97	5.85	-465.89	0.123	1486.4	6247.5	0.470	1.200
98	5.85	-471.74	0.123	1486.4	6247.5	0.470	1.200
99	5.85	-477.59	0.123	1486.3	6247.1	0.470	1.200
Halfspace			0.123	1486.0	6245.8	0.470	1.200

Table 03.07.01-25.5: Comparison of Total Seismic Shear

Element No.	Node	Elev TMSL (ft)	Total Shear Envelope (k)	Total Shear LB (k)	Total Shear Modified LB (k)	Total Shear UB (k)	Total Shear Modified UB (k)	Difference Modified LB & LB[1] (%)	Difference Modified UB & UB[1] (%)	Difference Modified LB & Env[2] (%)	Difference Modified UB & Env[2] (%)
7	108	72.83	1448	1438	1402	1448	1461	-2.48%	0.91%	-3.15%	0.91%
	107	56.27	1448	1438	1402	1448	1461	-2.48%	0.91%	-3.15%	0.91%
6	107	56.27	3230	3114	3093	3230	3213	-0.67%	-0.52%	-4.24%	-0.52%
	106	40.35	3230	3114	3093	3230	3213	-0.67%	-0.52%	-4.24%	-0.52%
5	106	40.35	4516	4198	4140	4516	4579	-1.40%	1.41%	-8.33%	1.41%
	105	25.92	4973	4611	4549	4973	5040	-1.34%	1.34%	-8.53%	1.34%
4	105	25.92	6898	6694	6679	6898	7065	-0.23%	2.42%	-3.18%	2.42%
	104	11.48	7388	7099	7091	7388	7541	-0.12%	2.07%	-4.02%	2.07%
3	104	11.48	10437	10325	10447	10392	10594	1.19%	1.94%	0.10%	1.50%
	103	-7.05	11008	10810	10936	11008	11191	1.16%	1.66%	-0.65%	1.66%
2	103	-7.05	13578	13157	13209	13578	13726	0.39%	1.09%	-2.72%	1.09%
	102	-26.9	14208	13628	13684	14208	14348	0.41%	0.98%	-3.69%	0.98%

1. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and Original Lower Bound or Upper Bound Case with 0.48 Poisson Ratio Limit
2. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and the Original Envelope of all soil cases
3. The forces in this table were adjusted for the use of the 1/4 symmetry by multiplying by 4 (four).
The Forces listed are for the entire structure

Table 03.07.01-25.6: Comparison of Total Seismic Moment

Element No.	Node	Elev TMSL (ft)	Total Moment Envelope (k-ft)	Total Moment LB (k-ft)	Total Moment Modified LB (k-ft)	Total Moment UB (k-ft)	Total Moment Modified UB (k-ft)	Difference Modified LB & LB[1] (%)	Difference Modified UB & UB[1] (%)	Difference Modified LB & Env[2] (%)	Difference Modified UB & Env[2] (%)
7	108	72.83	10812	10400	11476	10596	11360	10.35%	7.21%	6.14%	5.07%
	107	56.27	29296	28216	28548	29296	29512	1.18%	0.74%	-2.55%	0.74%
6	107	56.27	37828	36764	40720	36600	36852	10.76%	0.69%	7.65%	-2.58%
	106	40.35	85680	83840	82280	85680	86400	-1.86%	0.84%	-3.97%	0.84%
5	106	40.35	99860	99860	97236	97792	96708	-2.63%	-1.11%	-2.63%	-3.16%
	105	25.92	154408	151862	148571	154408	155436	-2.17%	0.67%	-3.78%	0.67%
4	105	25.92	205088	201290	199299	205088	207652	-0.99%	1.25%	-2.82%	1.25%
	104	11.48	254513	253523	252921	253915	256528	-0.24%	1.03%	-0.63%	0.79%
3	104	11.48	343555	333471	331185	343555	348248	-0.69%	1.37%	-3.60%	1.37%
	103	-7.05	454017	451089	448941	450884	457043	-0.48%	1.37%	-1.12%	0.67%
2	103	-7.05	484608	476685	474169	484608	491435	-0.53%	1.41%	-2.15%	1.41%
	102	-26.9	729261	718431	718279	726849	736582	-0.02%	1.34%	-1.51%	1.00%

1. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and Original Lower Bound or Upper Bound Case with 0.48 Poisson Ratio Limit
2. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and the Original Envelope of all soil cases
3. The forces in this table were adjusted for the use of the 1/4 symmetry by multiplying by 4 (four).
The Forces listed are for the entire structure

Table 03.07.01-25.7: Comparison of Total Seismic Axial Force

Element No.	Node	Elev TMSL (ft)	Total Axial Envelope (k)	Total Axial LB (k)	Total Axial Modified LB (k)	Total Axial UB (k)	Total Axial Modified UB (k)	Difference Modified LB & LB[1] (%)	Difference Modified UB & UB[1] (%)	Difference Modified LB & Env[2] (%)	Difference Modified UB & Env[2] (%)
7	108	72.83	1374	1241	1604	1374	1399	29.23%	1.78%	16.71%	1.78%
	107	56.27	1374	1241	1604	1374	1399	29.23%	1.78%	16.71%	1.78%
6	107	56.27	3359	2983	3691	3359	3426	23.74%	1.98%	9.87%	1.98%
	106	40.35	3359	2983	3691	3359	3426	23.74%	1.98%	9.87%	1.98%
5	106	40.35	4665	4119	5074	4665	4853	23.19%	4.03%	8.77%	4.03%
	105	25.92	5106	4531	5581	5106	5309	23.17%	3.96%	9.29%	3.96%
4	105	25.92	5997	5366	6571	5997	6199	22.46%	3.36%	9.56%	3.36%
	104	11.48	6461	5797	7090	6461	6661	22.31%	3.10%	9.74%	3.10%
3	104	11.48	7577	6795	8219	7577	7821	20.96%	3.21%	8.47%	3.21%
	103	-7.05	8164	7336	8863	8164	8416	20.81%	3.08%	8.56%	3.08%
2	103	-7.05	9161	8284	9896	9161	9586	19.46%	4.63%	8.02%	4.63%
	102	-26.9	9777	8867	10553	9777	10215	19.01%	4.48%	7.94%	4.48%

1. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and Original Lower Bound or Upper Bound Case with 0.48 Poisson Ratio Limit
2. The difference is calculated as the difference between the Modified Lower Bound or Upper Bound Case with 0.495 Poisson Ratio Limit and the Original Envelope of all soil cases
3. The forces in this table were adjusted for the use of the 1/4 symmetry by multiplying by 4 (four). The Forces listed are for the entire structure

Table 03.07.01-25.8: Comparison of Maximum Stick Node Horizontal Accelerations

Node	Elev TMSL (ft)	Location	Max. Horizontal Acc.[3] Envelope (g)	Max. Horizontal Acc.[3] LB (g)	Max. Horizontal Acc.[3] Modified LB (g)	Max. Horizontal Acc.[3] UB (g)	Max. Horizontal Acc.[3] Modified UB (g)	Difference Modified LB & LB[1] (%)	Difference Modified UB & UB[1] (%)	Difference Modified LB & Env[2] (%)	Difference Modified UB & Env[2] (%)
108	72.83	C/B Stick	0.140	0.139	0.135	0.140	0.141	-2.80%	0.64%	-3.22%	0.64%
107	56.27	C/B Stick	0.132	0.127	0.126	0.132	0.133	-0.47%	1.06%	-4.10%	1.06%
106	40.35	C/B Stick	0.130	0.117	0.116	0.130	0.130	-0.77%	0.69%	-10.50%	0.69%
105	25.92	C/B Stick	0.129	0.107	0.109	0.129	0.125	1.59%	-2.79%	-15.81%	-2.79%
104	11.48	C/B Stick	0.126	0.100	0.100	0.126	0.123	0.60%	-2.93%	-20.65%	-2.93%
103	-7.05	C/B Stick	0.121	0.090	0.091	0.121	0.119	1.00%	-1.41%	-24.57%	-1.41%
102	-26.90	C/B Stick	0.115	0.103	0.105	0.115	0.116	1.65%	0.78%	-8.97%	0.78%

1. The difference is calculated as the difference between the Modified Lower or Upper Bound Case with 0.495 Poisson Ratio Limit and Original Lower or Upper Bound Case with 0.48 Poisson Ratio Limit, respectively
2. The difference is calculated as the difference between the Modified Lower or Upper Bound Case with 0.495 Poisson Ratio Limit and the Original Envelope of all soil cases
3. Horizontal accelerations are the envelopes of x and y direction responses.

Table 03.07.01-25.9: Comparison of Maximum Floor Vertical Accelerations

Node	Elev TMSL (ft)	Location	Max. Vertical Acc.[3] Envelope (g)	Max. Vertical Acc.[3] LB (g)	Max. Vertical Acc.[3] Modified LB (g)	Max. Vertical Acc.[3] UB (g)	Max. Vertical Acc.[3] Modified UB (g)	Difference Modified LB & LB[1] (%)	Difference Modified UB & UB[1] (%)	Difference Modified LB & Env[2] (%)	Difference Modified UB & Env[2] (%)
108	72.83	C/B Stick	0.143	0.134	0.168	0.143	0.151	24.99%	5.31%	17.22%	5.31%
107	56.27	C/B Stick	0.140	0.133	0.165	0.140	0.147	23.71%	4.82%	17.50%	4.82%
106	40.35	C/B Stick	0.136	0.131	0.160	0.136	0.141	21.55%	3.86%	17.33%	3.86%
105	25.92	C/B Stick	0.130	0.127	0.148	0.130	0.130	16.94%	-0.07%	14.17%	-0.07%
104	11.48	C/B Stick	0.128	0.124	0.143	0.128	0.129	15.29%	1.37%	11.59%	1.37%
103	-7.05	C/B Stick	0.125	0.123	0.136	0.125	0.127	10.52%	2.12%	9.17%	2.12%
102	-26.90	C/B Stick	0.124	0.124	0.126	0.121	0.120	2.02%	-0.66%	2.02%	-2.61%
113	56.27	C/B Floor	0.228	0.163	0.251	0.228	0.241	53.74%	5.71%	10.14%	5.71%
112	40.35	C/B Floor	0.201	0.144	0.220	0.201	0.224	53.27%	11.76%	9.72%	11.76%
111	25.92	C/B Floor	0.174	0.134	0.190	0.174	0.199	41.97%	14.40%	9.50%	14.40%
110	11.48	C/B Floor	0.157	0.133	0.167	0.157	0.175	25.55%	11.31%	5.84%	11.31%
109	-7.05	C/B Floor	0.163	0.136	0.144	0.163	0.162	5.28%	-0.67%	-11.96%	-0.67%

1. The difference is calculated as the difference between the Modified Lower or Upper Bound Case with 0.495 Poisson Ratio Limit and Original Lower or Upper Bound Case with 0.48 Poisson Ratio Limit, respectively
2. The difference is calculated as the difference between the Modified Lower or Upper Bound Case with 0.495 Poisson Ratio Limit and the Original Envelope of all soil cases
3. Vertical accelerations include effects due to rocking.

Table 03.07.01-25.10: Comparison of ABWR DCD* & Envelope of Analyses with Modified Soil Properties Maximum Floor Accelerations

Node	Elev TMSL (ft)	Location	DCD Max. Acceleration (g) Horizontal	DCD Max. Acceleration (g) Vertical	0.495 Envelope Max. Acceleration (g) Horizontal	0.495 Envelope Max. Acceleration (g) Vertical
108	22.2	C/B Stick	1.02	0.48	0.141	0.168
107	17.15	C/B Stick	0.72	0.44	0.133	0.165
106	12.3	C/B Stick	0.52	0.36	0.130	0.160
105	7.9	C/B Stick	0.34	0.33	0.125	0.148
104	3.5	C/B Stick	0.32	0.31	0.123	0.143
103	–2.15	C/B Stick	0.31	0.31	0.119	0.136
102	–8.20	C/B Stick	0.31	0.3	0.116	0.126
113	17.15	C/B Floor	–	0.62	–	0.251
112	12.3	C/B Floor	–	0.58	–	0.224
111	7.9	C/B Floor	–	0.55	–	0.199
110	3.5	C/B Floor	–	0.51	–	0.175
109	–2.15	C/B Floor	–	0.49	–	0.162

1. Horizontal accelerations are the envelopes of x and y direction responses.
2. 0.495 Envelope is the envelope of the horizotnal and vertical accelerations from analyses using modified lower and upper bound soil properties, provided in Table 8 and Table 9.
3. Vertical accelerations include effects due to rocking.

*Provided in Table 3A-24 of Design Control Document/Tier 2

Table 03.07.01-25.11: Comparison of ABWR DCD Enveloping Seismic Forces * & Envelope of Total Seismic Forces with Modified Soil Properties of Stick Elements Only**

Element No.	Node	Elev. TMSL (m)	DCD Max. Shear (t)	0.495 Envelope Max. Shear (t)	DCD Max. Moment (MN-m)	0.495 Envelope Max. Moment (MN-m)	DCD Max. Torsion (MN-m)	0.495 Envelope Max. Torsion (MN-m)
7	108	22.20	4000	663	50.01	15.56	117.68	18.20
	107	17.15	4000	663	254.98	40.01	117.68	18.20
6	107	17.15	8200	1457	333.44	55.21	225.56	40.01
	106	12.30	8200	1457	686.49	117.14	225.56	40.01
5	106	12.30	11000	2077	735.53	131.83	294.21	57.03
	105	7.90	11000	2286	1176.84	210.74	294.21	62.77
4	105	7.90	11000	3205	1176.84	281.54	294.21	87.99
	104	3.50	11000	3420	1667.19	347.81	294.21	93.92
3	104	3.50	11000	4805	1667.19	472.16	294.21	131.94
	103	-2.15	11000	5076	2255.61	619.67	294.21	139.38
2	103	-2.15	11000	6226	2255.61	666.30	294.21	170.95
	102	-8.20	11000	6508	2942.1	998.67	294.21	178.69

* Provided in Table 3A-20 of Design Control Document/Tier 2.
**The forces and moments have been adjusted for the use of quarter-symmetry by multiplying model results by 4 (four). Results shown are for full structure.

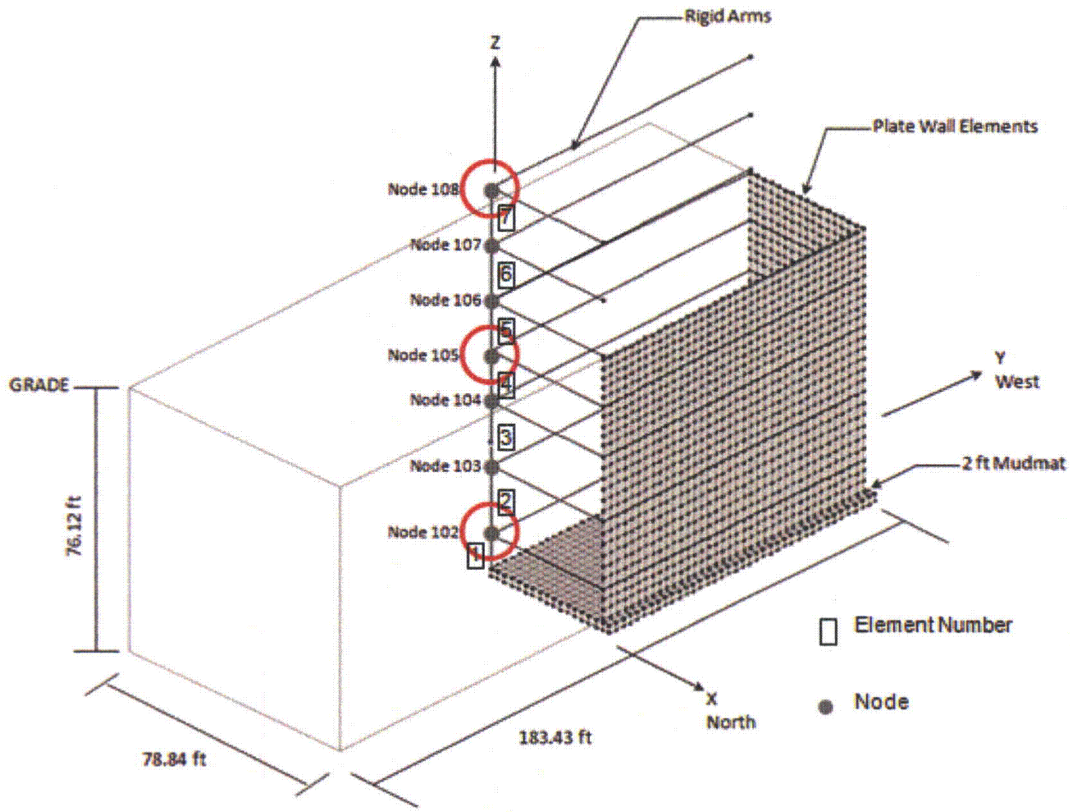


Figure 03.07.01-25.1: Control Building Model Schematic with Rigid Arms

Note: Nodes used in Poisson's ratio sensitive study for comparison of response spectra are circled in red.

Note: Beam element numbers are in square.

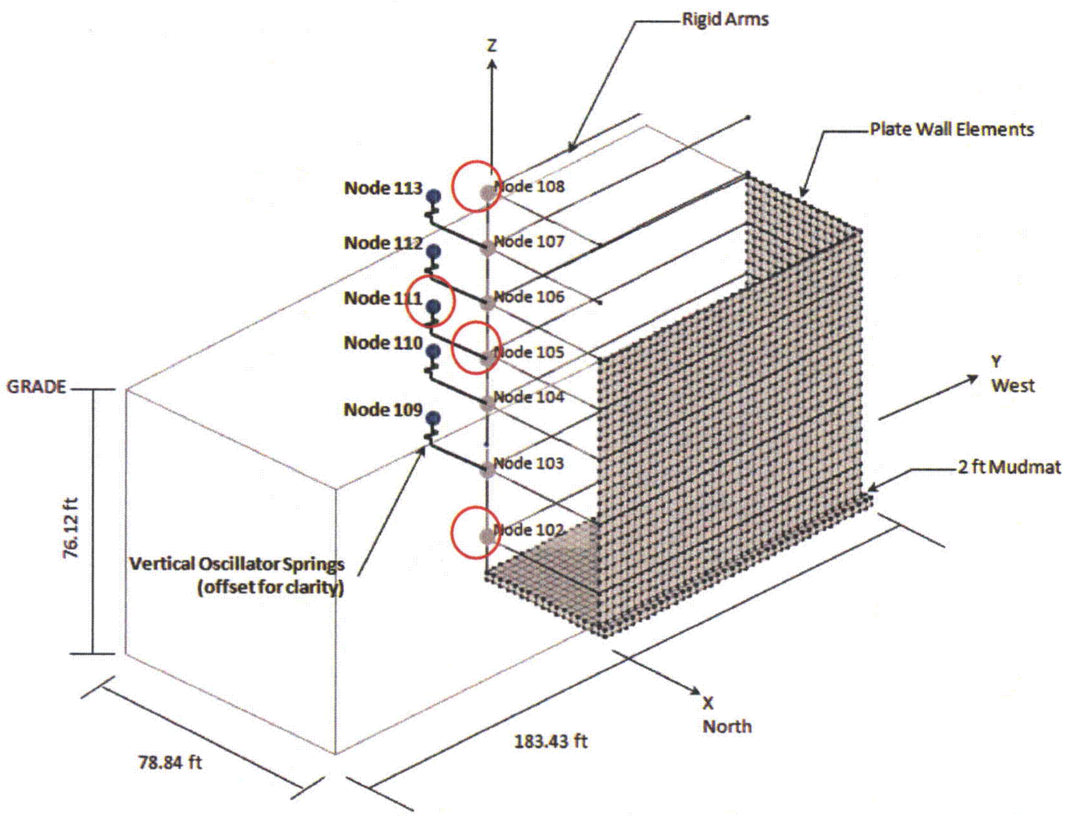


Figure 03.07.01-25.2: - Rigid Arms and Lumped Masses Connected by Springs

Note: Nodes used in Poisson's ratio sensitive study for comparison of response spectra are circled in red.

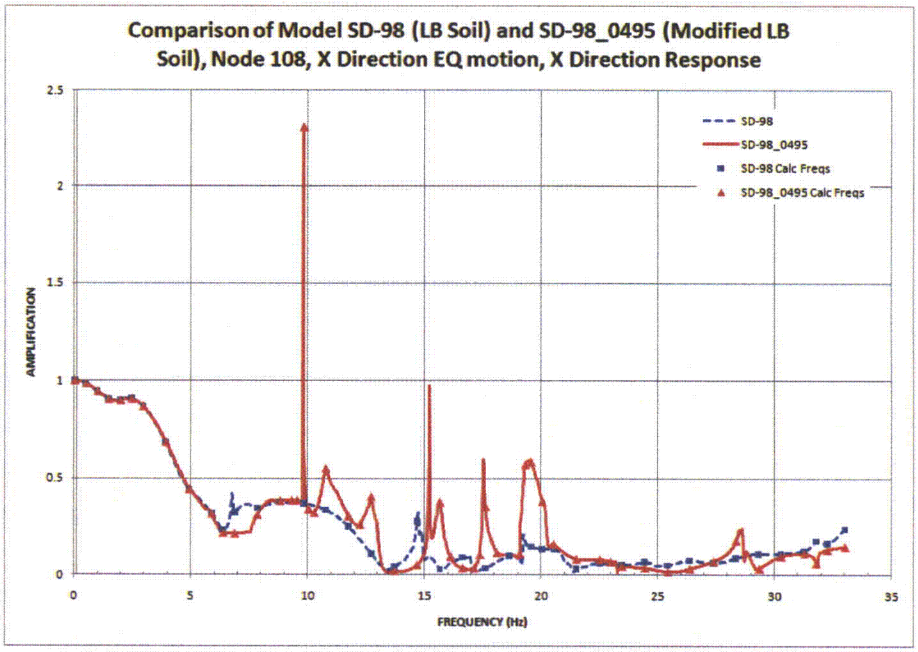


Figure 03.07.01-25.3: Comparison of Transfer Functions, Model SD-98 (LB Soil) and SD-98_0495 (Modified LB Soil), Node 108, X Direction EQ Motion, X Direction Response

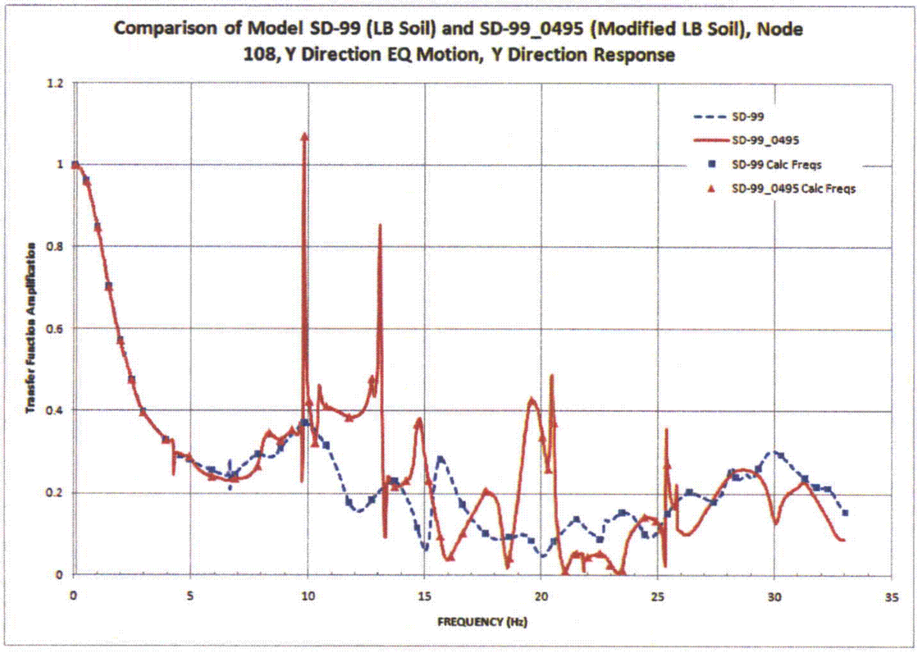


Figure 03.07.01-25.4: Comparison of Transfer Functions, Model SD-99 (LB Soil) and SD-99_0495 (Modified LB Soil), Node 108, Y Direction EQ Motion, Y Direction Response

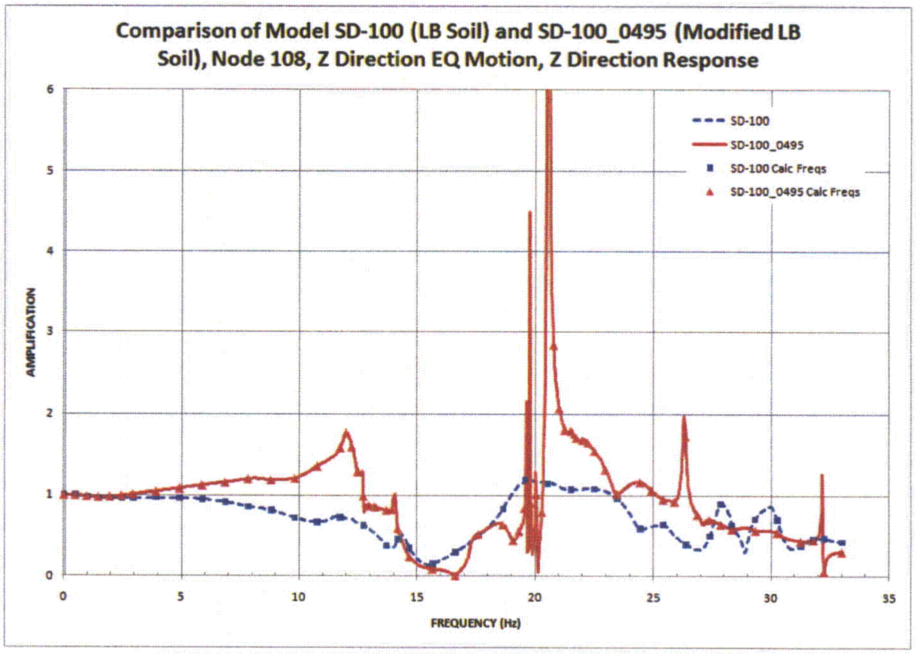


Figure 03.07.01-25.5: Comparison of Transfer Functions, Model SD-100 (LB Soil) and SD-100_0495 (Modified LB Soil), Node 108, Z Direction EQ Motion, Z Direction Response

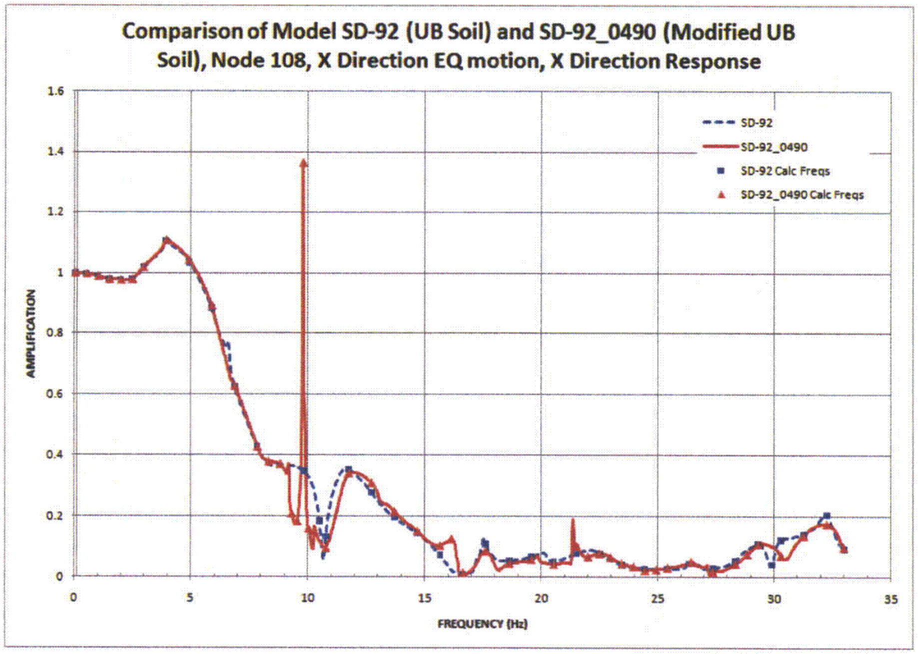


Figure 03.07.01-25.6: Comparison of Transfer Functions, Model SD-92 (UB Soil) and SD-92_0490 (Modified UB Soil), Node 108, X Direction EQ Motion, X Direction Response

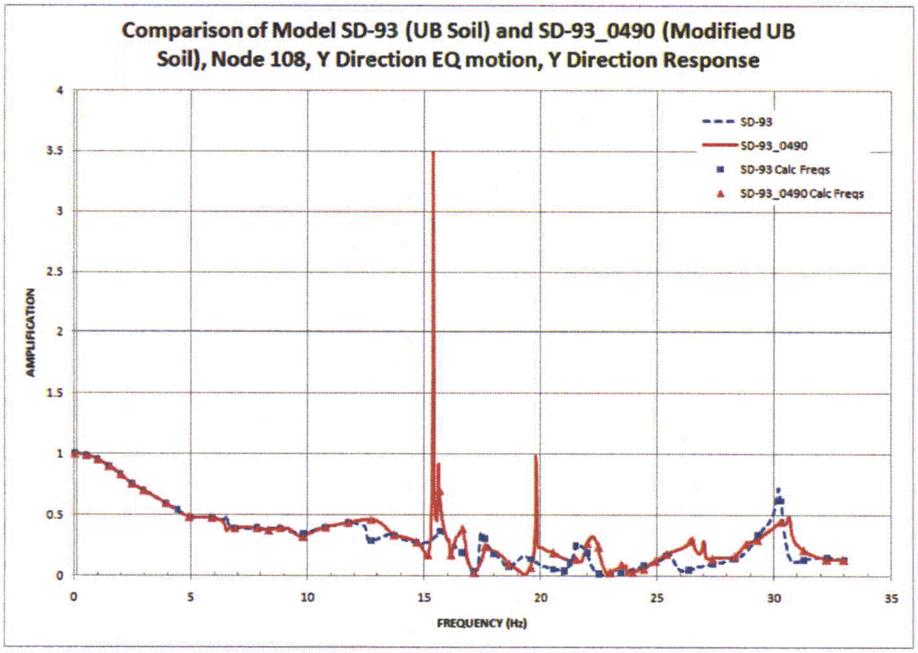


Figure 03.07.01-25.7: Comparison of Transfer Functions, Model SD-93 (UB Soil) and SD-93_0490 (Modified UB Soil), Node 108, Y Direction EQ Motion, Y Direction Response

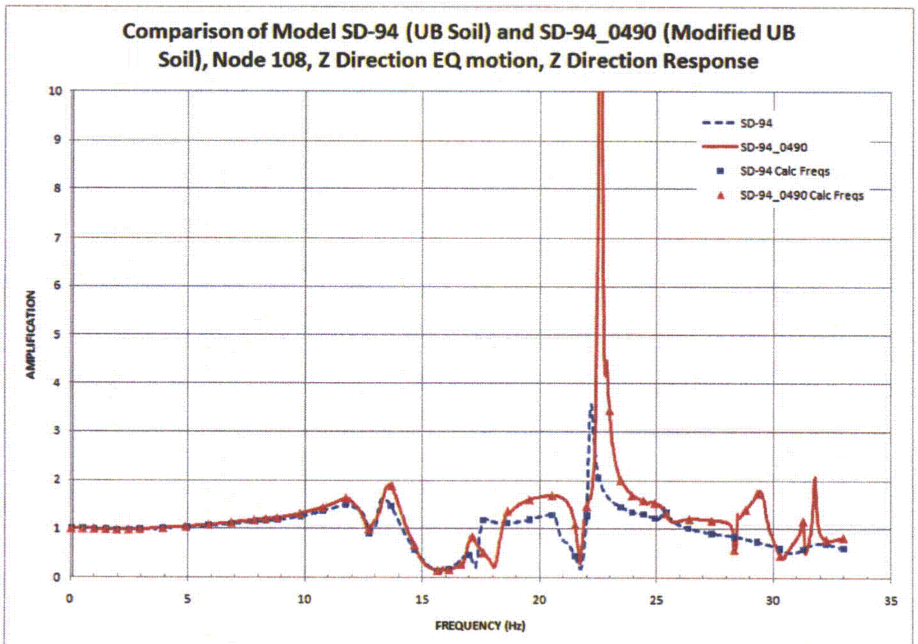


Figure 03.07.01-25.8: Comparison of Transfer Functions, Model SD-94 (UB Soil) and SD-94_0490 (Modified UB Soil), Node 108, Z Direction EQ Motion, Z Direction Response

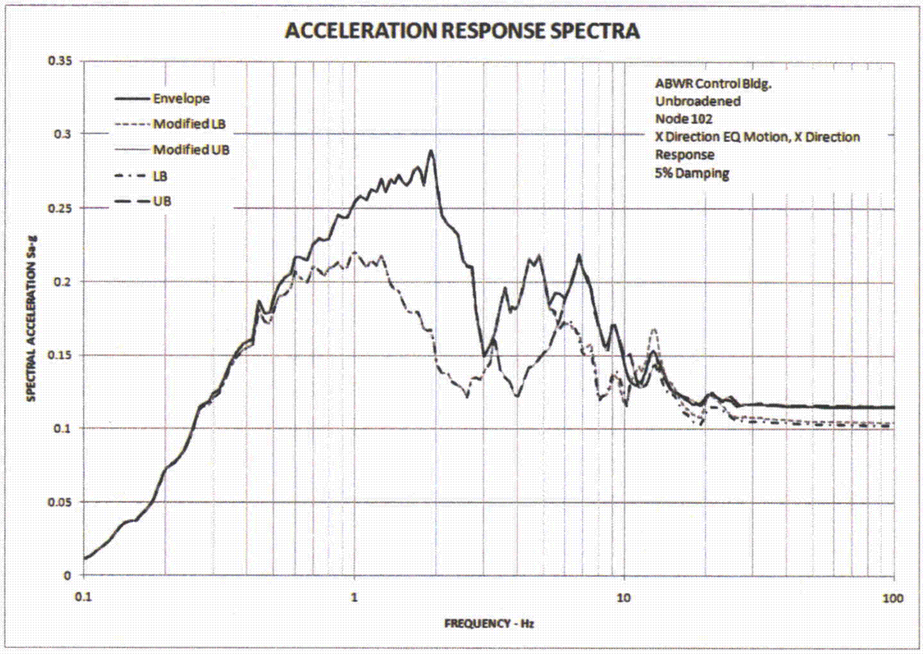


Figure 03.07.01-25.9: Comparison of Unbroadened Response Spectra, Node 102, X Direction EQ Motion, X Direction Response, 5% Damping

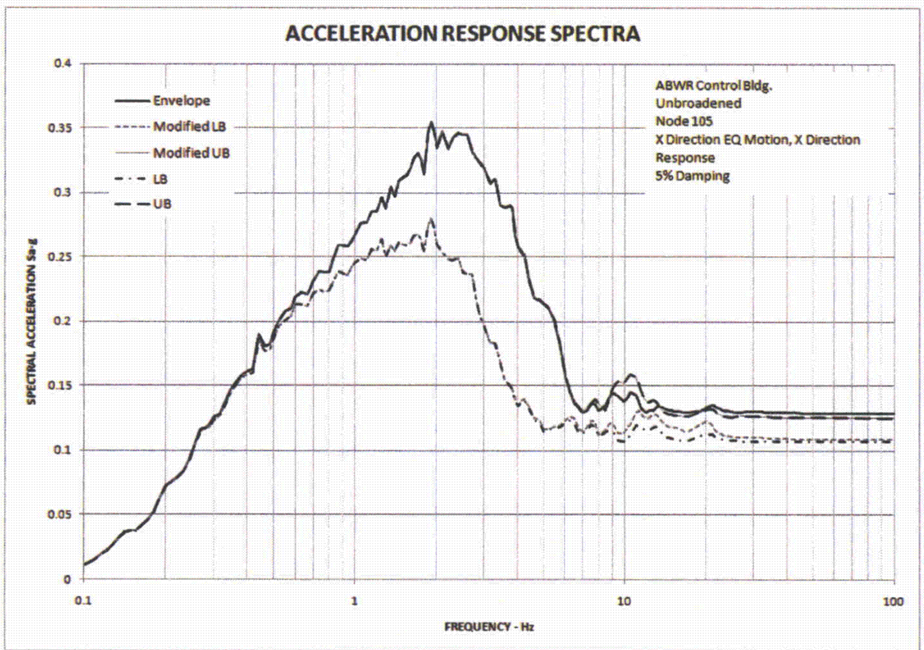


Figure 03.07.01-25.10: Comparison of Unbroadened Response Spectra, Node 105, X Direction EQ Motion, X Direction Response, 5% Damping

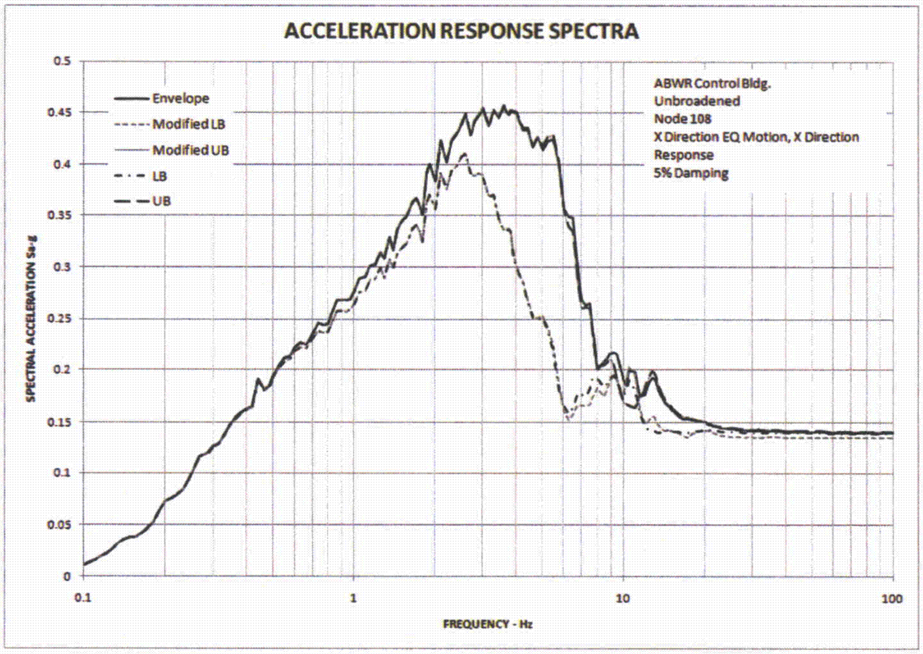


Figure 03.07.01-25.11: Comparison of Unbroadened Response Spectra, Node 108, X Direction EQ Motion, X Direction Response, 5% Damping

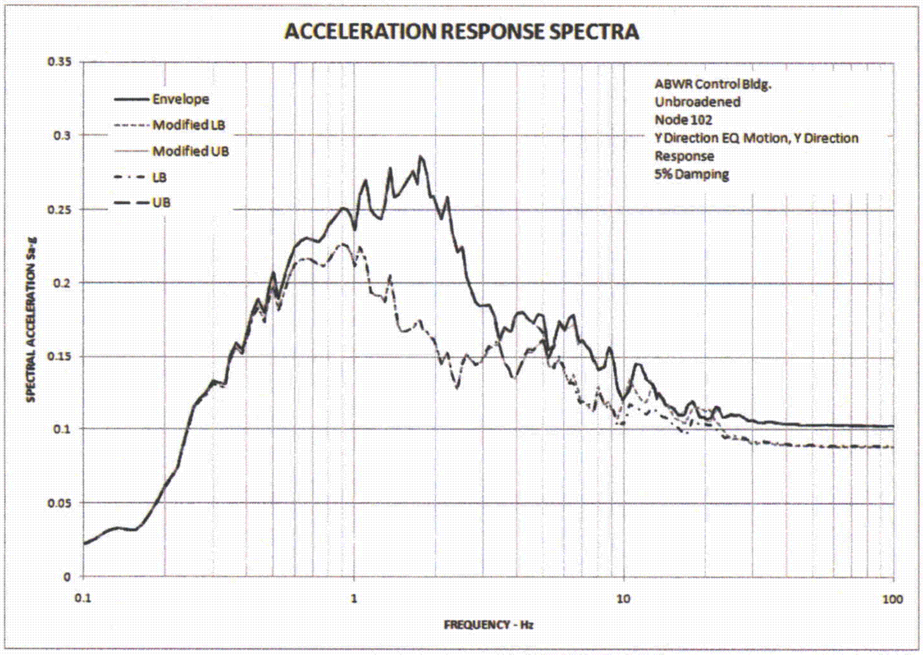


Figure 03.07.01-25.12: Comparison of Unbroadened Response Spectra, Node 102, Y Direction EQ Motion, Y Direction Response, 5% Damping

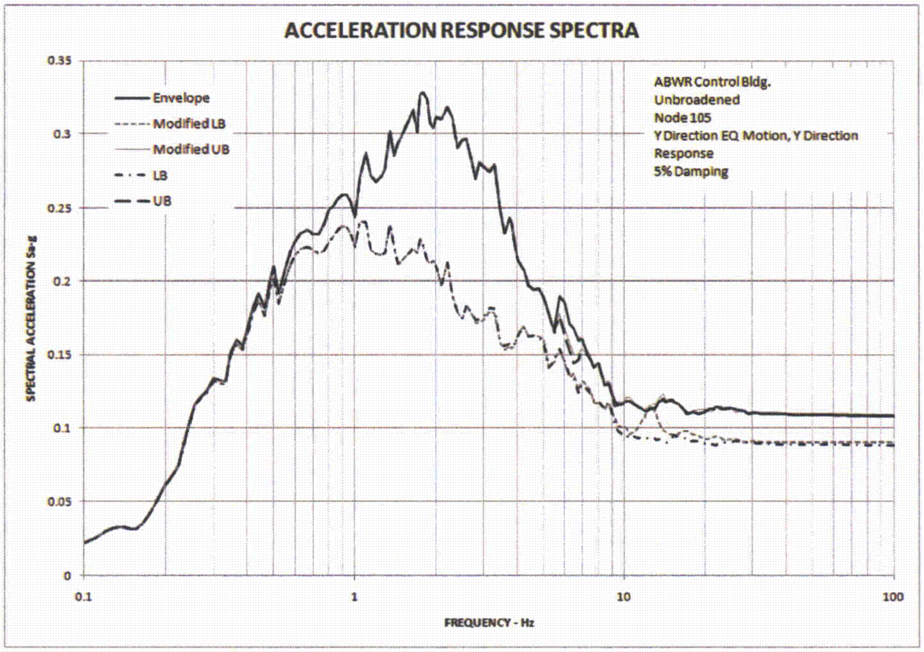


Figure 03.07.01-25.13: Comparison of Unbroadened Response Spectra, Node 105, Y Direction EQ Motion, Y Direction Response, 5% Damping

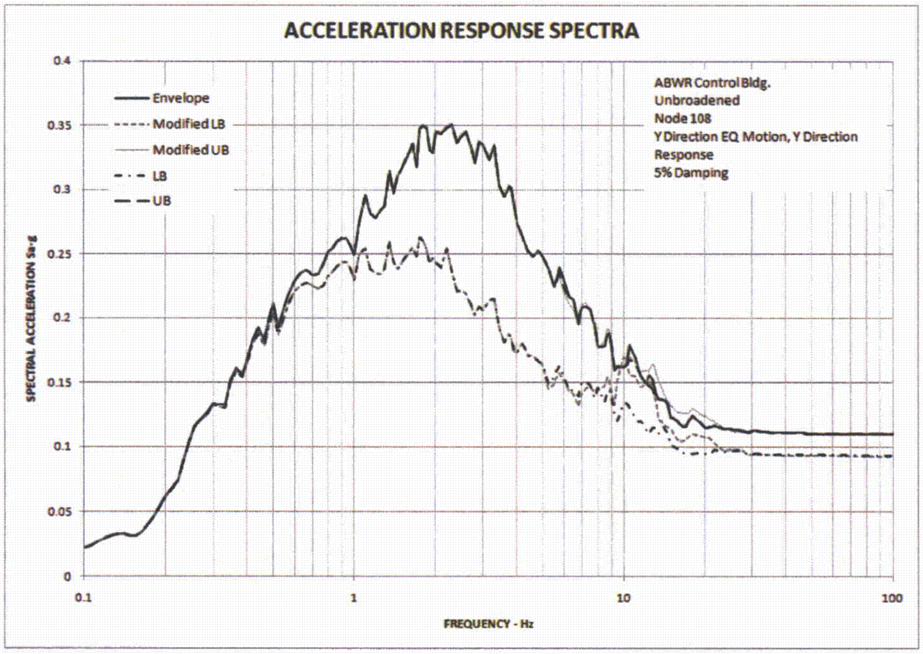


Figure 03.07.01-25.14: Comparison of Unbroadened Response Spectra, Node 108, Y Direction EQ Motion, Y Direction Response, 5% Damping

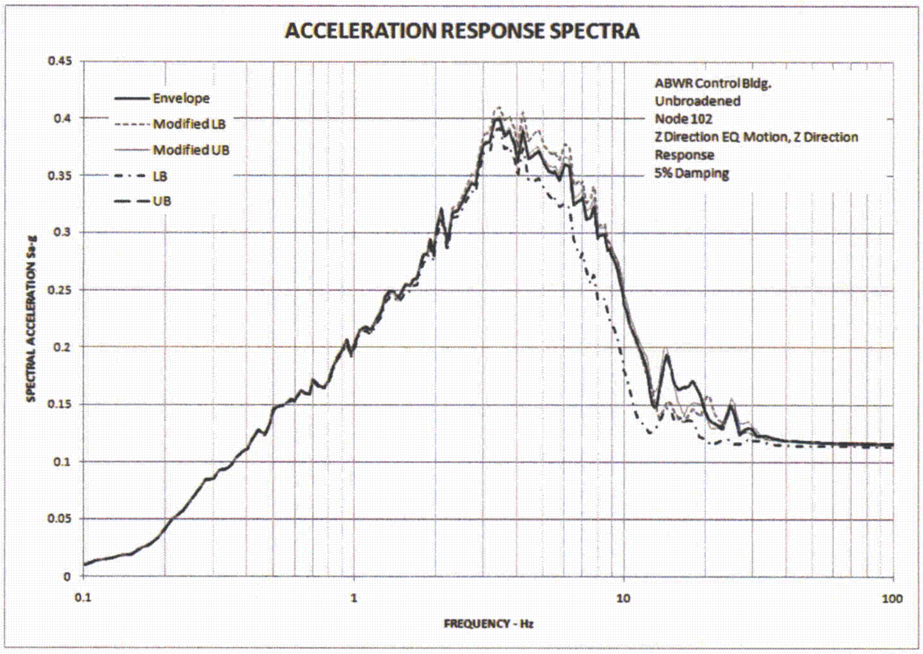


Figure 03.07.01-25.15: Comparison of Unbroadened Response Spectra, Node 102, Z Direction EQ Motion, Z Direction Response, 5% Damping

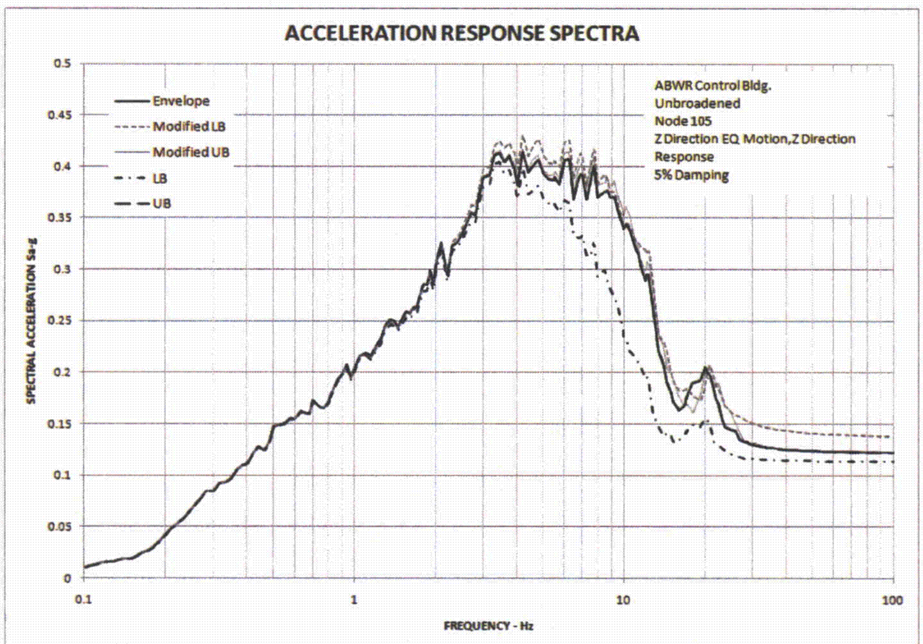


Figure 03.07.01-25.16: Comparison of Unbroadened Response Spectra, Node 105, Z Direction EQ Motion, Z Direction Response, 5% Damping

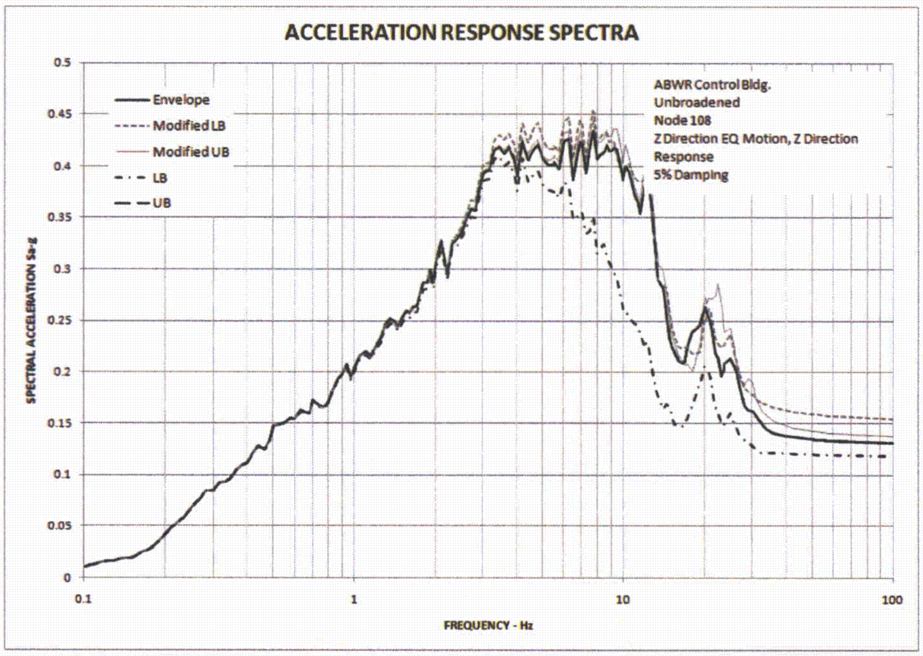


Figure 03.07.01-25.17: Comparison of Unbroadened Response Spectra, Node 108, Z Direction EQ Motion, Z Direction Response, 5% Damping

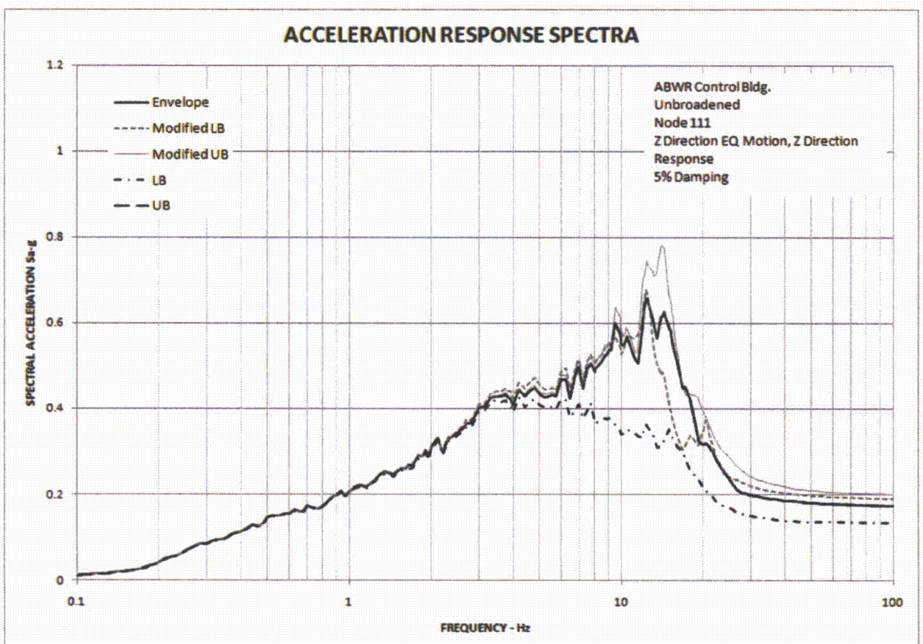


Figure 03.07.01-25.18: Comparison of Unbroadened Response Spectra, Node 111, Z Direction EQ Motion, Z Direction Response, 5% Damping

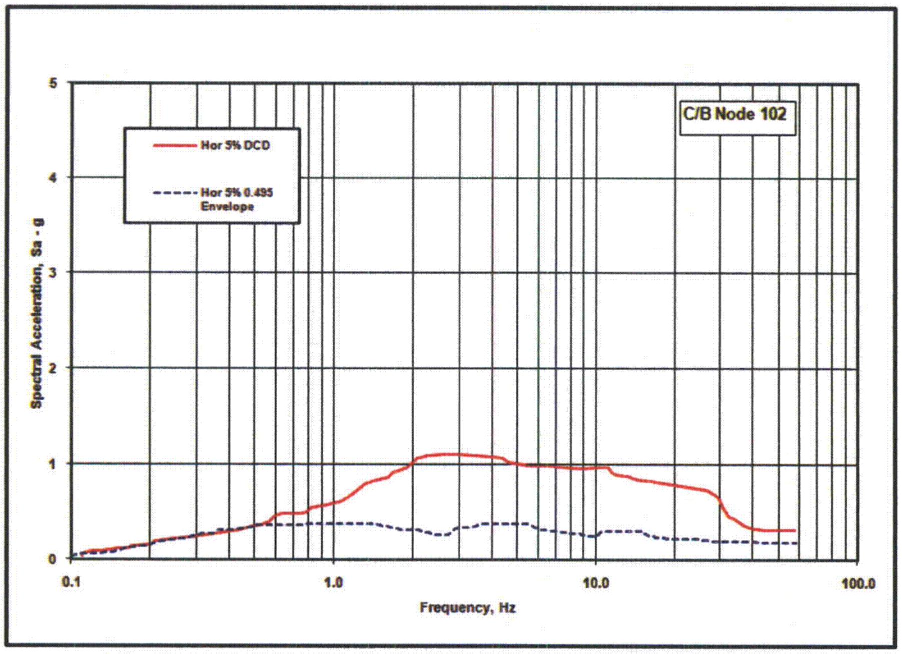


Figure 03.07.01-25.19: Comparison of Broadened Enveloped Horizontal Response Spectra using Modified Soil Properties with Broadened DCD Horizontal Response Spectra, Node 102, 5% Damping

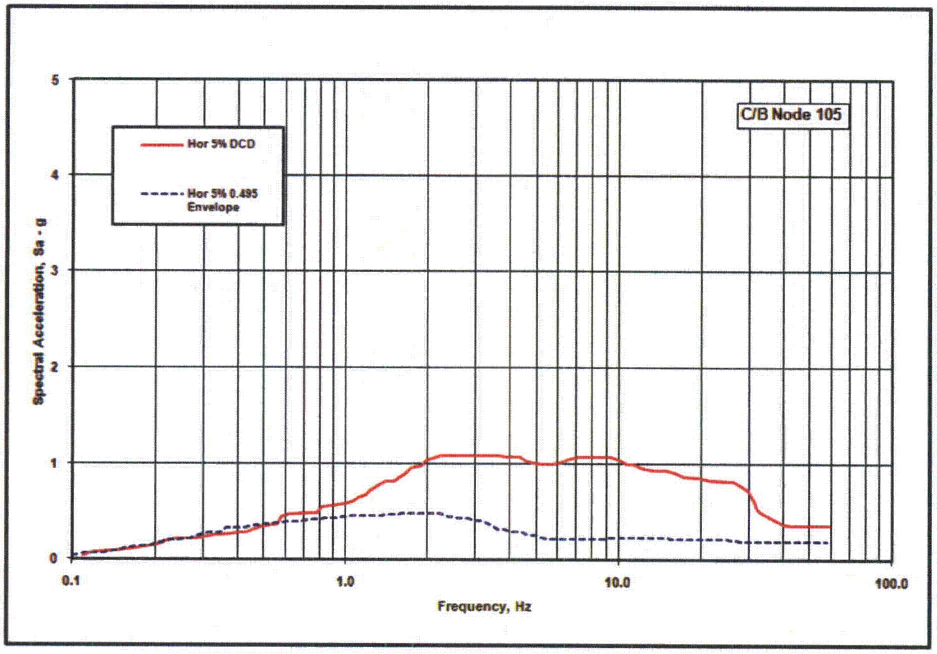


Figure 03.07.01-25.20: Comparison of Broadened Enveloped Horizontal Response Spectra using Modified Soil Properties with Broadened DCD Horizontal Response Spectra, Node 105, 5% Damping

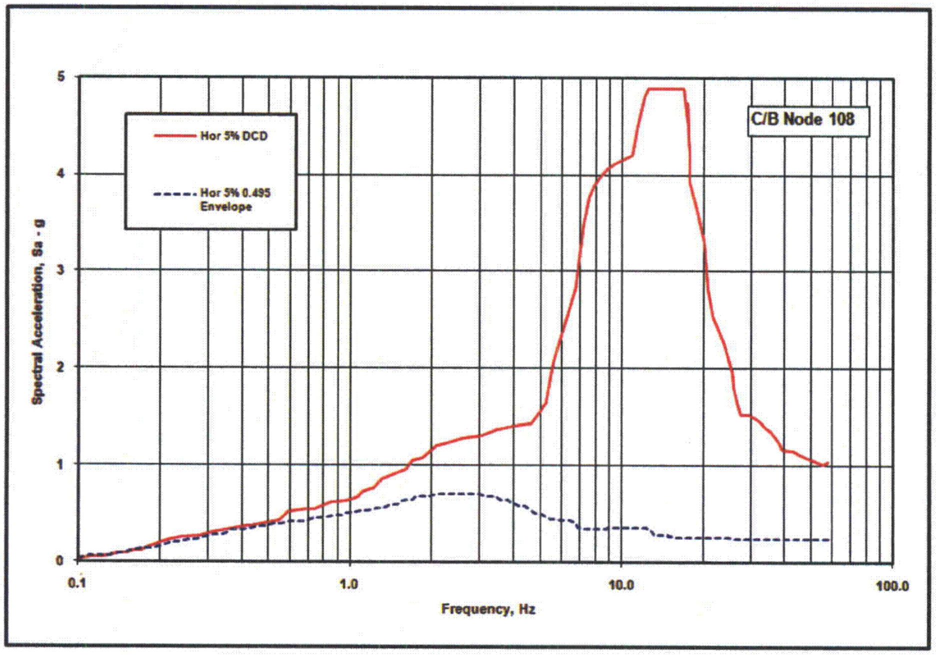


Figure 03.07.01-25.21: Comparison of Broadened Enveloped Horizontal Response Spectra using Modified Soil Properties with Broadened DCD Horizontal Response Spectra, Node 108, 5% Damping

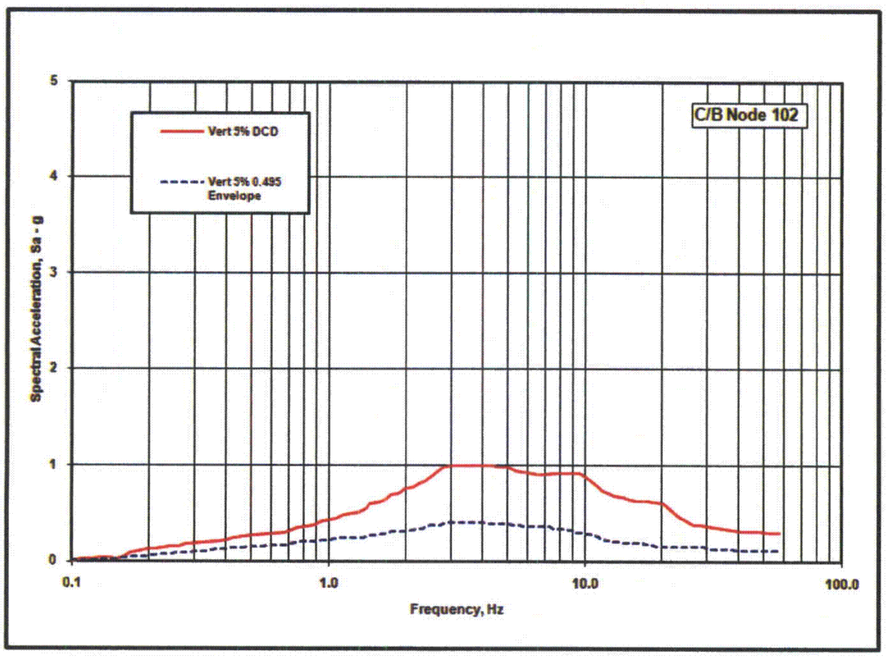


Figure 03.07.01-25.22: Comparison of Broadened Enveloped Vertical Response Spectra using Modified Soil Properties with Broadened DCD Vertical Response Spectra, Node 102, 5% Damping

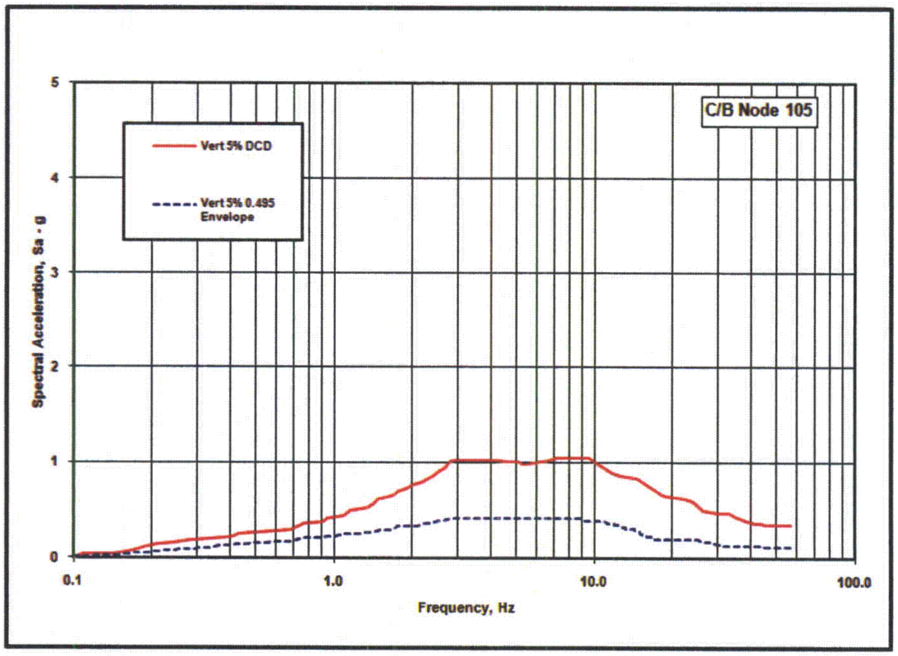


Figure 03.07.01-25.23: Comparison of Broadened Enveloped Vertical Response Spectra using Modified Soil Properties with Broadened DCD Vertical Response Spectra, Node 105, 5% Damping

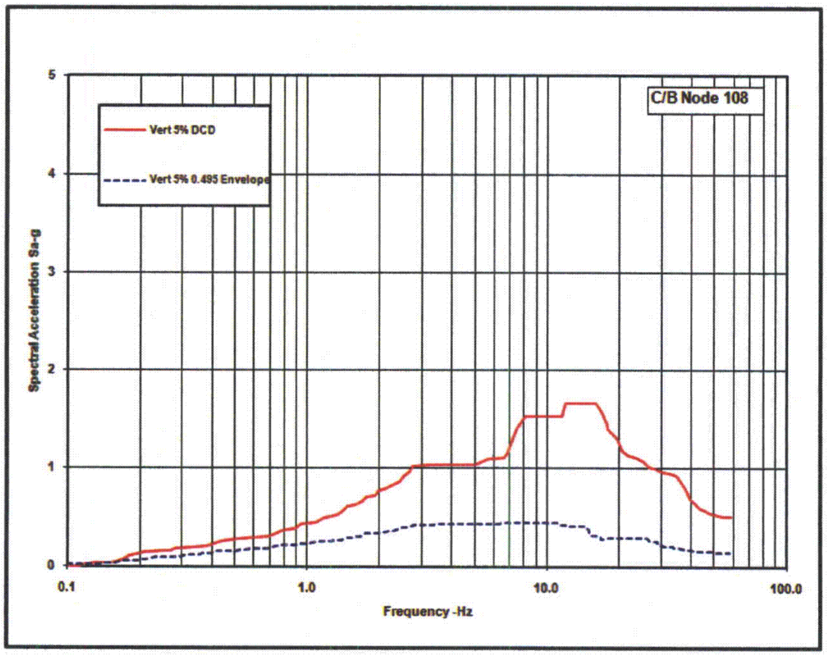


Figure 03.07.01-25.24: Comparison of Broadened Enveloped Vertical Response Spectra using Modified Soil Properties with Broadened DCD Vertical Response Spectra, Node 108, 5% Damping

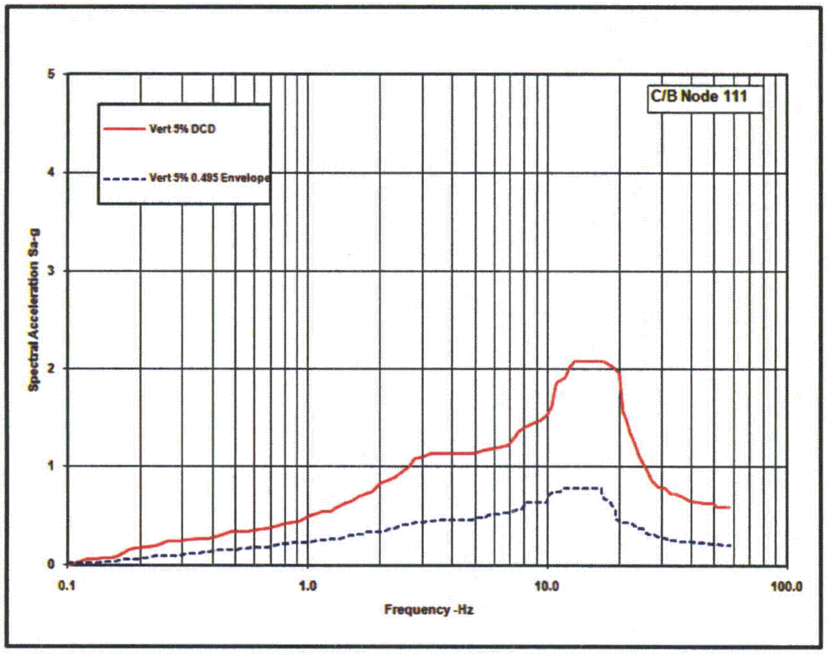


Figure 03.07.01-25.25: Comparison of Broadened Enveloped Vertical Response Spectra using Modified Soil Properties with Broadened DCD Vertical Response Spectra, Node 111, 5% Damping

RAI 03.07.02-24, Supplement 1

QUESTION:

Follow-up Question to RAI 03.07.02-15 (STP-NRC-100036)

UHS Basin and RSW Pump House:

1. 10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects and the expected duration of vibratory motion. In the response to Item 6 of RAI 03.07.01-15, the applicant has provided a table summarizing the frequencies at which transfer functions are calculated as well as the cut-off frequency used in the SSI analysis for various analysis cases including the lower bound (LB), best estimate (BE) and upper bound (UB) in-situ soil cases; LB, BE and UB backfill soil cases; the cracked concrete and de-bonded soil case. The selected cut-off frequency for the different analysis cases varies from a low of about 16 Hz to a high of 25 Hz. The applicant has stated that the lowest cut-off frequency of 16 Hz meets the ASCE 4-98 Section C3.3.3.4 recommended values.

   With respect to the selected frequency cut-off and frequencies of analysis, the staff needs the following information:

   a) Staff has not endorsed ASCE 4-98 Section C3.3.3.4 as acceptable criteria for selecting the cutoff frequency for the SSI analysis for detailed finite element model such as UHS Basin with cooling tower enclosure and RSW Pump House. The applicant is requested to provide comparisons of in-structure response spectra at some selected locations by increasing the frequency cut-off to a minimum of 33 Hz and using a SSI model capable of transmitting a frequency up to 33 Hz (refer to Follow-up Question to RAI 03.07.02-17) for all analysis cases considered demonstrating that cut-off frequencies used in the SSI analysis are acceptable. The staff needs this information to ensure that the selected cut off frequencies less than 33 Hz in SSI analysis will accurately or conservatively account for the expected frequency content of the SSE in the SSI analysis.

   b) In reviewing the tabulated SSI analysis frequencies, it is observed that some frequencies are excluded from the calculation of un-interpolated transfer functions in certain directions. For example, the frequency 14.16 Hz is not included in the z-response analysis for the mean soil case and 9.521 Hz is not included in the z-response analysis for the upper bound soil case. The applicant is requested to provide the basis for selecting the frequencies of analysis for calculating the un-interpolated transfer functions and excluding any frequencies from such calculations. The staff requires this information to ensure that the SSI analysis results are not adversely affected by any numerical instability that may be caused by large numbers of soil layers used in SASSI to model deep non-uniform soil site at the UHS/RSW Pump House.

RSW Piping Tunnel:

10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects and the expected duration of vibratory motion. In order to ensure that evaluation of RSW Piping Tunnel for SSE has appropriately taken into account SSI effects, the staff needs the following information:

1. In the response to Item 1 of RAI 03.07.02-15, the applicant has stated that a 2D SSI analysis of the RSW tunnel has been performed to quantify the in-structure response of the tunnel. No details of this analysis have been provided. As such, the applicant is requested to describe in sufficient detail in the FSAR how the SSI analysis of the RSW tunnel has been performed. The description shall include the SSI methodology, figures showing the SSI model and boundary conditions, summary of the soil and structure properties, the input motion, etc. so the review can be completed.

2. In the response to Item 2 of RAI 03.07.02-15, the applicant has stated that simple manual calculations were used for the analysis and design of individual components of the RSW piping tunnel. For this analysis, the tunnel walls, slabs and base mat are considered as rigid elements, and seismic loads are calculated based on a ZPA of 0.21g. The applicant further states that the analysis did not include any model or soil springs; the seismic loads are applied in terms of dynamic soil pressures on the exterior walls, calculated as per ASCE 4-98 recommendations. Staff has not endorsed ASCE 4-98 recommended dynamic soil pressures for design of tunnel walls. As such, the applicant is requested to provide comparisons of the dynamic soil pressures on the RSW tunnel walls calculated using 2D SSI model versus those of ASCE 4-98 to demonstrate that the design pressures are still bounding when the effects of kinematic interaction between tunnel structures and surrounding soils as well as the effects of structure-soil-structure interaction (SSSI) due to nearby heavy structures are considered.

SUPPLEMENTAL RESPONSE:

The original response to Part 1b of the UHS Basin/RSW Pump House of this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. This supplemental response provides the response to Parts 1 and 2 of the Reactor Service Water (RSW) Piping Tunnel. The response to Part 1a of the Ultimate Heat Sink (UHS) Basin/RSW Pump House is currently scheduled to be provided in a separate supplemental response in December 2010.

RSW Piping Tunnel:

Part 1

The RSW Piping Tunnel runs north from the UHS/ RSW Pump House to Control Building (CB) and passes between the Reactor Building (RB) and Radwaste Building (RWB). Since, the tunnel is a long structure, two-dimensional (2D) soil-structure interaction (SSI) analyses have been

performed for this tunnel. The following three sections of the RSW Tunnel have been used in the SSI analyses:

1. An east-west typical 2D section of the tunnel between the UHS/RSW Pump House and the RB for SSI analysis of the RSW Tunnel.
2. An east-west 2D section of the tunnel between the RWB and RB, for structure-soil-structure interaction (SSSI) analysis to determine the SSSI effect on the seismic soil pressures.
3. A north-south 2D section of the tunnel between the Diesel Generator Fuel Oil Storage Vault (DGFOSV) and the UHS/RSW Pump House, for SSSI analysis to determine the SSSI effect on the seismic soil pressures.

All of the above SSI analyses have been performed using SASSI2000 computer program. The following summarizes the details of the above stated SSI and SSSI analyses.

SSI Analysis of the Typical 2D Section of RSW Tunnel

Figure 3H.6-209 (all referenced figures and tables are included with COLA mark-ups in Enclosure 1) shows the structural part of the 2D plane-strain model of the reinforced concrete RSW Piping Tunnel with 2 ft thick mud mat under the base slab. The top of the Tunnel is 1.75 ft below grade. The model uses 4-node plane-strain elements to model the 3 ft thick exterior walls, 3 ft thick base slab, two 2 ft thick intermediate floors, 2 ft thick mud mat and the 1.75 ft soil above the Tunnel. As shown in Figure 3H.6-209, spring elements are added on the side walls of the Tunnel to calculate the seismic soil pressures on the Tunnel walls.

The specifics of this 2D SSI model are as follows:

- The structural properties (i.e. mass and stiffness) for the 2D model correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 124 ft depth with halfspace below it (more than two times the horizontal dimension of RSW Piping Tunnel plus its embedment depth).
- Six cases of strain dependent soil properties representing in-situ lower bound, mean and upper bound; and backfill lower bound, mean and upper bound are considered.
- Analysis cases also include one case with cracked concrete (50% concrete modulus value) and one case with soil separation (20 ft depth).
- Concrete and mud mat damping are assigned 4% for all cases, except 7% damping is assumed for the cracked case.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies for both vertical and horizontal directions at least up to 32.9 Hz.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is the amplified site specific SSE motion considering the effect of nearby heavy RB and UHS/RSW Pump House structures. These amplified motions were

obtained from three dimensional (3D) SSI analyses of the RB and UHS/RSW Pump House SSI analyses.

- The horizontal direction and vertical direction input motions were applied at the grade elevation.
- The responses from the horizontal and vertical direction excitations were combined using square root of sum of square (SRSS) method.
- The responses from all SSI analyses from the six soil cases, concrete cracked case and soil separation case were enveloped.
- The in-structure response spectra were peak widened by ± 15% at frequency scale.
- Envelope of the resulting response spectra for the base slab, intermediate floors and the roof slab are shown in revised COLA Part 2, Tier 2 Figures 3H.6-138 and 3H.6-139, which are used as the design in-structure response spectra for the RSW Piping Tunnel.

<u>SSSI Analysis of the East-West 2D section of the RSW piping tunnel between the RWB and RB</u>

Figure 3H.6-210 shows the structural part of the 2D plane-strain model of RB + RSW Piping Tunnel + RWB. Specifics of this SSSI analysis are as follows:

- The structural properties (mass and stiffness) for the 2D model of the individual structures correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the respective structure.
- Layered soil is modeled up to 551 ft depth with halfspace below it (more than two times the maximum horizontal dimension of any of the buildings plus their embedment depth).
- Upper bound in-situ strain-dependent soil properties were used in the SSSI analysis.
- The damping of structural part of the model is 4%.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies of at least up to 35.9 Hz in the vertical direction and 61.6 Hz in the horizontal direction.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is site specific SSE motion.
- The horizontal (E-W) input motion is applied at the grade elevation.
- Figures 3H.6-212 and 3H.6-213 show the resulting soil pressures.

<u>SSSI Analysis of the North-South 2D section of the RSW piping tunnel between the DGFOSV and UHS/RSW Pump House</u>

Figure 3H.6-211 shows the structural part of the 2D plane-strain model of RB + two DGFOSVs + RSW Piping Tunnel (adjacent to UHS/RSW Pump House) + UHS/RSW Pump House. Specifics of this SSI analysis are as follows:

- The structural properties (mass and stiffness) for the 2D model of the individual structures correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the respective structure.
- Layered soil is modeled up to 546 ft depth with halfspace below it (more than two times the maximum horizontal dimension of any of the buildings plus their embedment depth).
- Upper bound in-situ strain-dependent soil properties were used in the SSSI analysis.
- The damping of structural part of the model is 4%.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies of at least up to 35.9 Hz in the vertical direction and 61.6 Hz in the horizontal direction.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is site specific SSE motion.
- The horizontal (N-S) input motion is applied at the grade elevation.
- Figures 3H.6-214 and 3H.6-215 show the resulting soil pressures.

In the above described SSSI analyses, consistent with the SSSI analysis for certified design of the RB and CB, vertical input motion was considered to have negligible effect on the calculated soil pressures. To verify this, the SSSI analysis of the E-W 2D section of the RSW Piping Tunnel between the RWB and RB was analyzed for both the E-W and vertical input motions. The resulting soil pressures, based on SRSS of the results for the two motions, shown in Figures 3H.6-216 and 3H.6-217 show that the effect of vertical input motion is negligible.

Part 2

Figures 3H.6-212 through 3H.6-215 provide the requested comparison between the seismic soil pressures from the SSSI analysis, as described in Part 1 above, and the calculated seismic soil pressures per ASCE 4-98. The existing design as discussed in response to RAI 03.07.02-15 (submitted with letter U7-C-STP-NRC-100036 dated February 10, 2010) was re-evaluated for the resulting seismic soil pressures from the SSSI analysis. Although the existing design was found to be adequate for these SSSI soil pressures, a portion of the design for the access region near the UHS/RSW Pump House was revised due to design development. COLA Part 2, Tier 2 Table 3H.6-6 is revised to reflect this design change.

In addition, a finite element analysis using a two dimensional (2D) SAP2000 model with soil springs representing the foundation was performed to confirm adequacy of the design using manual calculations described in response to RAI 03.07.02-15. Furthermore, design of the RSW Piping Tunnel accounts for the axial tensile strain and induced forces at tunnel bends due to SSE wave propagation. The axial tensile strain is accounted for as described in COLA Part 2, Tier 2 Section 3H.6.6.2.2. The induced forces at the tunnel bends are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation.

The COLA will be revised as shown in Enclosure 1.

Enclosure 1

Revision to COLA Section 3H.6

3H.6.1 Objective and Scope

The objective of this appendix is to describe the structural analysis and design of the STP 3 & 4 site-specific seismic Category I structures that are identified below.

(1) Ultimate Heat Sink (UHS) for each unit consists of a water retaining basin with enclosed cooling towers situated above the basin and a Reactor Service Water (RSW) pump house that is integral with the UHS basin.

(2) RSW piping tunnel for each unit.

(3) Diesel Generator Fuel Oil Storage Vault for each unit.

The details of analysis and design for Items (1) and (2) are provided in Sections 3H.6.32 through 3H.6-6. The details for Item (3) are provided in Section 3H.6.7.

3H.6.2 Summary

For the design of the UHS basin and the pump house of each unit, the seismic effects were determined by performing a soil-structure interaction (SSI) analysis, as described in Subsection 3H.6.5. The free-field ground response spectra used in the analysis are described in Subsection 3H.6.5.1.1.1. The resulting seismic loads were used in combination with other applicable loads to develop designs of the structures. Hydrodynamic effects of the water in the basin were considered. The following results for the UHS/RSW Pump House are presented in tables and figures, as indicated. Results for the RSW Piping Tunnel are presented in Sections 3H.6.5.3 and 3H.6.6.2.2.

- Natural frequencies (Table 3H.6-3).
- Seismic accelerations (Table 3H.6-4).
- Seismic displacements (Table 3H.6-4).
- Floor response spectra (Figures 3H.6-16 through 3H.6-39).
- Factors of safety against sliding, overturning, and flotation (Table 3H.6-5).
- Combined forces and moments at critical locations in the structures along with required and provided rebar (Tables 3H.6-7 through 3H.6-9 and Figures 3H.6-51 through 3H.6-136).
- Lateral soil pressures for design (Figures 3H.6-41 through 3H.6-443).
- Lateral soil pressures for stability evaluation (Figures 3H.6-45 through 3H.6-50).
- Tornado evaluation results (Table 3H.6-10).

The final combined responses are used to evaluate the designs against the following criteria:

- Stresses in concrete and reinforcement are less than the allowable stresses in accordance with the applicable codes listed in Subsection 3H.6.4.1.
- The factors of safety against flotation, sliding, and overturning of the structures under various loading combinations are higher than the required minimum values identified in Subsection 3H.6.4.5.
- The calculated static and dynamic soil bearing pressures/displacements are less than the allowable values.
- The thickness of the roof slabs and exterior walls are more than the minimum required to preclude penetration, perforation, or spalling resulting from impact of design basis tornado missiles. In addition, the passage of tornado missiles through openings in the roof slabs and exterior walls is prevented by the use of missile-proof covers and doors, or the trajectory of missiles through ventilation openings is limited by labyrinth walls configured to prevent safety-related substructures and components from being impacted.

The RSW piping tunnel seismic analysis has been performed using SSI analysis~~an equivalent static approach~~, as discussed in Section 3H.6.5.3.

3H.6.4.3.1.4 Lateral Soil Pressures (H)

Lateral soil pressures are calculated using the following soil properties.

- Unit weight (moist):..120 pcf (1.92 t/m³)
- Unit weight (saturated): ....140 pcf (2.24 t/m³)
- Internal friction angle:...30°
- Poisson's ratio (above groundwater)...0.42
- Poisson's ratio (below groundwater)...0.47

The calculated lateral soil pressures are presented in figures as indicated:

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43.
- ~~Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44.~~
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

3H.6.4.3.3.3 Lateral Soil Pressures Including the Effects of SSE (H')

The calculated lateral soil pressures including the effects of SSE are presented in figures as indicated:

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43.
- ~~Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44.~~
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

3H.6.5.3 Seismic Analysis of RSW Piping Tunnels

~~The seismic analysis of the RSW piping tunnel was performed using a 2-dimensional SSI model of the tunnel section. In order to account for the effect of the adjacent Reactor Building on the input motion to be used for the SSI analysis, the site-specific design time history described in Section 3H.6.5.1.1.2 was amplified by 15%. The OBE damping (4%) was used for the analysis and in-structure response spectra generation. The analysis was performed for the upper-bound, mean, and lower-bound soil conditions. The in-structure response spectra at the base slab and all three levels of the tunnel were enveloped and broadened by 15% to obtain the horizontal and vertical response spectra presented in Figures 3H.6-138 and 3H.6-139 for the RSW tunnel design. The traveling wave effects during a seismic event that are acting on the structure have been considered per Section 3.5.2.1 of ASCE 4-98.~~

The RSW Piping Tunnel runs north from the UHS/RSW Pump House to Control Building (CB) and passes between the Reactor Building (RB) and Radwaste Building (RWB). Since, the tunnel is a long structure, two dimensional (2D) SSI analyses have been performed for this tunnel. The following three sections of the RSW Tunnel have been used in the SSI analyses:

- An east-west typical 2D section of the tunnel between the UHS/RSW Pump House and the RB for SSI analysis of the RSW tunnel.
- An east-west 2D section of the tunnel between the RWB and RB, for structure-soil-structure interaction (SSSI) analysis to determine the SSSI effect on the seismic soil pressures.
- A north-south 2D section of the tunnel between the Diesel Generator Fuel Oil Storage Vault (DGFOSV) and the UHS/RSW Pump House, for SSSI analysis to determine the SSSI effect on the seismic soil pressures.

All of the above SSI analyses have been performed using SASSI2000 computer program. The following summarizes the details of the above stated SSI and SSSI analyses.

SSI Analysis of the Typical 2D Section of RSW Tunnel

Figure 3H.6-209 shows the structural part of the 2D plane-strain model of the reinforced concrete RSW Piping Tunnel with 2 ft thick mud mat under the base slab. The top of the tunnel is 1.75 ft below grade. The model uses 4-node plane-strain elements to model the 3 ft thick exterior walls, 3 ft thick base slab, two 2 ft thick intermediate floors, 2 ft thick mud mat and the 1.75 ft soil above the tunnel. As shown in Figure 3H.6-209, spring elements are added on the side walls of the tunnel to calculate the seismic soil pressures on the tunnel walls.

The Specifics of this 2D SSI model are as follows:

- The structural properties (i.e. mass and stiffness) for the 2D model correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 124 ft depth with half space below it (more than two times the horizontal dimension of RSW Piping Tunnel plus its embedment depth).
- Six cases of strain dependent soil properties representing in-situ lower bound, mean and upper bound; and backfill lower bound, mean and upper bound are considered.
- Analysis cases also include one case with cracked concrete (50% concrete modulus value) and one case with soil separation (20 ft depth).
- Concrete and mud mat damping are assigned 4% for all cases, except 7% damping is assumed for the cracked case.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.

- Model is capable of passing frequencies for both vertical and horizontal directions at least up to 32.9 Hz.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is the amplified site specific SSE motion considering the effect of nearby heavy RB and UHS/RSW Pump House structures. These amplified motions were obtained from three dimensional (3D) SSI analyses of the RB and UHS/RSW PH SSI analyses.
- The horizontal direction and vertical direction input motions were applied at the grade elevation.
- The responses from the horizontal and vertical direction excitations were combined using square root of sum of square (SRSS) method.
- The responses from all SSI analyses from the six soil cases, concrete cracked case and soil separation case were enveloped.
- The in-structure response spectra were peak widened by ± 15% at frequency scale.

- Envelope of the resulting response spectra for the base slab, intermediate floors and the roof slab shown in Figures 3H.6-138 and 3H.6-139 are used as the design in-structure response spectra for the RSW Piping Tunnel.

SSSI Analysis of the East-West 2D section of the RSW piping tunnel between the RWB and RB

Figure 3H.6-210 shows the structural part of the 2D plane-strain model of RB + RSW Piping Tunnel + RWB. Specifics of this SSSI analysis are as follows:

- The structural properties (mass and stiffness) for the 2D model of the individual structures correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the respective structure.
- Layered soil is modeled up to 551 ft depth with halfspace below it (more than two times the maximum horizontal dimension of any of the buildings plus their embedment depth).
- Upper bound in-situ strain-dependent soil properties were used in the SSSI analysis.
- The damping of structural part of the model is 4%.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies of at least up to 35.9 Hz in the vertical direction and 61.6 Hz in the horizontal direction.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is site specific SSE motion.
- The horizontal (E-W) input motion is applied at the grade elevation.
- Figures 3H.6-212 and 3H.6-213 show the resulting soil pressures.

SSSI Analysis of the North-South 2D section of the RSW piping tunnel between the DGFOSV and UHS/RSW PH

Figure 3H.6-211 shows the structural part of the 2D plane-strain model of RB + two DGFOSVs + RSW Piping Tunnel (adjacent to UHS/RSW Pump House) + UHS/RSW PH. Specifics of this SSI analysis are as follows:

- The structural properties (mass and stiffness) for the 2D model of the individual structures correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the respective structure.
- Layered soil is modeled up to 546 ft depth with halfspace below it (more than two times the maximum horizontal dimension of any of the buildings plus their embedment depth).
- Upper bound in-situ strain-dependent soil properties were used in the SSSI analysis.

- The damping of structural part of the model is 4%.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies of at least up to 35.9 Hz in the vertical direction and 61.6 Hz in the horizontal direction.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is site specific SSE motion.
- The horizontal (N-S) input motion is applied at the grade elevation.
- Figures 3H.6-214 and 3H.6-215 show the resulting soil pressures.

3H.6.6.2.2 RSW Piping Tunnels

The individual components of the RSW Piping Tunnels (roof slab, intermediate slabs, base mat and walls) have out-of-plane frequency in excess of 33 Hz and their out-of- plane seismic loads are determined using a conservative acceleration of 0.21g which exceeds the maximum Zero Period Acceleration (ZPA) of response spectra Figures 3H.6-138 and 3H.6-139. Manual calculations are used for the analysis and design of individual components of the RSW Piping Tunnels (roof slab, intermediate slab, base mat, walls) considering all applicable loads and load combinations including dead load, live load, earth pressure loads, wind and tornado loads, SSE seismic loads, internal flood loads and external flood loads.

In general the walls and slabs are designed as one-way slabs with walls spanning in the vertical direction and the slabs spanning in the East-West direction (normal to the tunnel axis). All connections are conservatively considered pinned except for those connecting to the base mat, which are considered fixed. The resulting moments and shears from this simplified analysis along with any induced axial tension or compression due to dead load and/or reactions from adjoining elements are used to determine the required rebar in accordance with the requirements of ACI 349-97. Table 3H.6-6 provides the design summary for RSW Piping Tunnels.

The tensile axial strain on the RSW Tunnel due to Safe Shutdown Earthquake (SSE) wave propagation is determined based on the equations and commentary outlined in Section 3.5.2.1 of ASCE 4-98. Equation 3.5-1 of ASCE 4-98 is used to compute the axial strain. As this equation gives the upper bound, Equation 3.5-2 from Section 3.5.2.1.2 of ASCE 4-98 is conservatively neglected.

The maximum curvature is computed based on Equation 3.5-3 in Section 3.5.2.1.3 of ASCE 4 98. The maximum curvature is then converted into additional axial strain by multiplying the curvature by the distance from the centroid of the RSW Piping Tunnels to the extreme fiber of the RSW Tunnel. For these computations, the following parameters are considered:

- Rayleigh waves with apparent wave velocity of 3,000 ft/sec (as recommended in appendix C3.5.2.1 of ASCE 4-98)

- Conservative ground acceleration of 0.21g
- Maximum ground velocity of 10.08 in/sec (which is based on 48 in/sec per 1.0g ground acceleration)

The tensile axial strain and strain due to maximum curvature are conservatively added together to obtain the actual strain in the longitudinal direction of the RSW Tunnel. The actual strain is then compared to the cracking strain of concrete and maximum allowable strain of the reinforcing. The maximum computed tensile axial strain is 2.9 x 10^{-4} in/in which is about 14% of the rebar yield strain of 2.069 x 10^{-3} in/in. The design also accounts for the induced forces at tunnel bends due to SSE wave propagation. These forces are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation.

This analysis considered the loads identified below, combined in accordance with Subsection 3H.6.4.3.4.

- Dead load of the tunnel walls and the soil above the tunnel.
- Live load of 200 psf (9.6 kPa) applied to the floor of the tunnels.
- At-rest lateral soil pressure on the tunnel walls.
- Hydrostatic pressures on the tunnel walls due to groundwater.
- Envelope of d~~D~~ynamic lateral soil pressures on the tunnel walls, due to an SSE, calculated from: (a) ~~calculated~~ using the methodology defined in Subsection 3.5.3.2.2 of ASCE 4, (b) soil-structure interaction (SSI) analysis, and (c) the structure-soil-structure interaction (SSSI) analysis. At rest ~~L~~lateral soil pressures for typical section of the ~~used for design of~~ RSW Piping Tunnels using ASCE 4 methodology are presented in Figure 3H.6-44. Figures 3H.6-212 through 3H.6-215 provide comparison of lateral seismic soil pressures from SSSI analysis described in Section 3H.6.5.3 to those from ASCE 4 methodology.
- Surcharge pressure of 500 psf (23.9 kPa) applied to the ground above the tunnels.
- SSE forces corresponding to the weight of the tunnels being acted on by the accelerations established by the SSI analysis.

~~The tensile axial strain and strain due to maximum curvature are conservatively added together to obtain the actual strain in the longitudinal direction of the RSW Tunnel. The actual strain is then compared to the cracking strain of concrete and maximum allowable strain of the reinforcing. The maximum computed tensile axial strain is 2.9 x 10^{-4} in/in which is about 14% of the rebar yield strain of 2.069 x 10^{-3} in/in. This analysis considered the loads identified below, combined in accordance with Subsection 3H.6.4.3.4.~~

3H.6.6.5 Stability Evaluations

The factors of safety of the combined UHS basin and RSW pump house ~~and RSW Piping tunnel~~ against sliding, overturning, and flotation are provided in Table 3H.6-5. The factors of safety of the RSW Piping tunnel against sliding, overturning and flotation are provided in Table 3H.6-16.

Table 3H.6-6: Results of RSW Piping Tunnel Design

Location	Item	Thickness (ft)	Governing Load Combination	Design Moment (kip-ft/ft)	Design Shear (kip/ft)	Area of Reinforcement (in^2/ft)			
						Moment Reinforcement [1]		Shear Reinforcement	
						Required	Provided (both faces)	Required	Provided
Main Tunnel	Exterior Wall	3'-0"	1.4D+1.7L+1.4F+1.7H	136.47	21.96	1.16 (vertical)	1.27 (vertical)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	55.90	11.29	0.7 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	95.22	13.16	1.13 (east-west)	1.27 (east-west)	None	None
	Basemat	3'-0"	D+Lo+F+H'+E' [2]	123.94	19.10	0.97 (east-west)	1.00 (east-west)	None	None
North End of Main Tunnel (near Control Building)	Exterior Wall	3'-0"	1.4D+1.7L+1.4F+1.7H	324.37	34.23	2.19 (east-west)	2.25 (east-west)	None	None
	Interior Wall	2'-0"	D+Lo+F+H'+E' [2]	152.15	19.96	1.69 (east-west)	2.25 (east-west)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	86.64	15.29	0.70 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	136.30	18.03	1.49 (east-west)	2.25 (east-west)	None	None
	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	70.42	28.27	0.36 (north-south)	0.79 (north-south)	None	None
			1.4D+1.7L+1.4F+1.7H	155.74	36.39	1.16 (east-west)	1.27 (east-west)	None	None
Main Tunnel (near Access Region 1)	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	46.60	20.54	0.70 (north-south)	0.79 (north-south)	None	None

Table 3H.6-6: Results of RSW Piping Tunnel Design (Continued)

Location	Item	Thickness (ft)	Governing Load Combination	Design Moment (kip-ft/ft)	Design Shear (kip/ft)	Area of Reinforcement (in²/ft)			
						Moment Reinforcement (1)		Shear Reinforcement	
						Required	Provided (both faces)	Required	Provided
Main Tunnel (near Access Region 2)	Exterior Wall	3'-0"	D+Lo+F+H'+E'	321.96	29.22	2.21 (vertical)	2.25 (vertical)	None	None
				214.84	29.22	1.40 (horizontal)	1.56 (horizontal)	None	None
	Basemat	6'-0"	D+Lo+F+H'+E' (2)	530.76	66.74	1.66 (east-west)	2.25 (east-west)	None	None
			1.4D+1.7L+1.4F+1.7H / D+Lo+F+H'+E' (2)	500.50	66.74	1.78 (north-south)	2.25 (north-south)	None	None
Main Tunnel (near Access Region 3) North of Pump House	Exterior Wall	3'-0"	1.4D+1.7L+1.4F+1.7H	147.60	21.99	1.16 (vertical)	1.56 (vertical)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	344.53	37.20	2.56 (north-south)	4.68 (north-south)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' (2)	150.97	19.29	1.70 (north-south)	3.12 (north-south)	None	None
	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	236.52	38.12	1.74 (north-south)	3.12 (north-south)	0.18	0.20

Notes:

1) Unless noted otherwise, the required reinforcement in the direction not reported in the table is controlled by the minimum required reinforcement. The minimum required reinforcement for 2'-0" thick and 3'-0" thick elements is 0.36 in²/ft and 0.54 in²/ft. For such cases the provided reinforcement is 0.79 in²/ft.

2) The loading also includes loads due to internal flooding.

3) The following additional reinforcement is required due to SSE Wave Propagation:
- For the Main Tunnel, #8 bars at 12" o.c. in the longitudinal direction of the Main Tunnel for 96'-0" (measured north from the centerline of the intersection of the Main Tunnel and Access Region 3)
- For Access Region 3 from 0'-0" to 56'-0" (measured east from the centerline of the intersection of the Main Tunnel and Access Region 3)
  - i. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the roof
  - ii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the interior slab
  - iii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the basemat
- For Access Region 3 from 56'-0" to 103'-0" (measured east from the centerline of the intersection of the Main Tunnel and Access Region 3)
  - i. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the roof
  - ii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the basemat

Table 3H.6-16: Factors of Safety Against Sliding, Overturning, and Flotation for Reactor Service Water Tunnel

Load Combination	Calculated Safety Factor			Notes
	Overturning	Sliding	Flotation	
D + F'	---	---	1.18	2
D + H + W	2.29	50.76	---	
D + H + Wt	2.23	21.31	---	
D + H + E'	1.1	1.29	---	2, 3

Notes:

1) Loads D, H, W, Wt, and E` are defined in Subsection 3H.6.4.3.4.1. F` is the buoyant force corresponding to the design basis flood.

2) Coefficients of friction for sliding resistance are 0.45 for static conditions and 0.30 for dynamic conditions for the RSW Tunnel.

3) The calculated safety factors consider less than half of the full passive pressure. The calculated safety factors increase if full passive pressure (Kp = 3.0) is considered.

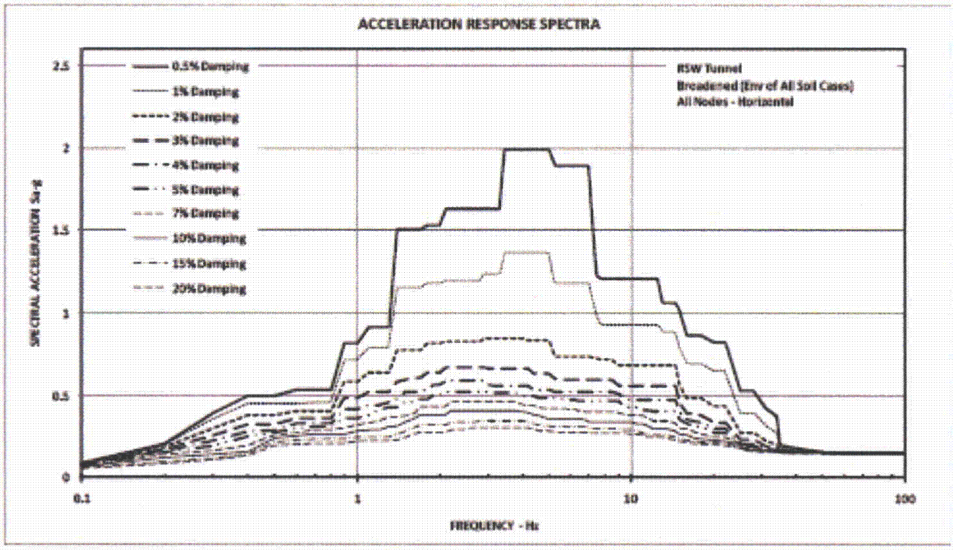


Figure 3H.6-138: RSW Piping Tunnel, Horizontal Response Spectra

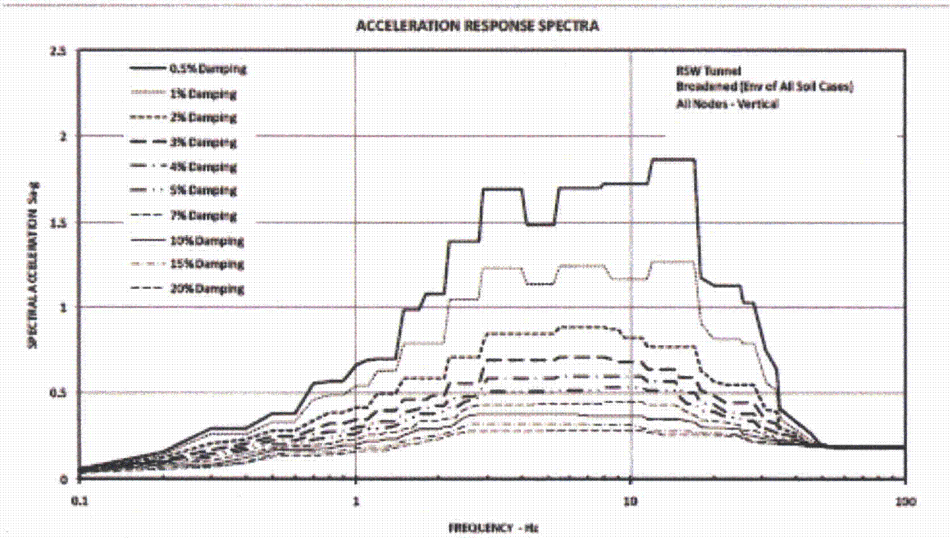


Figure 3H.6-139: RSW Piping Tunnel, Vertical Response Spectra

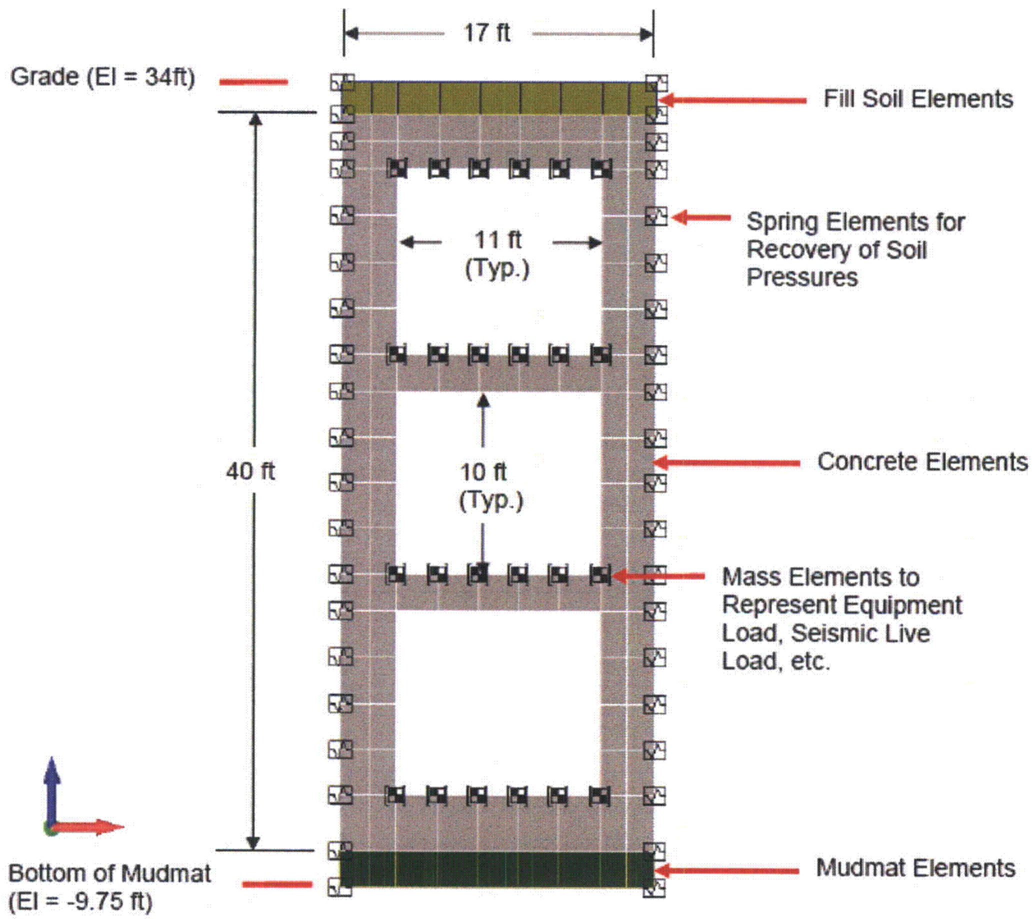


Figure 3H.6-209: SSI Model of RSW Piping Tunnel

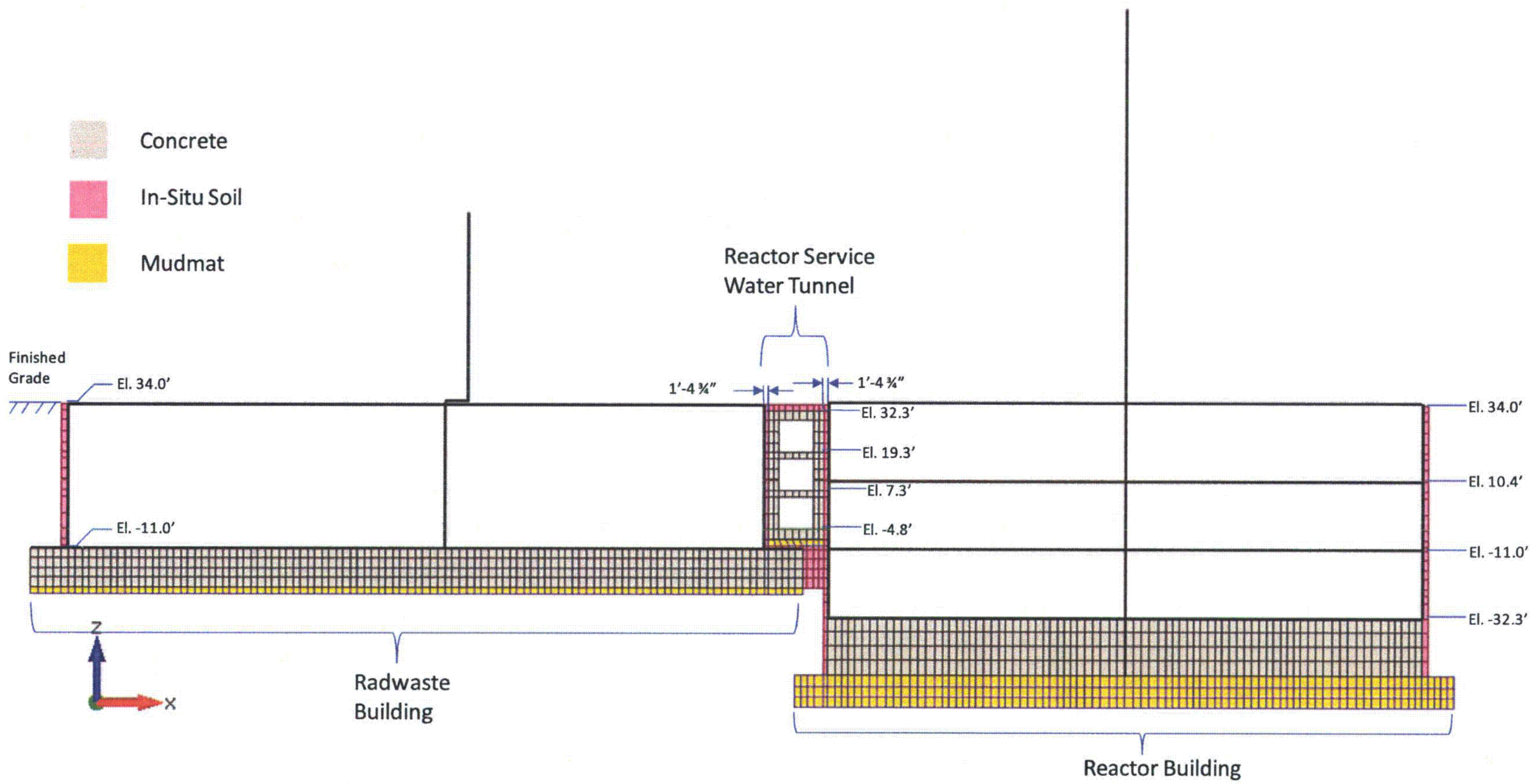


Figure 3H.6-210: SSSI 2D Model of RB + RSW Piping Tunnel + RWB

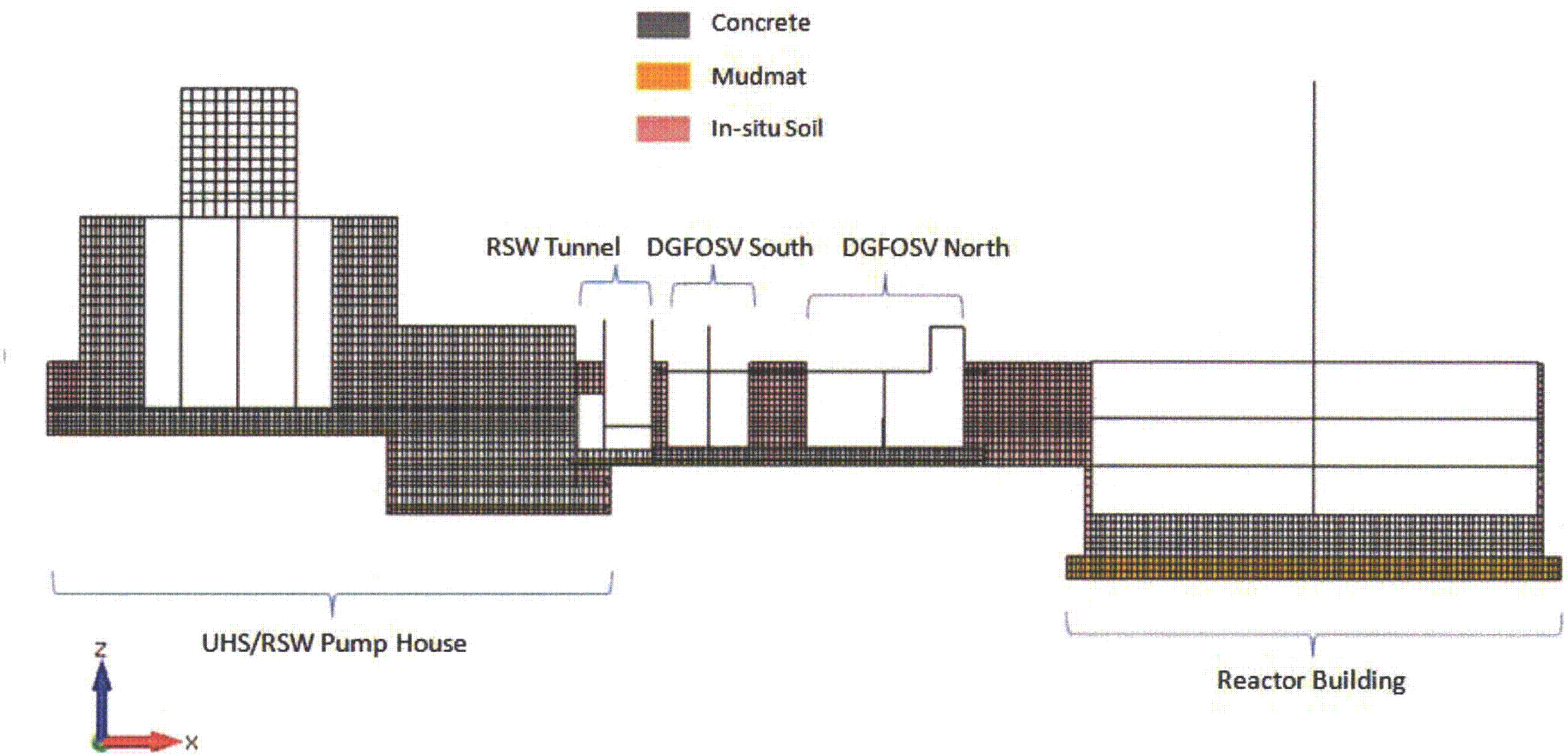


Figure 3H.6-211: 2D Model of UHS/RSW Pump House, RSW Piping Tunnel, DGFOSVs and RB

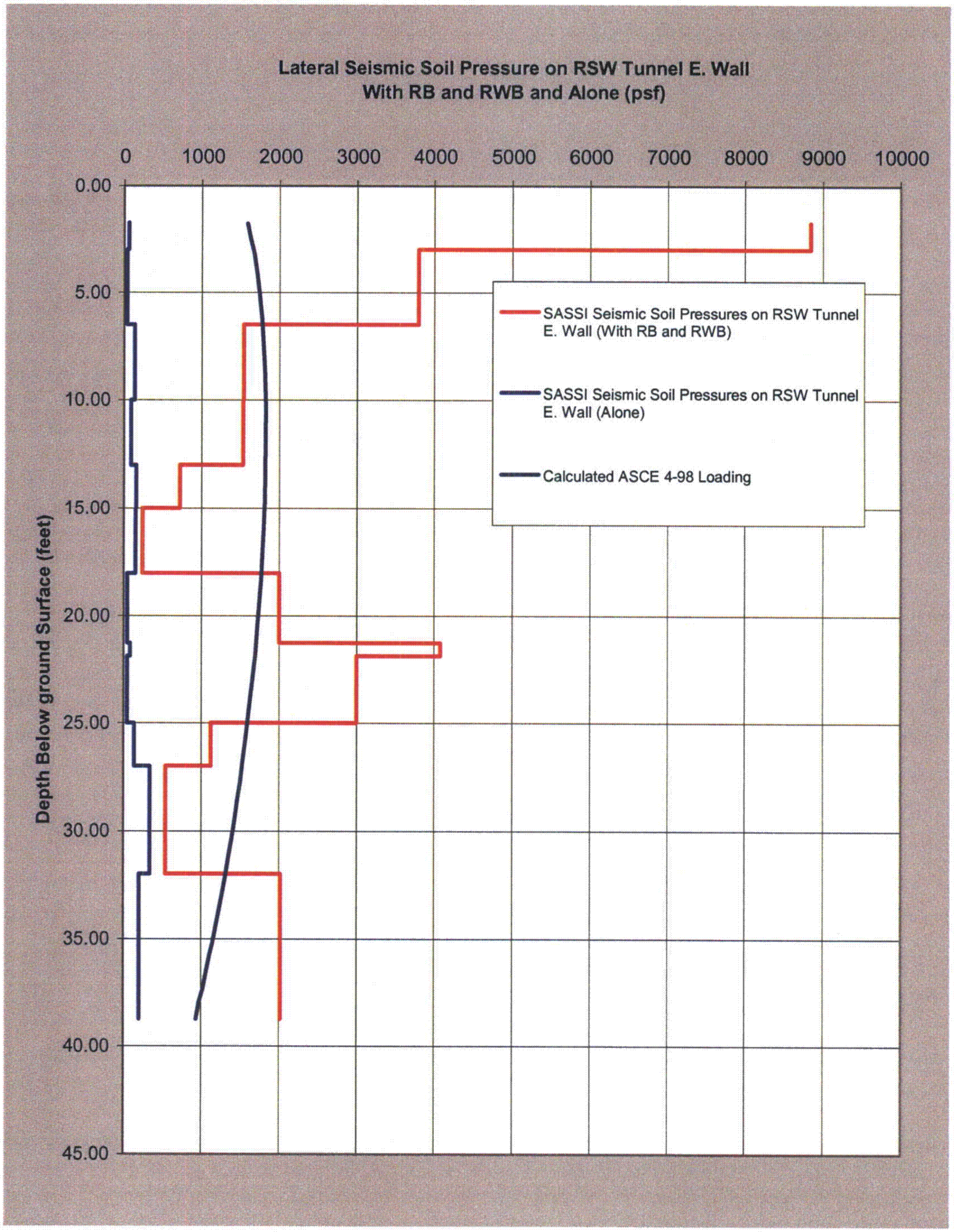


Figure 3H.6-212: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel East Wall (Main Cross Section of RSW Piping Tunnel)

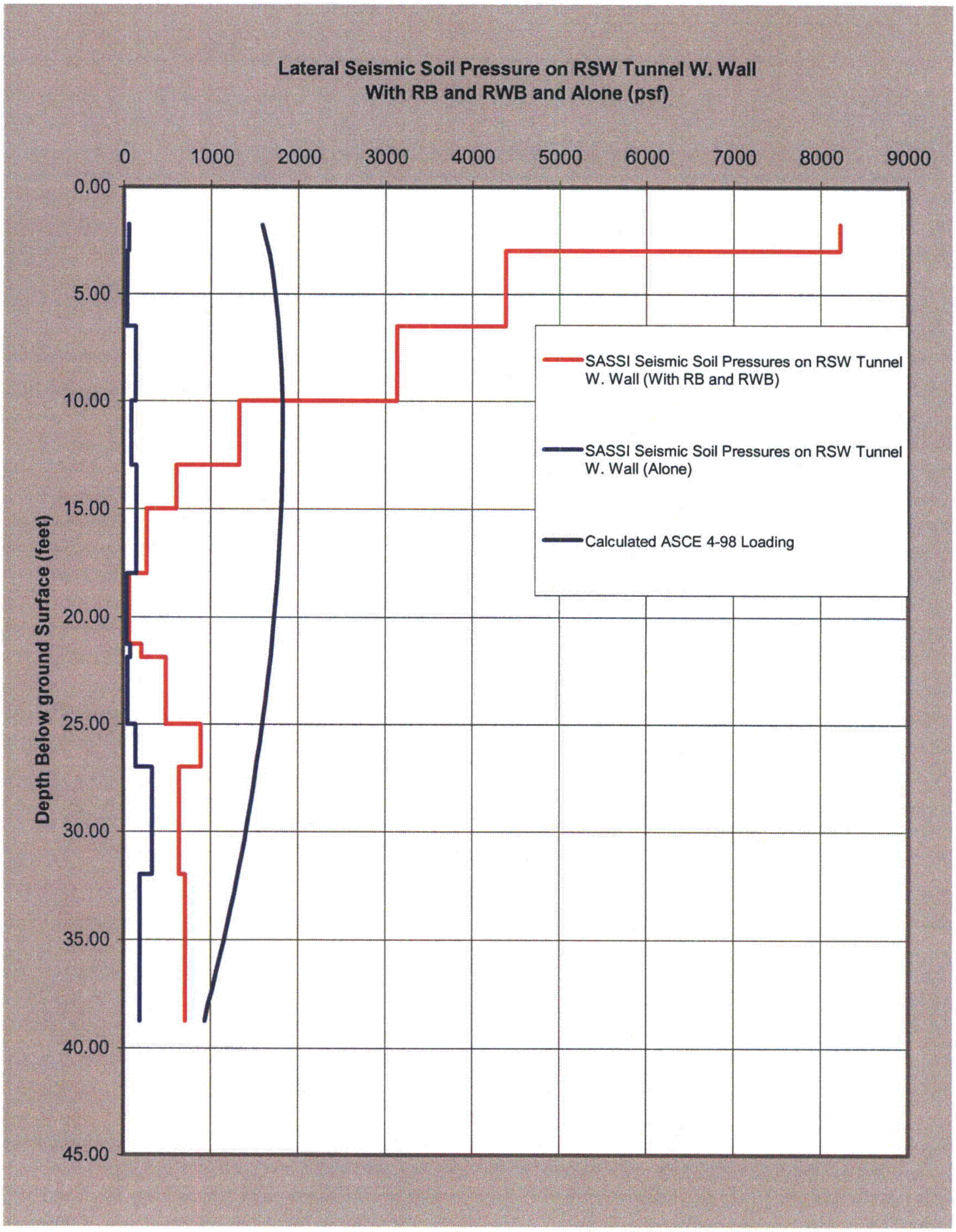


Figure 3H.6-213: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel West Wall (Main Cross Section of RSW Piping Tunnel)

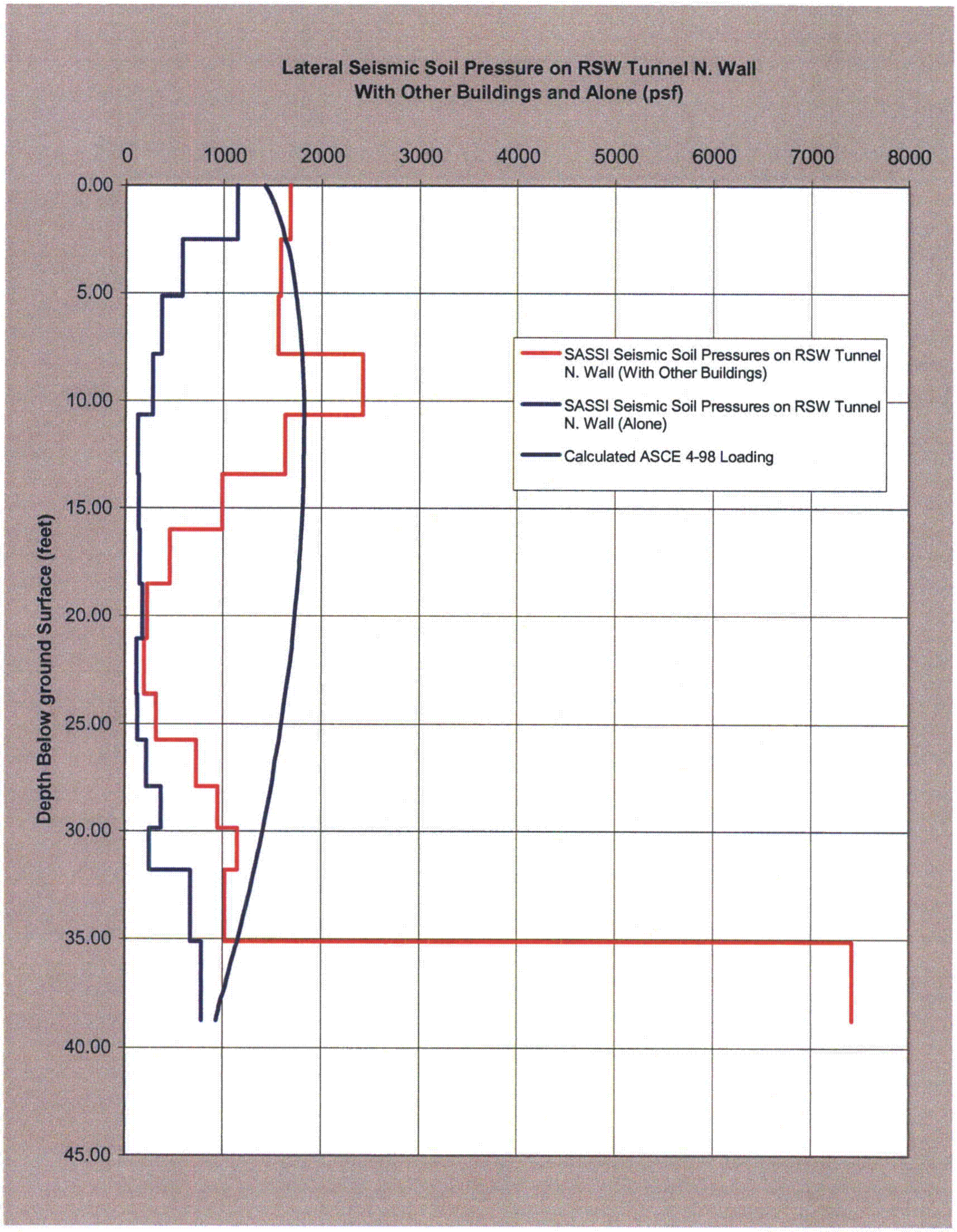


Figure 3H.6-214: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel North Wall (RSW Piping Tunnel near UHS/RSW Pump House)

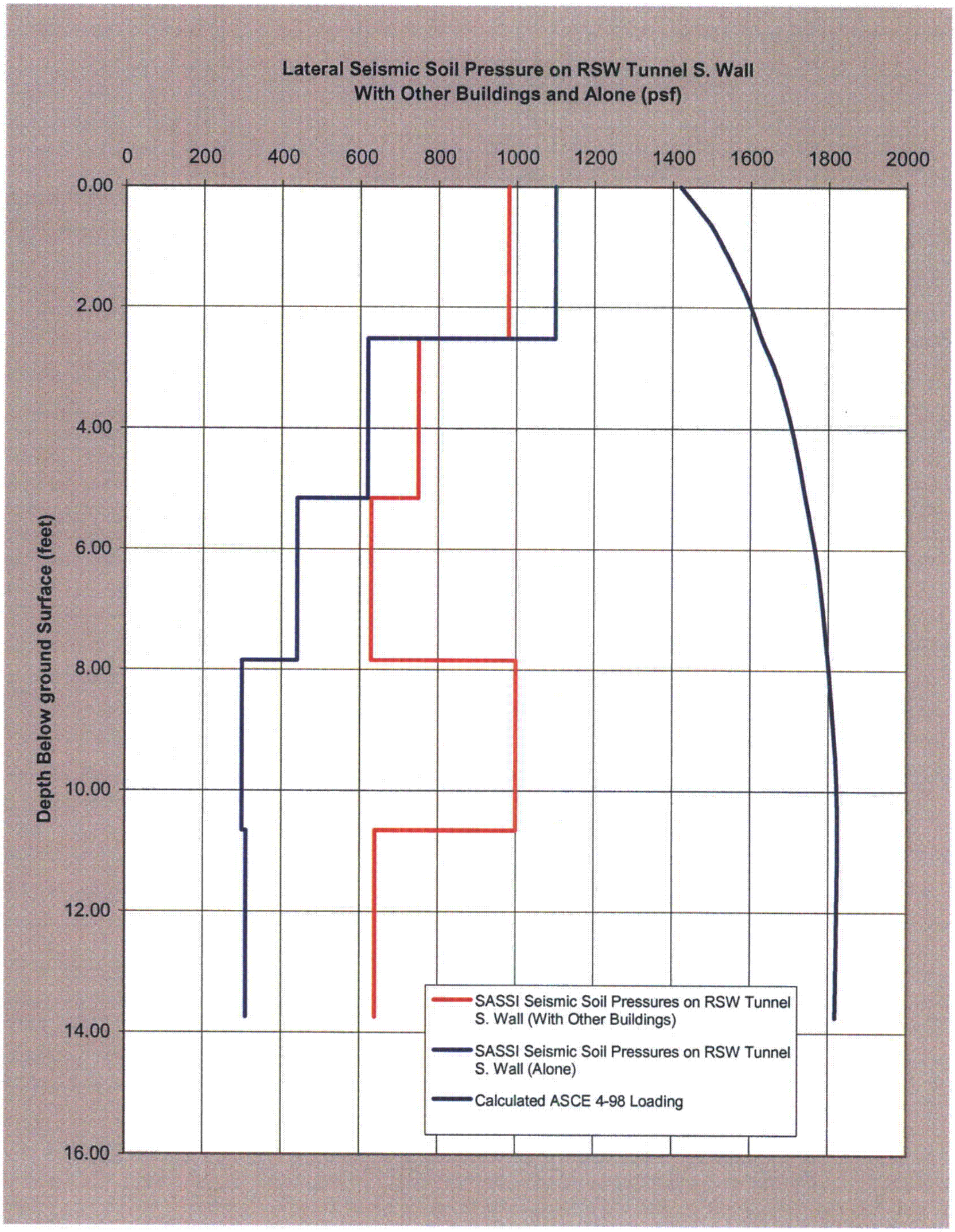


Figure 3H.6-215: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel South Wall (RSW Piping Tunnel near UHS/RSW Pump House)

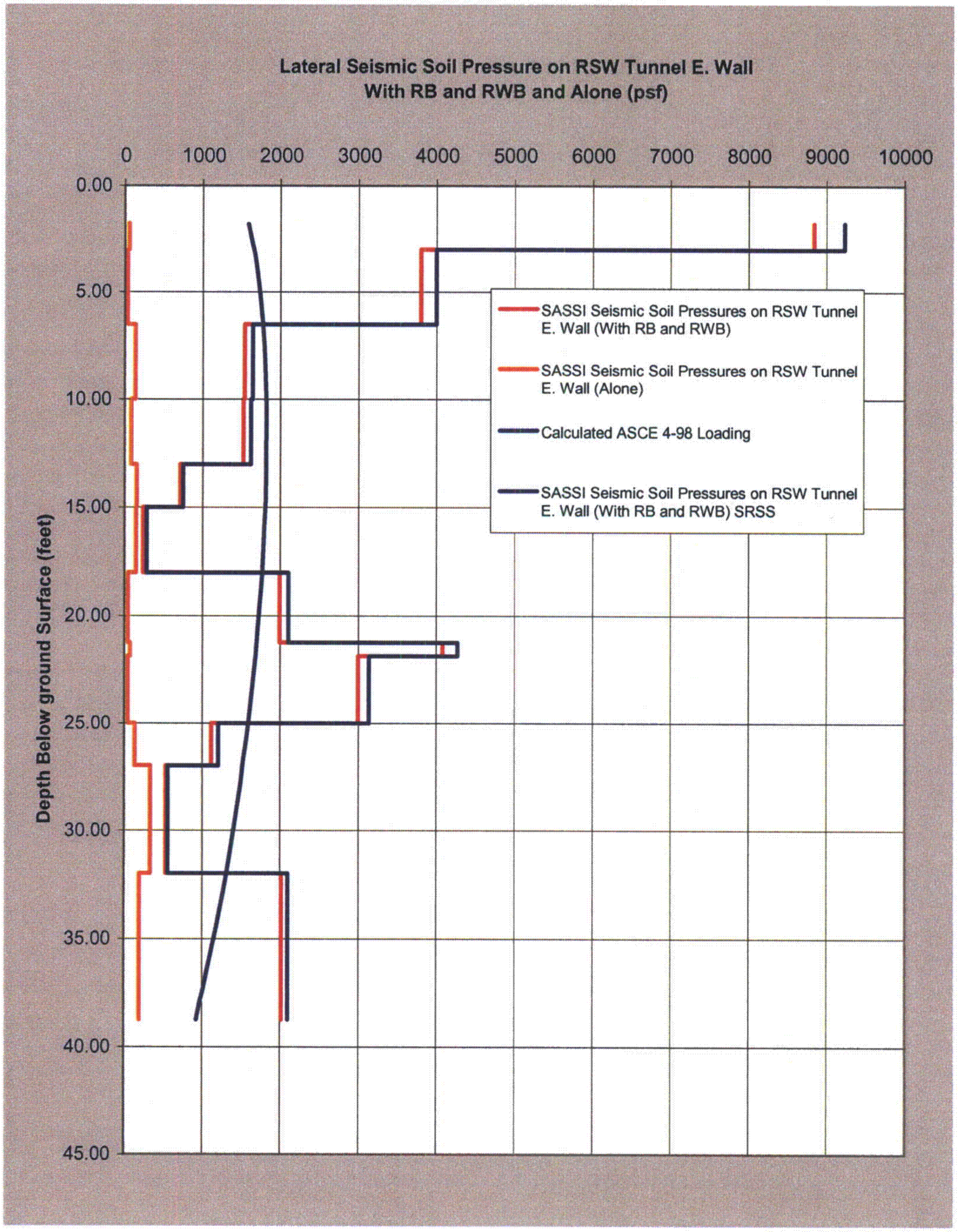


Figure 3H.6-216: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel East Wall (Main Cross Section of RSW Piping Tunnel, Including Effect of Vertical Excitation)

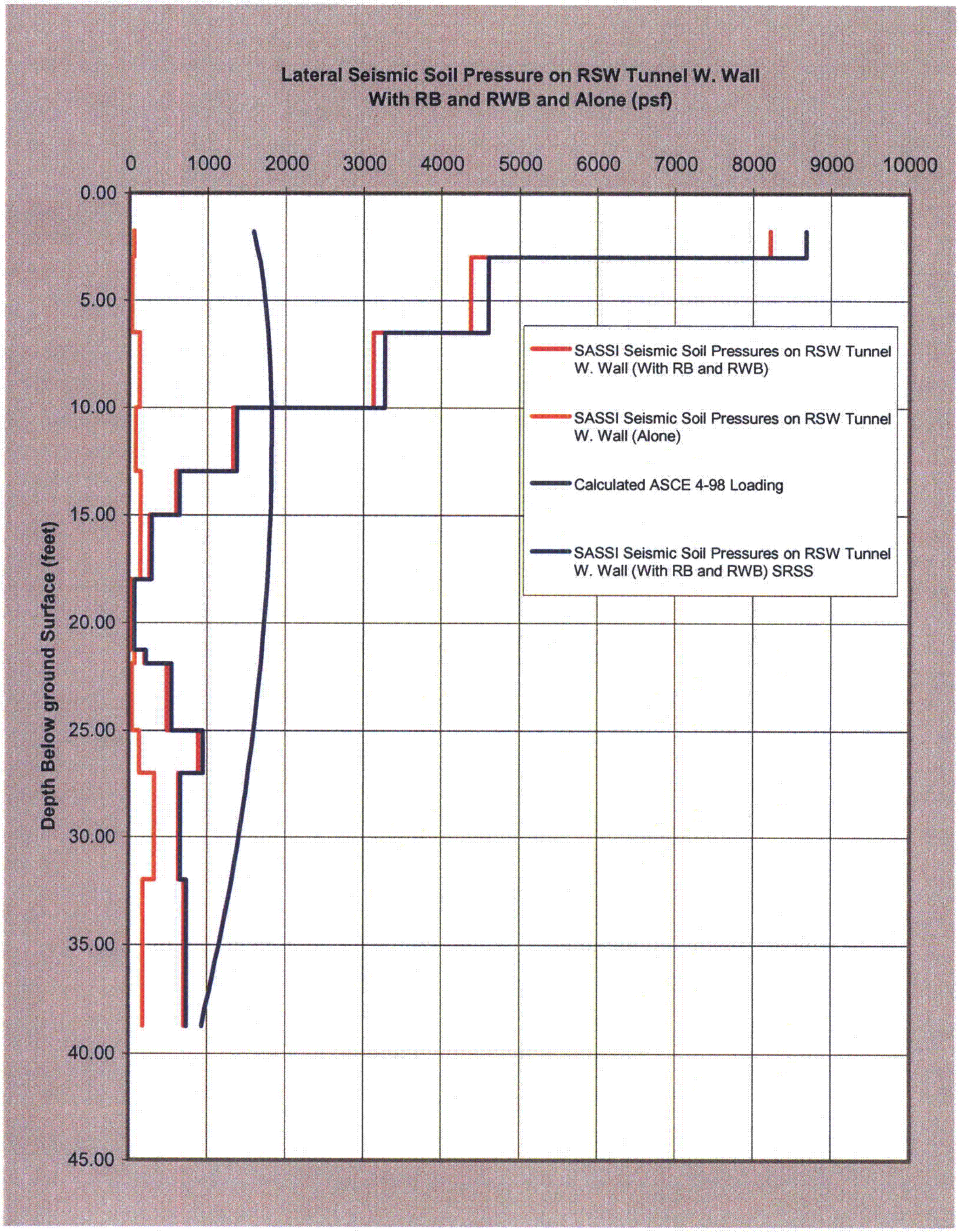


Figure 3H.6-217: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel West Wall (Main Cross Section of RSW Piping Tunnel, Including Effect of Vertical Excitation)

RAI 03.08.01-4, Revision 2

QUESTION:

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 doesn't effect 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.

REVISED RESPONSE:

This response is completely superseded by Revision 2 of the response to RAI 03.08.01-7, which is being submitted concurrently with this response.

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

RAI 03.08.01-7, Revision 2

QUESTION:

Follow-up question to Question 03.08.01-4 (RAI 2962)

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:

(1) 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.

(2) 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.

(3) 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.

REVISED RESPONSE:

Revision 1 response to RAI 03.08.01-7 was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. This revised response completely supersedes that response. This response also incorporates the response to RAI 03.08.01-4 Revision 1 submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010 into this response for simplicity and clarity. The revised portions of the response are marked with revision bars in the right margin. This revision has been updated to incorporate items discussed during the NRC's audit held during the week of October 18, 2010, which include:

- The flood water density change from 62.4 pcf to 63.85 pcf to include the weight of sediment.
- The clarification that the Design Basis Flood elevation is 40 feet MSL rather than the calculated flood level of 38.8 feet MSL
- The comparison of the shear and moment demands of the above grade walls for the Control Building (CB) and the Reactor Building (RB) based on the flooding conditions and tornado conditions.

(1) Comparison of Out-of-Plane Shear and Moment Demands

The following loading information is based on the Main Cooling Reservoir (MCR) embankment breach analysis results provided in Revision 1 of the response to RAI 03.04.02-11that is being submitted concurrently with this response:

- The design flood level is conservatively established at elevation 40 ft.
- Maximum hydrodynamic drag force due to flood water flow is 44 pounds per square foot of the projected submerged area.
- Hydrodynamic forces due to wind generated wave forces are as shown in Figure 3.4-1 provided in Revision 1 of the response to RAI 03.04.02-11 which is being submitted concurrently with this response.
- Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec shall be considered.

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, hydrodynamic force due to flood flow of 44 lb/ft^2, hydrodynamic force due to wind generated waves as shown in Figure 3.4-1 provided in the response to RAI 03.04.02-11, and impact due to a 500 lbs floating debris traveling at 4.72 ft/sec. These loads are only applicable to the portion above grade elevation of 34 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 the DCD flood elevation of 33 ft to site design basis flood level of 40 ft) of water head.

As shown in the following section, 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 tornado wind loads are as follows:

For Reactor Building:

- Calculated out-of-plane shear and moment demands due to MCR breach are 1.74 k/ft and 3.87 k-ft/ft, respectively.
- Out-of-plane shear and moment demands due to Tornado Wind Loads are 2.42 k/ft and 12.10 k-ft/ft, respectively.

For Control Building:

- Calculated out-of-plane shear and moment demands due to MCR breach are 1.69 k/ft and 3.63 k-ft/ft, respectively.
- Out-of-plane shear and moment demands due to Tornado Wind Loads are 2.04 k/ft and 8.64 k-ft/ft, respectively.

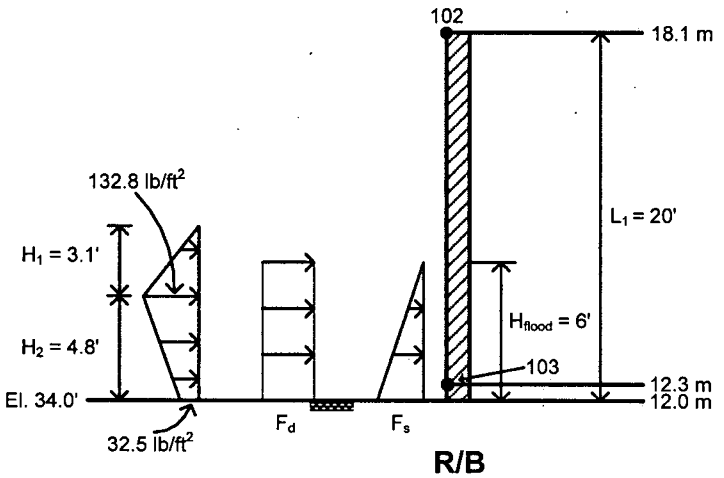
<u>Impact on the design of above grade walls</u>

Above grade exterior walls of the RB and CB are designed for tornado loading which includes tornado generated missiles. Referring to Table 5.0 of DCD, Tier 1, the maximum tornado wind speed is 483 km/h (~300 mph) and the tornado missile spectrum includes an 1800 kg (~4000 lbs) automobile with horizontal impact velocity of 169.05 km/h (i.e. 0.35 x 483 = 169.05 km/h) or about 154 ft/sec. The kinetic energy of this tornado missile is over 8,500 times the kinetic energy of a 500 lbs floating debris traveling at 4.72 ft/sec [i.e. $(4000/500)(154/4.72)^2 = 8516.2$].

Also, the shear and moment demands for the above grade exterior walls of the RB and CB for the Design Basis Flood are less than those for the tornado wind load. The calculated values are as follows:

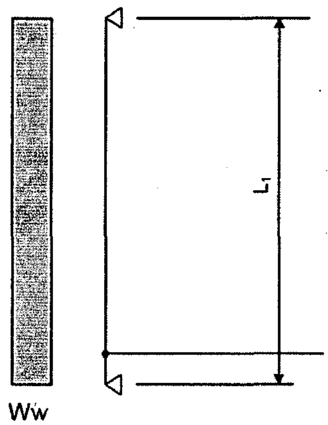
Reactor Building Above Grade

Flood Loading:
Conservatively assumed simply supported span = 20 ft (L_1)
Flood height = 6 ft (above grade)
Water density = 63.85 lb/ft^3
Hydrostatic head at grade (F_s) = 6 x 63.85 = 383.1 lb/ft^2
Hydrodynamic drag load due to flood water flow (F_d) = 44 psf (See revised response to RAI 03.04.02-11)
Hydrodynamic force due to wind generated waves (See figure below and revised response to RAI 03.04.02-11)
Calculated shear demand = 1.74 k/ft
Calculated moment demand = 3.87 k-ft/ft



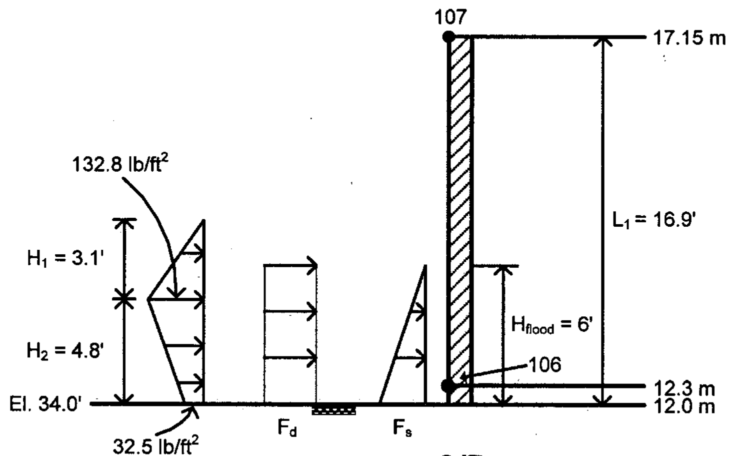
Note: For nodes 102 and 103, see DCD Figure 3A-8

Tornado Wind Loading:
Conservatively assumed simply supported span = 20 ft (L_1)
The tornado design wind velocity = 300 mph (Section 3H.1.4.2(8) of DCD)
Maximum local tornado wind pressure = 241.92 psf (W_w)
Calculated shear demand = 2.42 k/ft
Calculated moment demand = 12.10 k-ft/ft



Control Building Above Grade

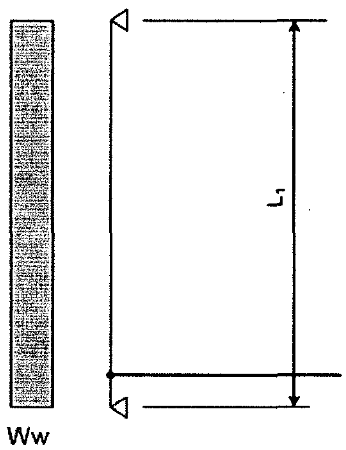
Flood Loading:
Conservatively assumed simply supported span = 16.9 ft (L_1)
Flood height = 6 ft (above grade)
Water density = 63.85 lb/ft^3
Hydrostatic head at grade (F_s) = 6 x 63.85 = 383.1 lb/ft^2
Hydrodynamic drag load due to flood water flow (F_d) = 44 psf (See revised response to RAI 03.04.02-11
Hydrodynamic force due to wind generated waves (See figure below and revised response to RAI 03.04.02-11)
Calculated shear demand = 1.69 k/ft
Calculated moment demand = 3.63 k-ft/ft



C/B

Note: For nodes 106 and 107, see DCD Figure 3A-27

Tornado Wind Loading:
Conservatively assumed simply supported span = 16.9 ft (L_1)
The tornado design wind velocity = 300 mph (Section 3H.2.4.3.3.1 of DCD)
Maximum local tornado wind pressure = 241.92 psf (W_w)
Calculated shear demand = 2.04 k/ft
Calculated moment demand = 8.64 k-ft/ft



Thus, the design of above grade exterior walls of the RB and CB for tornado wind pressure due to a wind speed of 300 mph in conjunction with tornado generated missiles bounds the design for flood loading in conjunction with impact loading due to a 500 lbs floating debris traveling at 4.72 ft/sec.

<u>Impact on the design of below grade walls</u>

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 7x63.85= 447 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 447 psf due to flood.

Based on the above, the out-of-plane flood loading on the exterior above grade walls of the RB and CB are enveloped by out-of-plane tornado loading and the out-of-plane flood loading on the exterior below grade walls of the RB and CB are enveloped by out-of-plane SSE loading. Thus, the exterior walls of the RB and CB are adequate for resisting the induced flood loads from MCR embankment breach.

(2) Impact of Increased Flood Level on Sliding and Overturning Stability:

Stability requirements for the Reactor and Control Buildings are specified in Sections 3H.1.4.5 and 3H.2.4.5 of the ABWR DCD Tier 2, respectively. These requirements are consistent with Standard Review Plan (SRP) Section 3.8.5.

Referring to SRP Section 3.8.5 as well as the above-noted DCD Tier 2 requirements, the following load combinations and acceptance criteria are applicable:

"........., the combinations used to check against sliding and overturning attributable to earthquakes, winds, tornadoes and against flotation because of floods are acceptable if found to be in accordance with the following:

A. D + H + E
B. D + H + W
C. D + H + E'
D. D + H + W_t
E. D + F'

Where D, E, W, E', and Wt are as referenced in Subsection II.3 of SRP Section 3.8.4, where H is the lateral earth pressure, and F' is the buoyant force of the design-basis flood. Justification should be provided for including live loads or portions thereof in these combinations.

Structural Acceptance Criteria. For the loading combinations referenced in the first paragraph of Subsection II.3 of this SRP section, the allowable limits that constitute the acceptance criteria are referenced in Subsection II.5 of SRP Section 3.8.1 for the containment foundation and in Subsection II.5 of SRP Section 3.8.4 for all other foundations. In addition, for the five other load combinations in Subsection II.3 of this SRP section, the factors of safety against overturning, sliding, and flotation are acceptable if found to be in accordance with the following:

Minimum Factors of Safety

For Combination	Overturning	Sliding	Flotation
a.	1.5	1.5	---
b.	1.5	1.5	---
c.	1.1	1.1	---
d.	1.1	1.1	---
e. --	---	---	1.1 "

As can be seen from the above, when considering design-basis flood, neither SRP Section 3.8.5 nor DCD require checking sliding and/or overturning. Nonetheless, even if one were to check sliding and overturning due to unbalanced forces on the Reactor and Control Buildings due to the design-basis flood (only 6 feet above grade), the unbalanced forces due to design-basis flood in comparison to the unbalanced loads due to seismic SSE will be quite negligible such that even with increased buoyant force due to additional 7 feet of water (from ground water elevation of 33 ft to design-basis flood level of 40 ft), the seismic load combination will remain as the controlling load combination for sliding and overturning of the Reactor and Control Buildings.

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 3.8.5.

(3) Update of Tables 3H.1-23 and 3H.2-5:

The COLA will be revised with the following site-specific supplemental information from DCD Tier 2, Subsections 3H.1.6, 3H.2.6 and Table 3H.1-23 and 3H.2-5 as revised below:

a. Section 3H.1.6

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, including hydrodynamic drag load due to flood water flow and hydrodynamic load due to wind generated wave action as described in Section 3.4.2, on the exterior ~~above and below grade~~ RB walls is less than the ABWR Standard Plant RB seismic or tornado loads~~,~~. The design of above grade RB exterior walls for design basis tornado loading per Tier 1 Table 5.0, including tornado generated missiles, bounds the design for flood loading including impact due to floating debris. The design of below grade RB exterior walls for design basis seismic loading bounds the design for flood loading. H~~h~~ence ~~it~~ the increased flood loading 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.

The factor of safety against floatation has been calculated and is shown in revised Table 3H.1-23.

Table 3H.1-23 Factors of Safety for Foundation Stability*

Load Combination	Overturning		Sliding		Floatation	
	Req'd.	Actual	Req'd.	Actual	Req'd.	Actual
D + F'					1.1	2.24
D + Lo + F + H+ E_{ss}	1.1	490	1.1	1.11		

Here:
F = Buoyant Forces from Design Ground Water (0.61m Below Grade)
F' = Buoyant Forces from Design Basis Flood (1.83m Above Grade)
H = Lateral Soil Pressure
L_o = Live Load Acting During an Earthquake (Zero Live Load is Considered).
E_{ss} = SSE Load
D = Dead Load

b. Section 3H.2.6

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, including hydrodynamic drag load due to flood water flow and hydrodynamic load due to wind generated wave action as described in Section 3.4.2, on the exterior ~~above and below grade~~ CB walls is less than the ABWR Standard Plant seismic or tornado loads ~~.~~; The design of above grade CB exterior walls for design basis tornado loading per Tier 1, Table 5.0, including tornado generated missiles bounds the design for flood loading including impact due to floating debris. The design of below grade CB exterior walls for design basis seismic loading bounds the design for flood loading. ~~H~~hence ~~it~~ the increased flood loading does not 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.

The factor of safety against floatation has been calculated and is shown in revised Table 3H.2-5.

Table 3H.2-5 Stability Evaluation–Factors of Safety

Load Combination	Overturning		Sliding		Flotation	
	Required	Actual	Required	Actual	Required	Actual
D+F'	–	–	–	–	1.1	1.30
D+F+H+W	1.5	2.79	1.5	2.74	–	–
D+F+H+W_t D+L_o+F+H'+E'**	1.1 1.1	2.66 123*	1.1 1.1	2.69 1.14	– –	– –

* Based on the energy technique
** Zero live load is considered.
F' = Buoyant Forces from Design Basis Flood (1.83m Above Grade)

RAI 03.08.01-9, Revision 1

QUESTION:

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 lood water, etc., and the orresponding design values used.

    b. The applicant defined pressure load 'P' as hydrostatic or differential pressure, and used t in several loading combinations. Please explain why only pressure load 'P' need to be onsidered 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.

REVISED RESPONSE:

The original response to RAI 03.08.01-09, submitted with STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010, is completely superseded by this revised response. The revisions are indicated by revision bars in the margin. This revision, based on the discussion and comments in the NRC audit held during the week of October 18, 2010, includes the following requirements:

- The interior redundant water stops are to be Seismic Category I components.
- The testing program will demonstrate that the seal material can withstand ±25% movement in any resultant direction (due to settlement) and still be watertight.
- Testing will ensure that the seal material will function as a watertight barrier following Safe Shutdown Earthquake (SSE).
- The water stop on the interior side of the joint will be tested to withstand the SSE maximum displacements without degradation.

1. The watertight door between the Control Building and the Radwaste Building Access Corridor shown in response to RAI 03.08.01-6, submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010, was deleted in the revised response to RAI 03.08.01-6, submitted with STPNOC letter U7-C-STP-NRC-100154 dated June 29, 2010. Therefore, the sketch provided in response to RAI 03.08.01-6 was removed in the revised response to RAI 03.08.01-6 and no FSAR revision is required to include this door.

2a. It is acknowledged that the load combinations in ANSI/AISC N690 and ACI 349 do not specifically address flood loads. However, Section R9.2.7 of the Commentary to ACI 349-97 states that

"Apart from the extreme environmental loads generated by the safe shutdown earthquake and by the design basis tornado, other extreme environmental loads may also be required for the plant design. Examples of such loads are those induced by flood, aircraft impact, or an accidental explosion.

These environmental loads should be treated individually in a manner similar to the loads generated by the design basis tornado in determining the required strength according to the equations in Section 9.2.1. Abnormal loads are not considered concurrently with the above extreme environmental loads."

The controlling flood at STP 3&4 site is due to the Main Cooling Reservoir dike breach. This load is considered to be an extreme environmental load, and therefore is treated as described in Section 9.2.7 of ACI 349-97. Consistent with Section 9.2.7 of ACI 349-97, the load factors are taken as 1.0.

The COLA markup provided with RAI 03.04.02-6, submitted with STPNOC letter U7-C-STP-NRC-100154 dated June 29, 2010 included the following load combination for flooding:

$1.6S = D + P + E'$

In this load combination P included the load due to the flood. The load combinations will be revised as follows:

$S = D + W + P_o$
$1.6S = D + E' + P_o$
$1.6S = D + W_t + P_o$
$1.6S = D + FL + P_o$

Where:

S = Normal allowable stresses as defined in AISC N690
D = Dead loads
P_o = Normal Operating Differential Pressure
E' = Loads generated by SSE, per Sections 3H.1 and 3H.2.
FL = Design basis extreme flood loads, including the hydrostatic load due to flood elevation at 40 ft MSL, the associated drag effects of 44 psf, hydrodynamic load due to wind-generated wave action per Figure 3.4-1, and impact due to floating debris per Section 3.4.2. The weight of the water (above ground) due to the flood loads shall be 63.85 pcf in order to include the effects of suspended sediments in

the water. (Figure 3.4-1 and revised Section 3.4.2 are included in the revised response to RAI 03.04.02-11, Revision 1, which is being submitted concurrently with this response).

W = Normal wind loads, per DCD Sections 3H.1 and 3H.2

W_t = Tornado loads per DCD Sections 3H.1 and 3H.2, including wind velocity pressure W_w, differential pressure W_p, and tornado-generated missiles (if not protected) W_m

2b. With the revised load combinations and load definitions provided in 2a. above the question related to definition of P and flood loads is answered. Drag load and load from debris transported by flood load is considered, as discussed above.

3. For the site-specific Diesel Generator Fuel Oil Storage Vault the applicable version of ANSI/AISC N690 is 1994 with Supplement 2 in accordance with the Standard Review Plan (SRP) Section 3.8.4, Revision 2 (the revision applicable to site-specific structures). COLA Table 1.8-21a will be revised to include this revision of the Code for site-specific application, as shown in the response to RAI 03.08.04-33, which was submitted in STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. For the Reactor and Control Building, the applicable version of ANSI/AISC N690 is 1984, as listed in DCD Table 1.8-21. These versions will be used in the design of the doors, as applicable.

4. The watertight doors will be designed by vendors in accordance with specific requirements given in the procurement specification. The procurement specification will include the requirement that the detailed analysis and design comply with the requirements of applicable revision of SRP Section 3.8.4 and AISC N690. The seismic loads will be determined using the applicable response spectra. The method of analysis for evaluation of seismic and other reactor building vibratory loadings, if applicable, will be the static equivalent method as described in DCD Section 3.7.3.8.1.5.

5a. The criterion for Type 2 closure is to allow slight seepage during the hydrostatic pressure conditions of flooding. Specifically, the requirements for Type 2 Closures are defined in U.S. Army Corps of Engineers (COE) EP 1165-2-314 Section 701.1.2 and requires that the closure:

"shall form essentially dry barriers or seals, allowing only slight seepage during the hydrostatic pressure conditions of flooding to the RFD."

There are less than 1000 linear feet of gasket material for all the watertight doors used for protection against external flooding. A leakage rate of 1/10 gallon per linear foot of gasket per hour equates to 100 gallons/hour or 0.006 m^3/min. The allowable leakage of 1/10 gallon per linear foot of gasket per hour is far less than the 1.34 m^3/min accepted for internal flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for internal flooding in

the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. The safety related equipment potentially subjected to external flooding is protected by curbs and raised equipment pads, similar to the safety related equipment potentially subjected to internal flooding.

5b. During the test, the hydrostatic head will be raised at a rate not more than 1 ft/min to a level of 25% higher than the flood level. Any leaks that occur during this time will be detected and if the leakage rate begins to diminish as the hydrostatic head increases, the assembly will be tested at a lower hydrostatic head. This requirement is added to the COLA Revision 4 markup provided in the revised response to RAI 03.04.02-6, Revision 2, being submitted concurrently with this response.

5c. The revised responses to RAI 03.04.02-6, Revision 2 (submitted concurrently with this response) and RAI 19-30 (submitted with STPNOC letter U7-C-STP-NRC-100119 dated May 27, 2010) now state that all doors that protect against the design basis flood will be normally closed. For requirements pertaining to inspection and maintenance, see the response to RAI 03.04.01-6 submitted with STPNOC letter U7-C-STP-NRC-090045 dated May 13, 2009.

6. The joint seals between the Reactor Building and the Control Building below the design basis flood level will be made using a polyurethane foam impregnated with a waterproof sealing compound between the concrete surfaces and an interior redundant water stop. The testing program will demonstrate that the seal material can withstand ±25% movement in any resultant direction (due to settlement) and still be watertight. Testing will also ensure that the seal material will function as a watertight barrier following SSE. During SSE maximum displacements, the water stop on the interior side of the joint will function to resist any significant water leakage. The water stop will be tested for its capacity to withstand the SSE maximum displacements without degradation.

   The lowest required watertight joint seal is in the slab at nominal elevation 4.8m (the lowest elevation of the Clean Access Corridor between the Reactor Building and Control Building) and the hydrostatic head associated with this watertight joint seal is not anticipated to exceed 35 ft. The watertight joint seal and interior redundant water stops used to protect the safety-related buildings against external water entry are classified as Seismic Category I with respect to their ability to remain in-place to stop significant water leakage into the safety-related buildings during and after a seismic event. The gap size is determined based on the displacement under a SSE load plus long-term settlement, similar to the joints discussed in RAI 03.08.04-25, submitted with STPNOC letter U7-C-STP-NRC-100108 dated May 13, 2010. Movements of ±25% of the gap size will envelope any expected displacements anticipated under normal settlement loading. This will show that the watertight joint seal material is capable of being watertight after the effects of long-term settlement and tilt, as well as during normal operating vibratory loads, such as SRV actuation. Although this will provide margin to accommodate additional

differential displacements from the majority of the movements from short duration extreme environmental loading, such as SSE and tornado, the watertight joint seals need not be designed to be watertight during the differential displacements from these extreme environmental loadings. For these events, the interior redundant water stop will act as a water-resistant barrier, which will only allow slight leakage during the event. Because of the interior water stop, leakage during local seal failure due to extreme environmental loading events will be less than the 1.34 m^3/min accepted for flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for flooding in the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. An in-service inspection program will ensure that the watertight joint seals and interior water stops do not significantly degrade during normal plant operation and after being subjected to an extreme environmental loading event. This will ensure that the watertight joint seals and interior water stops adequately protect safety-related equipment from significant leakage of water into the Reactor Building and Control Building. The requirements discussed above are added to the COLA Revision 4 markup provided in response to RAI 03.04.02-6, Revision 2.

The markups to COLA Revision 4 resulting from this response are included in the revised COLA markup included in the revised response to RAI 03.04.02-6, Revision 2, being submitted concurrently with this response. No additional COLA revision is required as a result of this response.

RAI 03.08.01-10, Revision 1

QUESTION:

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.

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.

REVISED RESPONSE:

The original response to RAI 03.08.01-10 was submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010. This revised response completely supersedes that response. The revisions are indicated by revision bars in the margin. This revision incorporates the change to the flood water density to include sediment based on discussions in the NRC audit held during the week of October 18, 2010.

1) In order to account for impact of floating debris, guidance provided in Section C5 of the Commentary to ASCE 7-05 was used. Based on this, impact due to a floating piece of debris weighing 500 lbs and traveling at maximum flood water velocity of 4.72 ft/sec is considered. For evaluation of effect of flooding and floating debris, please see RAI 03.08.01-7 Revision 2 response, being submitted concurrently with this response.

The flood water density, considering maximum sediment concentration, is 63.85 pounds per cubic foot (pcf) per COLA Section 2.4S.4.2.2.4.3. The density of 80 pcf noted in response to RAI 03.08.04-22 was a conservatively assumed value. This value was revised to 63.85 pcf in the revised response to RAI 03.08.04-22, Revision 1, submitted in STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010.

2. The load combination used for flood loading is based on requirements of Section 9.2.7 of ACI 349-97 shown below:

   "9.2.7 If resistance to other extreme environmental loads such as extreme floods is specified for the plant, then an additional load combination shall be included with the additional extreme environmental load substituted for ***Wt*** in Load Combination 5 of 9.2.1"

3. The reported shear and moment demands in the original response to RAI 03.08.01-7 submitted with STPNOC letter U7-C-STP-NRC-100018 dated January 14, 2010 were conservatively calculated considering a uniform loading of 418.4 psf for the entire flood height of 6 ft (i.e. 374.4 psf due to 6 ft water head plus 44 psf due to drag load due to flood water). Please see Revision 2 response for RAI 03.08.01-7, submitted concurrently with this response, for the latest calculated shear and moment demands due to flood loading, including hydrodynamic loads due to wind generated waves.

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