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Sent: Wednesday, February 10, 2010 6:53 PM
To: Muniz, Adrian; Dyer, Linda; Wunder, George; Tonacci, Mark; Eudy, Michael; Kallan, Paul; Plisco, Loren; Anand, Raj; Foster, Rocky; Joseph, Stacy; Govan, Tekia; Tai, Tom
Subject: Transmittal of Letter U7-C-STP-NRC-100036
Attachments: U7-C-STP-NRC-100036.pdf

Please find attached a courtesy copy of letter number U7-C-STP-NRC-100036, which contains responses to the NRC staff questions included in Request for Additional Information (RAI) letter numbers 299 and 302 related to Combined License Application (COLA) Part 2, Tier 2, Sections 3.7.1, 3.7.2 and 3.8.4.

The official version of this correspondence will be placed in today's mail. Please call John Price at 972-754-8221 if you have any questions concerning this letter.

Thank you,

Loree Elton
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Hearing Identifier: SouthTexas34Public_EX
Email Number: 1998

Mail Envelope Properties (C7F098E3C31A0141A02043F0B8E656EE268573DBF8)

Subject: Transmittal of Letter U7-C-STP-NRC-100036
Sent Date: 2/10/2010 6:52:59 PM
Received Date: 2/10/2010 6:53:55 PM
From: Elton, Loree

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Options
Priority: Standard
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Nuclear Operating Company

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February 10, 2010
U7-C-STP-NRC-100036

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
Response to Request for Additional Information

Attached are the responses to the NRC staff questions included in Request for Additional Information (RAI) letter numbers 299 and 302 related to Combined License Application (COLA) Part 2, Tier 2, Sections 3.7.1, 3.7.2 and 3.8.4.

Attachments 1 through 14 address the responses to the RAI questions listed below:

RAI 03.07.01-20	RAI 03.08.04-17
RAI 03.07.01-24	RAI 03.08.04-18
RAI 03.07.02-13	RAI 03.08.04-19
RAI 03.07.02-14	RAI 03.08.04-22
RAI 03.07.02-15	RAI 03.08.04-23
RAI 03.07.02-16	RAI 03.08.04-25
RAI 03.07.02-18	RAI 03.08.04-27

Table 1 provided in Attachment 15 addresses the current schedule for supplemental RAI information associated with RAI letter numbers 297, 299, and 302 related to COLA Part 2, Tier 2, Sections 3.7 and 3.8. This supplemental information also includes details requested by the NRC Staff in the January 19-20, 2010 meeting.

There are no commitments in this letter.

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

STI 32609456

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

Executed on 2/10/10

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

jep

Attachments:

1. RAI 03.07.01-20
2. RAI 03.07.01-24
3. RAI 03.07.02-13
4. RAI 03.07.02-14
5. RAI 03.07.02-15
6. RAI 03.07.02-16
7. RAI 03.07.02-18
8. RAI 03.08.04-17
9. RAI 03.08.04-18
10. RAI 03.08.04-19
11. RAI 03.08.04-22
12. RAI 03.08.04-23
13. RAI 03.08.04-25
14. RAI 03.08.04-27
15. Table 1 - Supplemental Information Dates

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

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RAI 03.07.01-20

QUESTION:

(Follow-up Question to RAI 03.07.01-7)

(1) As shown in Table 2 in the response to RAI 03.07.01-7, a Poisson's ratio equal to 0.46 to 0.48 is used for calculating the soil spring constants that are used for the settlement evaluation and mat design. This high Poisson's ratio assumes that the vertical stresses transmitted to the saturated foundation soils are resisted by the incompressible pore water. Nonetheless, depending on the foundation soil permeability, the excess pore water pressures can dissipate quickly; thus, transferring the stresses to the soil grains. In light of the above, the applicant is requested to provide a comparison of the soil spring constant values, calculated using drained Poisson's ratio of foundation soils, with those of the ABWR DCD and justify any differences as to their effect on mat design forces.

(2) In the response to RAI 03.07.01-7, the applicant stated that "***The spring constant values are provided only for the Reactor Building in the DCD. Therefore, a comparison of the spring constant values is provided only for the Reactor Building.***" This justification for not evaluating the effect of site-specific shear wave velocity on the Control Building (CB) foundation design is not acceptable. The applicant is requested to further justify that the design of the CB foundation at the STP site would still be bounded by the standard plant CB design.

RESPONSE:

1a) Estimated Spring Constant for the Reactor Building (RB)

The estimated spring constant values under the mat foundation for the RB for the STP site conditions are provided in Table 03.07.01-20a below. The potential degree of variability is indicated by the spread of values from lower range to upper range. The soil properties used to compute the values in Table 03.07.01-20a are strain-compatible and were developed from the site response analyses described in COLA Part 2, Tier 2, Section 2.5S.2.5. Soil depths for the vertical and horizontal spring constant calculations are shown in Table 03.07.01-20b. Soil layers at depths greater than shown in Table 03.07.01-20b were ignored due to their insignificant contribution to the spring values.

The equations for the soil spring constant values in Gazetas (Reference 1) require a single value of shear modulus (and Poisson's ratio) as input. For the layered conditions such as those at the STP site, the equivalent single value of shear modulus is determined using information in Christiano, et al., 1974 (Reference 2). Application of the curves and equations of Christiano, et al. to the site-specific layer values of shear modulus and Poisson's ratio yielded the single values of shear modulus and Poisson's ratio provided in Table 03.07.01-20b. The shear modulus and Poisson's ratio values in Table 03.07.01-20b were used to compute the soil spring values for the RB in Table 03.07.01-20a.

Table 03.07.01-20a: Reactor Building Foundation Spring Constants

			ν = ν undrained	ν drained = 0.30[2]	ν drained = 0.15[2]	DCD ν = 0.38
		Depth Below Grade Elevation 34 ft to Bottom of Foundation (Concrete Fill) (ft)	94.25	94.25	94.25	85.3[3]
		Foundation Width, B (ft)	187.7	187.7	187.7	186
		Foundation Length, L (ft)	197.5	197.5	197.5	196
Vertical Mode		Lower Range Vertical, k_z (kips/ft^3)	132	100	82	87
		Best Estimate Vertical, k_z (kips/ft^3)	197	149	123	
		Upper Range Vertical, k_z (kips/ft^3)	288	222	183	
Horizontal Mode	North-South Direction	Lower Range Horizontal, k_{N-S} (kips/ft^3)	94	84	77	78
		Best Estimate Horizontal, k_{N-S} (kips/ft^3)	141	126	116	
		Upper Range Horizontal, k_{N-S} (kips/ft^3)	210	189	174	
	East-West Direction	Lower Range Horizontal, k_{E-W} (kips/ft^3)	94	84	78	78
		Best Estimate Horizontal, k_{E-W} (kips/ft^3)	142	127	116	
		Upper Range Horizontal, k_{E-W} (kips/ft^3)	211	190	175	

(1) Width and length of concrete fill below Reactor Buildings.

(2) ν drained = 0.30 (sand layers); 0.15 (clay layers); layer weighted value would lie between these limits.

(3) Bottom of Basemat (DCD).

Table 03.07.01-20b: Reactor Building Depth Weighted Shear Modulus and Poisson's Ratio Values

			$\nu = \nu$ undrained	$\nu = \nu$ drained [2]	DCD $\nu = 0.38$
Depth Below Grade Elevation 34 ft to Bottom of Foundation (Concrete Fill) (ft)			94.25	94.25	85.3[4]
Foundation Width, B (ft)			187.7	187.7	186
Foundation Length, L (ft)			197.5	197.5	196
Vertical Mode	Lower Range	Shear Modulus, G (ksf)	4,185	4,185	
		Damping, β (%)	1.91	N/A	
		Poisson's Ratio, ν	0.47	0.15 to 0.30	
	Best Estimate	Shear Modulus, G (ksf)	6,245	6,245	3,732[3]
		Damping, β (%)	1.32	N/A	
		Poisson's Ratio, ν	0.47	0.15 to 0.30	0.38[3]
	Upper Range	Shear Modulus, G (ksf)	9,324	9,324	
		Damping, β (%)	0.91	N/A	
		Poisson's Ratio, ν	0.46	0.15 to 0.30	
	Depth of Soil Profile Analyzed [2] (ft)		2,500	2,500	
	Number of Soil Sublayers, n		71	71	
Horizontal Mode	Lower Range	Shear Modulus, G (ksf)	3,011	3,011	
		Damping, β (%)	2.14	N/A	
		Poisson's Ratio, ν	0.48	0.15 to 0.30	
	Best Estimate	Shear Modulus, G (ksf)	4,513	4,513	3,732[3]
		Damping, β (%)	1.51	N/A	
		Poisson's Ratio, ν	0.48	0.15 to 0.30	0.38[3]
	Upper Range	Shear Modulus, G (ksf)	6,775	6,775	
		Damping, β (%)	1.06	N/A	
		Poisson's Ratio, ν	0.47	0.15 to 0.30	
	Depth of Soil Profile Analyzed [2] (ft)		1,300	1,300	Halfspace
	Number of Soil Sublayers, n		59	59	

[1] Width and length of concrete fill below Reactor Buildings.
[2] ν drained =0.30 (sand layers); 0.15 (clay layers). Layer weighted value would lie between these limits.
[3] DCD, Section 3H.1.5.2, G = 1.821 E+04 tonnes/m^2.
[4] Bottom of Basemat (DCD).

Soil Modulus of Elasticity, $E = 2(G)(1+\nu)$

The soil spring constants for the RB in DCD Part 2, Tier 2, Section 3H.1.5.2 (vertical springs 1398/t/m/m^2 (87.27 kips/ft^3) and horizontal springs 1250 t/m/m^2 (78.04 kips/ft^3)) were computed using a shear wave velocity = 305 m/s (1000 ft/s) and a Poisson's ratio = 0.38.

The site-specific soil spring constants for upper range and best estimate conditions with drained and undrained Poisson's ratios are higher than the DCD values. For the lower range with drained Poisson's ratio of 0.15, the spring constants are nearly the same as the DCD springs, with the maximum difference of about 5% (i.e. 82 kips/ft^3 vs 87 kips/ft^3). Considering that the layer weighted value of the Poisson's ratio will be in between 0.15 and 0.3, even for the lower range and drained condition, the STP RB spring constants will be either same or higher than the DCD spring constants. This occurs even though the shear wave velocities of some of the site-specific soil layers are below the V_s value (305 m/s, or 1000 ft/s) used in the DCD to compute soil springs. The following is noted:

- The site-specific layers having V_s < 1000 ft/s are limited in thickness. The soil springs in DCD Tier 2, Section 3H.1.5.2 are calculated assuming a homogeneous elastic half-space (of infinite depth) as the supporting medium for the structure. The site-specific supporting medium for the STP Units 3 and 4 is modeled as a layered elastic half-space. Even though some of the layers have a shear wave velocity (V_s) somewhat lower than the value used in the DCD for the homogeneous half-space, the deeper layers have higher V_s and therefore exert an overcompensating effect, leading to a soil spring constant value that is higher for the site-specific layered half-space than for the homogeneous half-space of the DCD.

1b) Impact on the Reactor Building Mat Design Forces

The soil profiles considered in the ABWR DCD design range from soft soil to hard rock. The enveloping mat design for this range of soil profiles was performed considering spring constants corresponding to the softest soil (i.e. shear wave velocity 305 m/s or 1000 ft/s). Softer soil springs would result in higher mat design forces.

As noted in part (1a) of this response, the calculated STP site-specific soil spring constants are higher than the soil spring constants used for the standard design. Higher soil spring constants at the STP site will result in mat design forces smaller than those used for the ABWR RB design. Therefore, the ABWR RB mat design is adequate for the STP site.

2a) Control Building Estimated Spring Constants

The spring constant values are provided only for the RB in the ABWR DCD. The DCD soil parameters for the Control Building (CB) as specified in Section 3H.2.4.2.1 are the same as those for the RB (i.e., V_s = 305 m/s, γ = 1.92 tonnes /m^3(120 lb/ft^3), ν = 0.38, G=1.821 E+04 tonnes/m^2 (3,732 kips/ft^2)). Therefore, the best estimate spring constant for the CB is determined using the DCD soil input as a "DCD spring constant" for comparative purposes.

The estimated site-specific spring constant values under the mat foundation for the CB for the STP site conditions are provided in Table 03.07.01-20c. The potential degree of variability is indicated by the spread of values from lower range to upper range. The soil properties used to compute the values in Table 03.07.01-20c are strain-compatible and were developed from the site response analyses described in COLA Part 2, Tier 2, Section 2.5S.2.5. Soil depths for the vertical and horizontal spring constant calculations are shown in Table 03.07.01-20d. Soil layers at depths greater than shown in Table 03.07.01-20d were ignored due to their insignificant contribution to the spring values.

As can be seen from Table 03.07.01-20c, the spring constants for the best estimate and upper range exceed the best estimate DCD spring constant. The site-specific spring constants for the lower range are in general less than the best estimate DCD spring constants.

Table 03.07.01-20c: Control Building Foundation Spring Constants

			ν = ν undrained	ν drained = 0.30[2]	ν drained = 0.15[2]	DCD ν = 0.38
		Depth Below Grade Elevation 34 ft to Bottom of Foundation (Concrete Fill) (ft)	78.3	78.3	78.3	76.1[1]
		Foundation Width, B (ft)	80.1	80.1	80.1	78.7
		Foundation Length, L (ft)	185.0	185.0	185.0	183.7
Vertical Mode		Lower Range Vertical, k_z (kips/ft^3)	181	137	113	143[3]
Vertical Mode		Best Estimate Vertical, k_z (kips/ft^3)	270	205	169	
Vertical Mode		Upper Range Vertical, k_z (kips/ft^3)	403	305	251	
Horizontal Mode	North-South Direction	Lower Range Horizontal, k_{N-S} (kips/ft^3)	130	116	107	160[3]
Horizontal Mode	North-South Direction	Best Estimate Horizontal, k_{N-S} (kips/ft^3)	195	174	160	
Horizontal Mode	North-South Direction	Upper Range Horizontal, k_{N-S} (kips/ft^3)	293	262	241	
Horizontal Mode	East-West Direction	Lower Range Horizontal, k_{E-W} (kips/ft^3)	117	109	101	147[3]
Horizontal Mode	East-West Direction	Best Estimate Horizontal, k_{E-W} (kips/ft^3)	176	163	152	
Horizontal Mode	East-West Direction	Upper Range Horizontal, k_{E-W} (kips/ft^3)	264	244	228	

(1) Bottom of Basemat (DCD).

(2) ν drained = 0.30 (sand layers); 0.15 (clay layers); layer weighted value would lie between these limits.

(3) Estimated using soil input from DCD, Section 3H.2.4.2.1.

Table 03.07.01-20d: Control Building Depth Weighted Shear Modulus and Poisson's Ratio Values

			ν = ν undrained	ν = ν drained [2]	DCD ν = 0.38
Depth Below Grade Elevation 34 ft to Bottom of Foundation (Concrete Fill) (ft)			78.3	78.3	76.1[1]
Foundation Width, B (ft)			80.1	80.1	78.7
Foundation Length, L (ft)			185.0	185.0	183.7
Vertical Mode	Lower Range	Shear Modulus, G (ksf)	3,528	3,528	
		Damping, β (%)	1.99		
		Poisson's Ratio, ν	0.48	0.15 to 0.30	
	Best Estimate	Shear Modulus, G (ksf)	5,283	5,283	3,732[3]
		Damping, β (%)	1.37		
		Poisson's Ratio, ν	0.47	0.15 to 0.30	0.38[3]
	Upper Range	Shear Modulus, G (ksf)	7,869	7,869	
		Damping, β (%)	0.95		
		Poisson's Ratio, ν	0.47	0.15 to 0.30	
	Depth of Soil Profile Analyzed (ft)		1,500	1,500	
	Number of Soil Sublayers, n		65	65	
Horizontal Mode	Lower Range	Shear Modulus, G (ksf)	2,738	2,738	
		Damping, β (%)	2.01		
		Poisson's Ratio, ν	0.48	0.15 to 0.30	
	Best Estimate	Shear Modulus, G (ksf)	4,104	4,104	3,732[3]
		Damping, β (%)	1.36		
		Poisson's Ratio, ν	0.48	0.15 to 0.30	0.38[3]
	Upper Range	Shear Modulus, G (ksf)	6,158	6,158	
		Damping, β (%)	0.93		
		Poisson's Ratio, ν	0.47	0.15 to 0.30	
	Depth of Soil Profile Analyzed (ft)		700	700	Halfspace
	Number of Soil Sublayers, n		57	57	

[1] Bottom of Basemat (DCD).
[2] ν drained =0.30 (sand layers); 0.15 (clay layers). Layer weighted value would lie between these limits.
[3] DCD, Section 3H.2.4.2.1, G = 1.821 E+04 tonnes/m^2.

2b) Impact on the Control Building Mat Design Forces

The soil profiles considered in the ABWR DCD design range from soft soil to hard rock. The enveloping mat design for this range of soil profiles was performed considering spring constants corresponding to the softest soil (i.e., the shear wave velocity of 305 m/s or 1000 ft/s). Softer soil springs would result in higher mat design forces.

As noted in part (2a) of this response, the calculated STP site-specific soil spring constants for the upper range and best estimate cases are the same as, or higher than, the best estimate soil spring constants used for the standard design. Higher soil spring constants at the STP site will result in mat design forces smaller than those used for the ABWR CB design. The lower range site-specific soil spring constants are lower than the best estimate DCD spring constants. However, even with lower range site-specific spring constants, the ABWR CB mat design is adequate for the STP site for the following reasons:

- Considering the size and geometry of the CB, arrangement of the exterior and interior shear walls, thickness of the shear walls (39 inch exterior walls and 63 inch interior walls), and the mat thickness (i.e. 118 inches), the CB mat design is quite rigid and not so sensitive to spring constant values. This can be seen from the parametric study results presented in Figures 03.07.01-20a through 03.07.01-20i provided with this response.

  Figure 03.07.01-20a shows the layout of the mat and the shear walls of a structure with very similar arrangement to that of the DCD CB. The model used for this parametric study is a three dimensional finite element model. This model was analyzed twice for the total dead load of the structure along with significant seismic moment about the X-axis (along East-West), once with DCD best estimate spring constants and the second time with lower bound site-specific spring constants. Figures 03-07-01-20b through 03-07-01-20e present contour plots of the resulting out-of-plane moments and shears when using DCD spring constants. Figures 03-07-01-20f through 03-07-01-20i present contour plots of the resulting out-of-plane moments and shears when using lower range site-specific spring constants. Comparison of the resulting out-of-plane moments and shears from these two cases show that there is no significant change in mat design forces.

- The controlling load combinations for the CB mat design are shown in DCD Table 3H.2-3. As can be seen from this table, the governing load combination for the mat design is the seismic load combination. The site-specific SSE is less than half the DCD SSE (i.e., 0.13g modified Regulatory Guide 1.60 spectra vs. 0.3g Regulatory Guide 1.60 spectra).

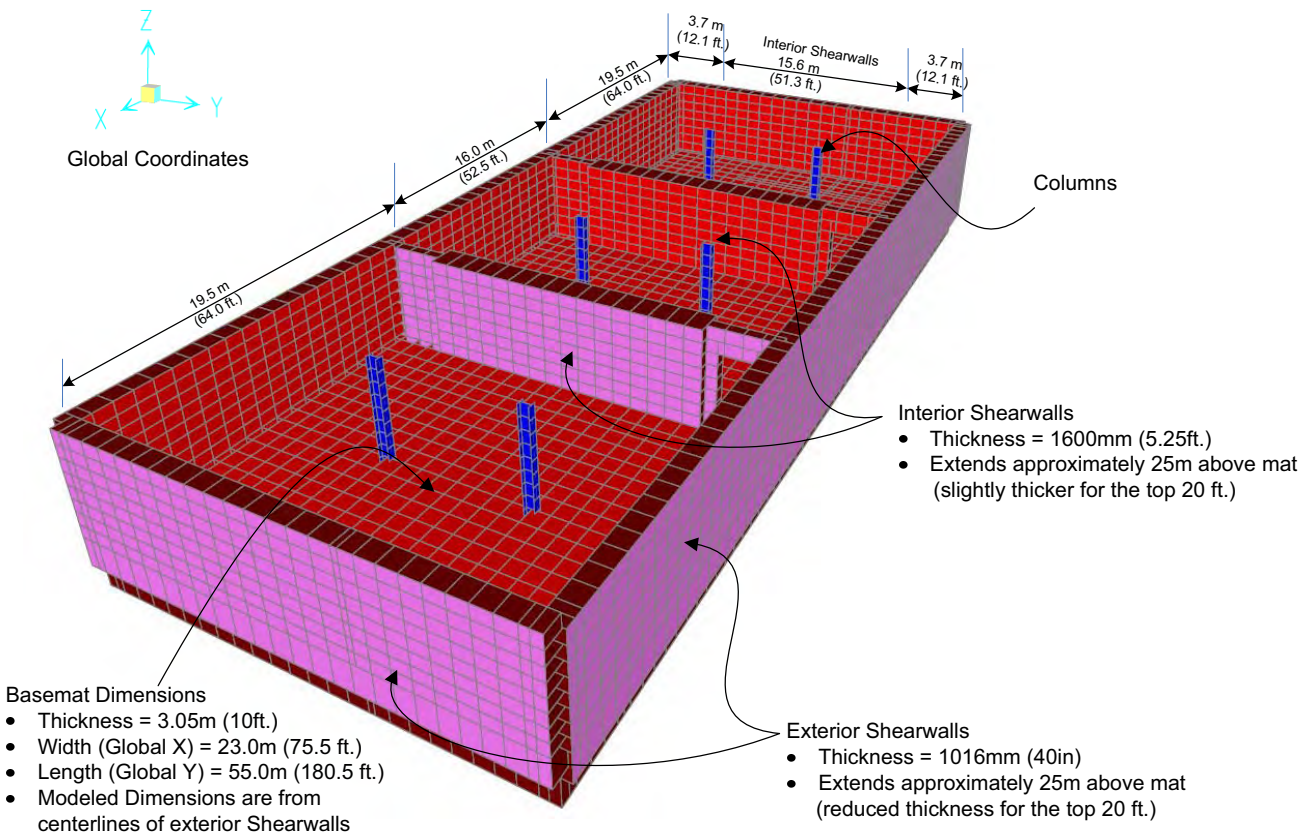
- Based on the required and provided reinforcement data in DCD Table 3H.2-3 for elements No. 200 and 66, the minimum design margin for the CB mat design is about 13.8% (i.e. 101.6/89.3 = 1.138).

REFERENCES Used in this RAI Response:

1. Gazetas, G., 1991. "Formulas and Charts for Impedances of Surface and Embedded Foundations," Journal of Geotechnical Engineering, Vol. 117, No. 9, pages 1363-1381.

2. Christiano, P.P., Rizzo, P.C., and Jarecki, S.J., 1974. "Compliances of Layered Elastic Systems," Proceedings of the Institute of Civil Engineers, Part 2, Vol. 57, December, Pages 673-683.

No COLA change is required for this response.



3-D View of Parametric Study Model

- Cross Section up to 1st Level above Mat for clarity.
  - Non Shearwalls not shown.

Figure 03.07.01-20a

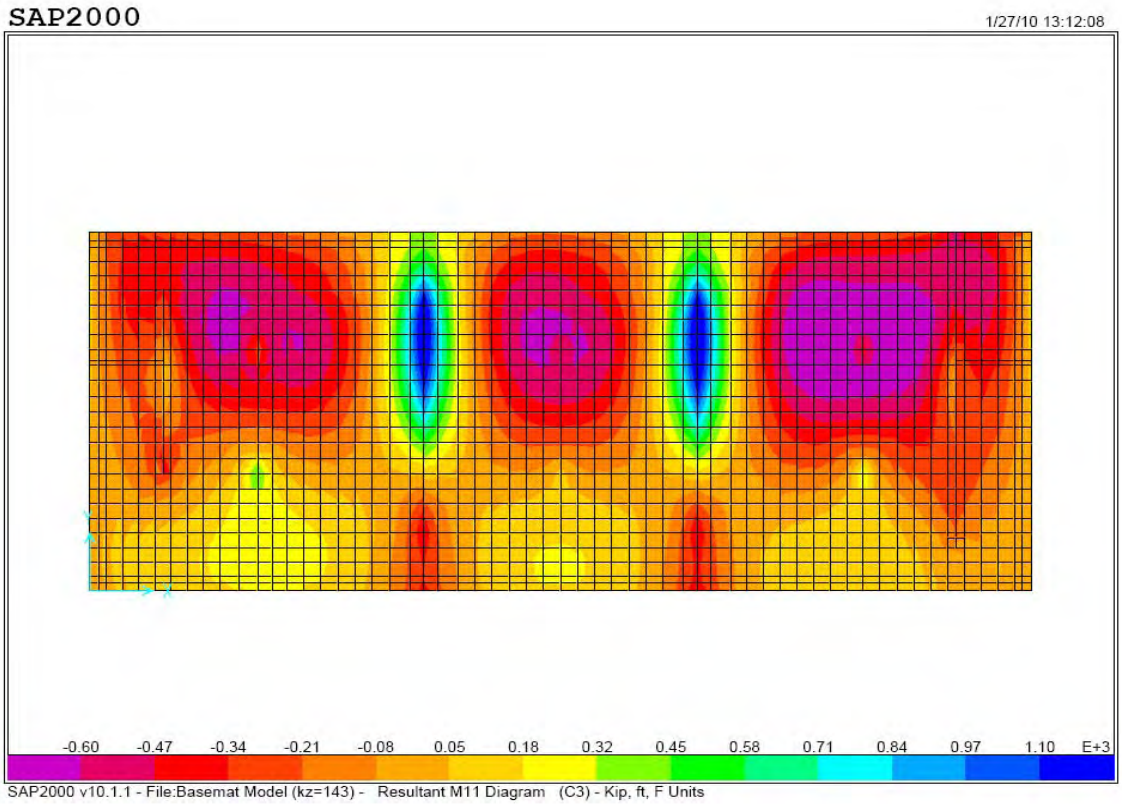


Figure 03.07.01-20b: Resultant Out-of-Plane Moment M11 Diagram
(Using DCD Spring Constants)

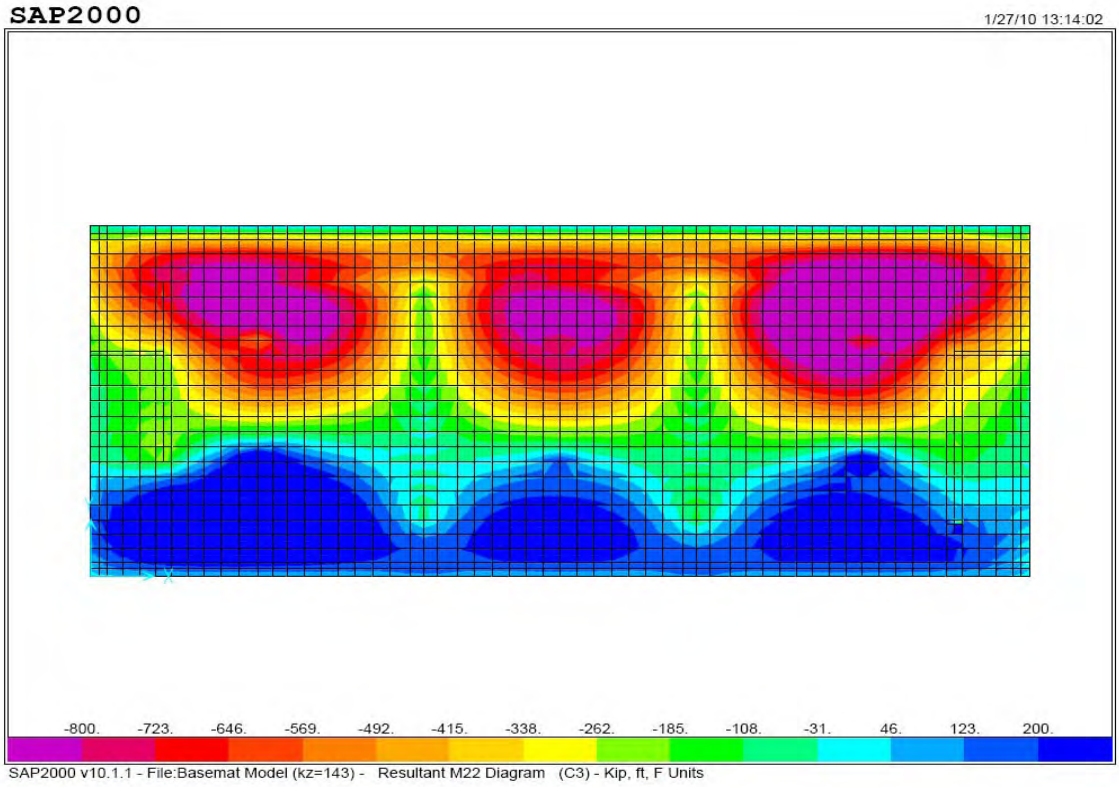


Figure 03.07.01-20c: Resultant Out-of-plane Moment M22 Diagram
(Using DCD Spring Constants)

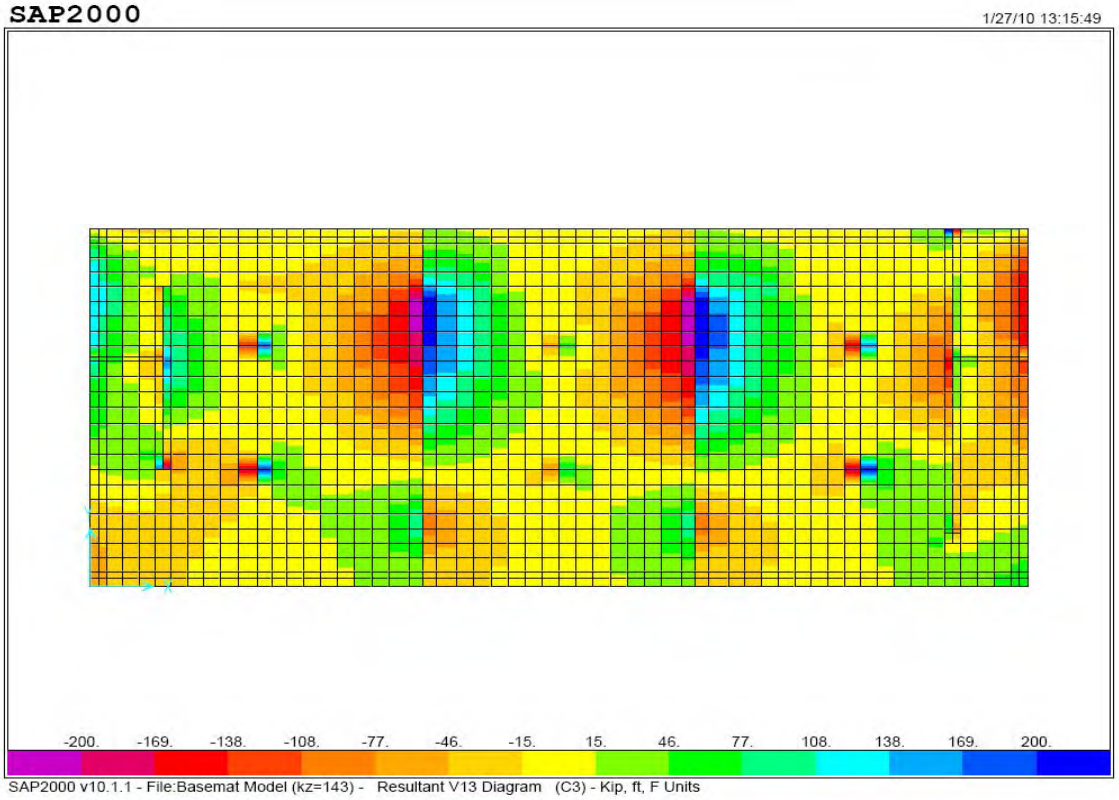


Figure 03.07.01-20d: Resultant Out-of-Plane Shear V13 Diagram
(Using DCD Spring Constants)

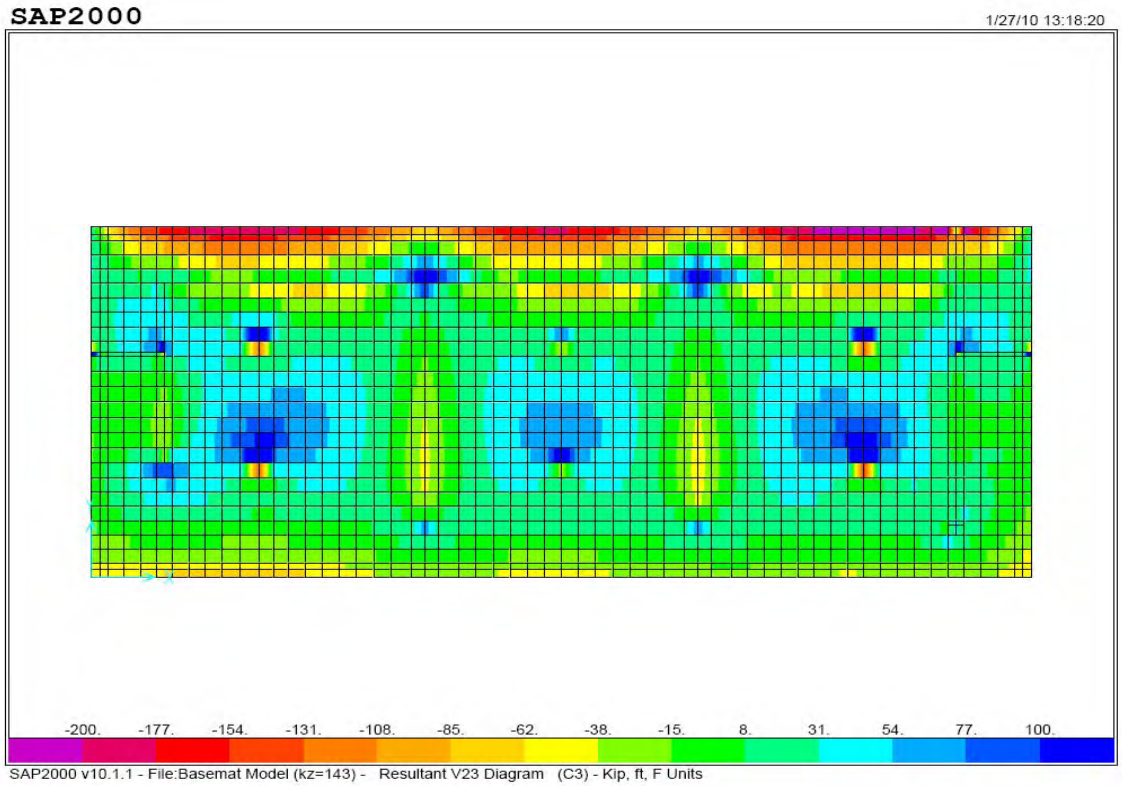


Figure 03.07.01-20e: Resultant Out-of-Plane Shear V23 Diagram
(Using DCD Spring Constants)

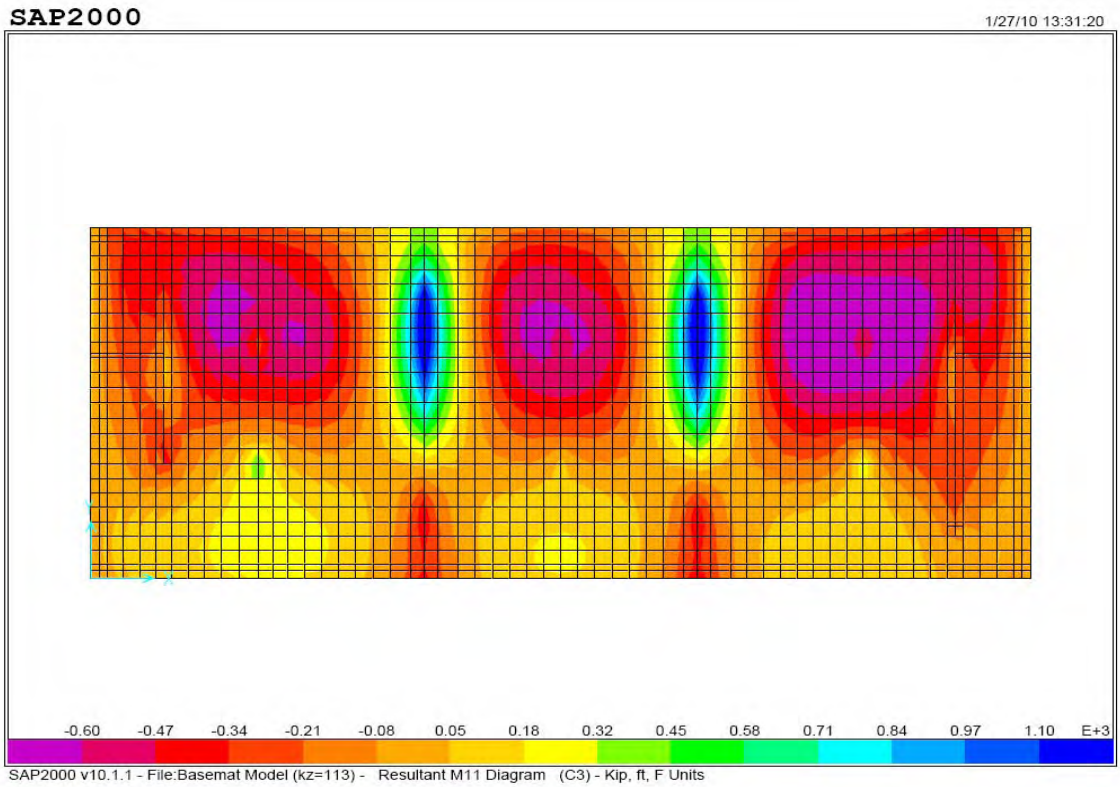


Figure 03.07.01-20f: Resultant Out-of-Plane Moment M11 Diagram
(Using Lower Range Site-Specific Spring Constants)

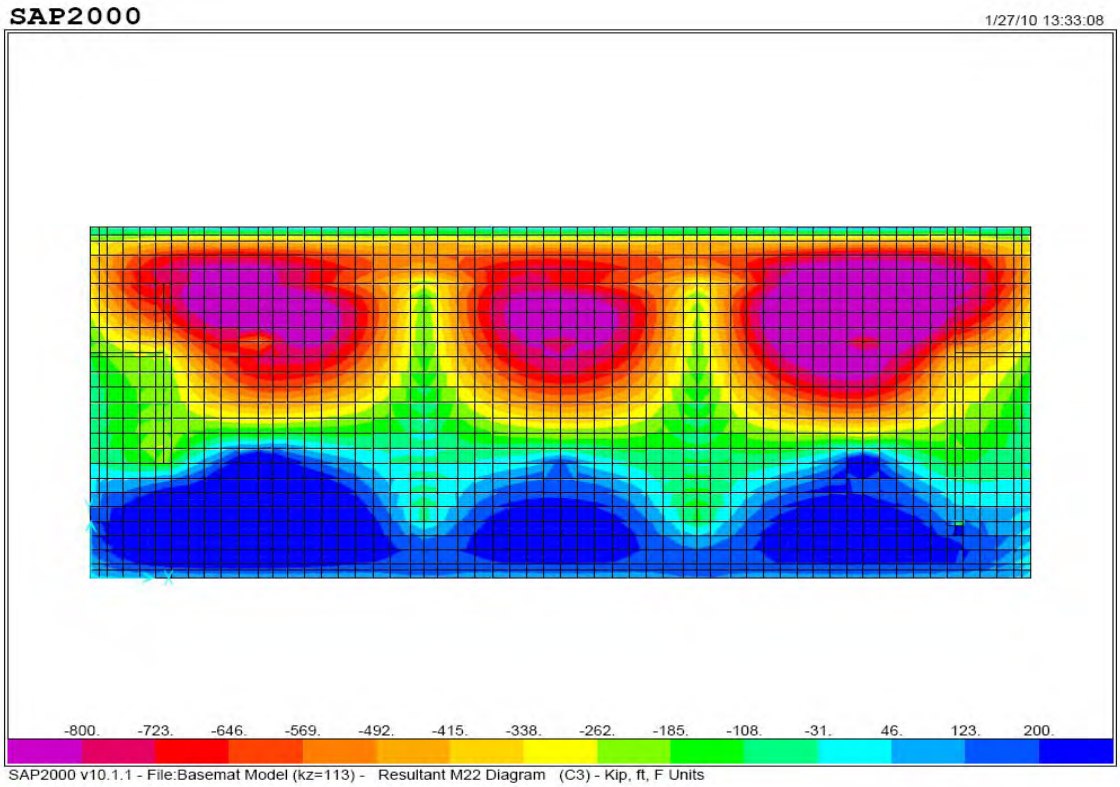


Figure 03.07.01-20g: Resultant Out-of-Plane Moment M22 Diagram
(Using Lower Range Site-Specific Spring Constants)

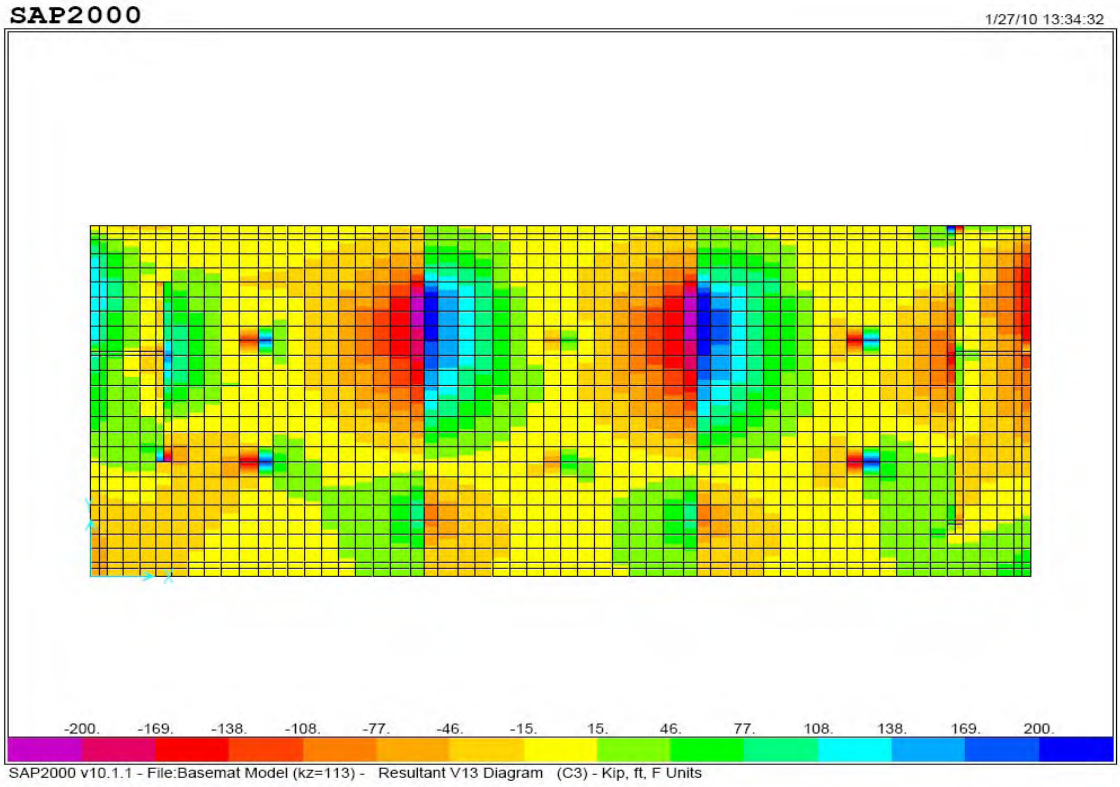


Figure 03.07.01-20h: Resultant Out-of-Plane Shear V13 Diagram
(Using Lower Range Site-Specific Spring Constants)

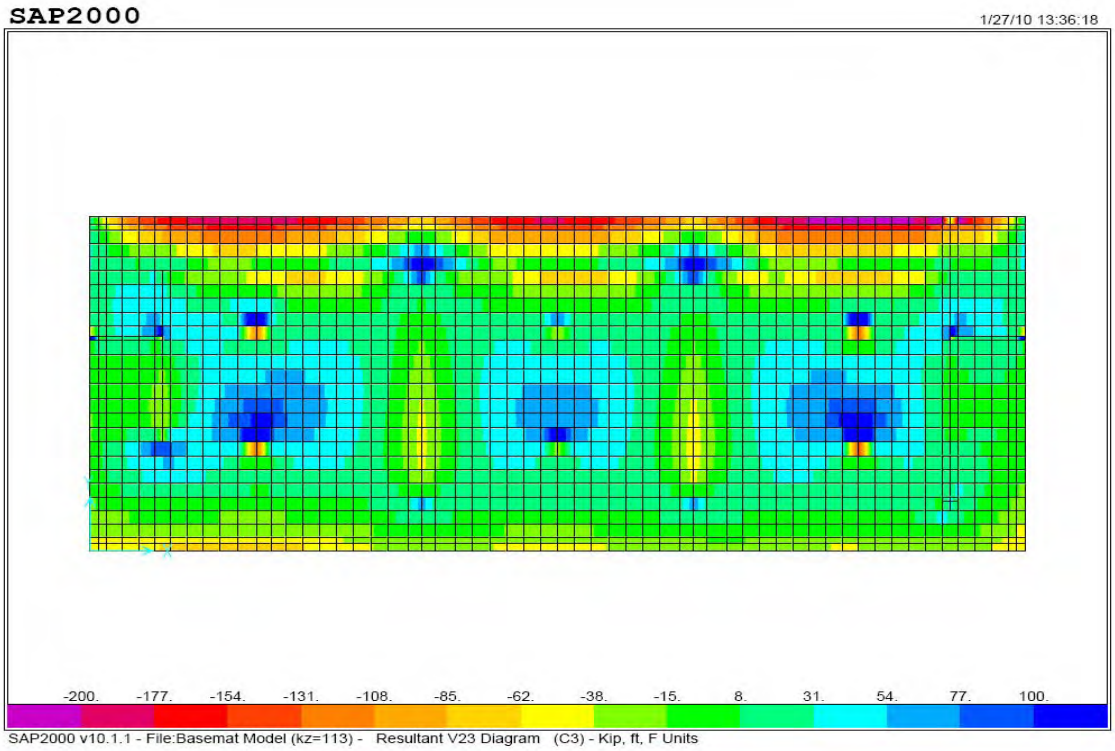


Figure 03.07.01-20i: Resultant Out-of-Plane Shear V23 Diagram
(Using Lower Range Site-Specific Spring Constants)

RAI 03.07.01-24

QUESTION:

(Follow-up Question to RAI 03.07.01-14)

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

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

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

RESPONSE:

1. We acknowledge that the seismic soil pressure on the exterior walls of the Reactor Building (RB) and Control Building (CB) could be affected by the presence of the retaining wall. However, since the site-specific safe shutdown earthquake (SSE) Input Spectra are only about 43% of the DCD SSE Spectra (i.e., 0.13g modified RG 1.60 spectra vs. 0.3g RG 1.60 spectra) and since the retaining wall is a relatively light structure, the change in the seismic soil pressure due to presence of the retaining wall will be more than offset by the reduction due to lower input motion. Fundamentally, the relatively small retaining wall is inconsequential to the massive RB and CB and has no significant effect on these structures.

2. As indicated in the Response to Item 1, above, since the site-specific safe shutdown earthquake (SSE) Input Spectra are only about 43% of the DCD SSE Spectra (i.e., 0.13g modified RG 1.60 spectra vs. 0.3g RG 1.60 spectra) the change in the seismic response of the RB and CB due to any kinematic interaction effect will be significantly enveloped by the DCD response.

SSI analysis to confirm the above conclusions is in progress. These confirmatory results will be provided by April 15, 2010.

No COLA change is required for this response.

RAI 03.07.02-13

QUESTION:

(Follow-up Question to RAI 03.07.02-1)

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

1. The FSAR mark-up in the response to item (b) of RAI 03.07.02-1, did not include the list of non- Category I structures requiring the enhanced seismic design and analysis. The applicant is requested to include in FSAR 3.7.2.8 the five identified non-Category I structures that could interact with the Category I structures.

2. The response to item (c) of RAI 03.07.02-1 indicated that non-Category I structures with the potential to interact with Category I structures have not yet progressed to a point where sliding and overturning potential as a result of the SSE can be evaluated. However, as identified in SRP guidance 3.7.2I.8., the staff must review the applicant's seismic design of these non- Category I structures. As such, the applicant is requested to provide in the FSAR factors of safety against sliding and overturning including the basis of coefficient of friction used in the analysis during an SSE for Turbine Building, Radwaste Building, Service Building, Control Building Annex, and Plant Stack.

RESPONSE:

1. As requested, COLA Part 2, Tier 2, Section 3.7.2.8 will be revised to include the five identified non-Category I structures that could interact with the Category I structures. See proposed COLA revision at the end of this response.

2. The stack located on the Reactor Building (RB) roof is an integral part of the RB roof and positively anchored to the roof. The stack and its anchorage to the RB roof are designed to withstand all applicable loads including safe shutdown earthquake (SSE). Thus, calculation of stability safety factors is not applicable to this stack.

   Stability evaluations of the four other structures are performed using the following criteria:

   - Per response to RAI 02.04.12-35 (see letter U7-C-STP-NRC-090146, dated September 21, 2009), the design maximum groundwater level is at elevation 28 ft MSL.
   - Per COLA Part 2, Tier 2, Table 2.0-2 the design flood level is at elevation 40 ft MSL.
   - For the Turbine Building, the seismic input motion is the site-specific SSE. For the Radwaste Building, Service Building and Control Building Annex, the seismic input motion shall be the amplified site-specific SSE considering the effect of nearby heavy Reactor and Control Buildings. To determine the amplified site-specific input

motion, in the SSI analyses of the RB and CB for each of these structures, five interaction nodes at the depth corresponding to the bottom elevation of the foundation are added. These five nodes correspond to the corners and center of the foundation. For each of these structures, the amplified input motion is determined by the envelope of site-specific SSE and the average response of the five nodes from the soil-structure interaction (SSI) analysis of the RB and CB for site-specific conditions.

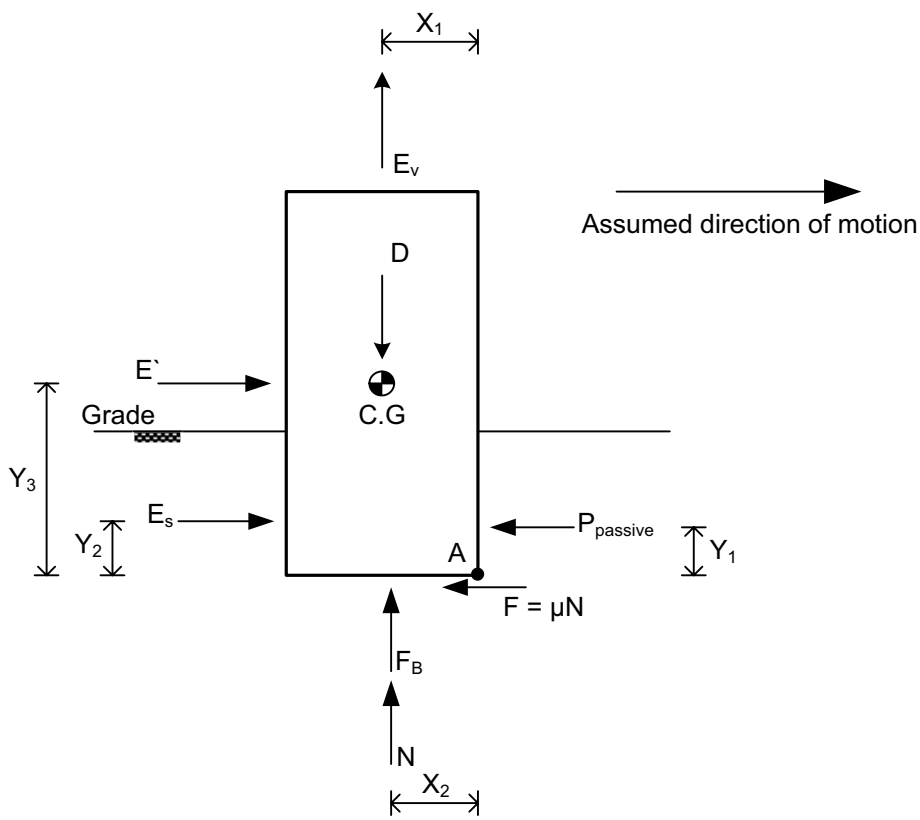
- Sliding and overturning evaluations are performed as shown in Figure 03.07.03-13a.
- Coefficient of friction for sliding evaluations for the four buildings shall be based on site-specific soil conditions.
- For simultaneous application of seismic forces in three directions the 100%, 40%, 40% combination rule as shown below will be used:

  ±100% X-excitation ±40% Y-excitation +40% Z-excitation
  ±40% X-excitation ±100% Y-excitation +40% Z-excitation
  ±40% X-excitation ±40% Y-excitation +100% Z-excitation

  Where X and Y are in the horizontal plane and Z is in the vertical direction.

Confirmation that the design meets the minimum required factor of safety for sliding and overturning and the basis for coefficient of friction for the Turbine Building, Service Building, Radwaste Building, and Control Building Annex will be provided by April 30, 2010.

Figure 03.07.01-13a



Factors of Safety against Sliding and Overturning about point A are calculated as follows:

$$SF_{sliding} = \frac{P_{passive} + F}{E_s + E`}$$

$$SF_{OT\_A} = \frac{(P_{passive})(Y_1) + (D)(X_1) - (F_B)(X_2)}{(E_s)(Y_2) + (E`)(Y_3) + (E_v)(X_1)}$$

Where:

$SF_{sliding}$ = Safety factor against sliding

SF_{OT_A} = Safety factor against overturning about "A"

D = Dead load

$P_{passive}$ = Total passive soil pressure

$F = \mu N$ = friction force and μ is the coefficient of friction

E_s = Static and dynamic soil pressure (active condition)

$E`$ = Self weight excitation in the horizontal direction

E_v = Self weight excitation in the vertical direction

F_B = Buoyancy force

N = Vertical reaction $= D - F_B - E_v$

COLA Part 2, Tier 2, Section 3.7.2.8 will be revised as shown below:

3.7.2.8 Interaction of Non-Seismic Category I Structures, Systems and Components with Seismic Category I Structures, Systems and Components

The Category I structures and their physical proximity to nearby non-Category I structures are shown in Figure 3.7-38. None of the non-Category I structures proposed as part of STP Units 3 and 4 is intended to meet Criterion (2) of DCD Section 3.7.2.8. Rather, for each non-Category I structure, either: (1) it is determined that the collapse of the non-Category I structure will not cause the non-Category I structure to strike a Category I structure; or (2) the non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions in a manner such that the margin of safety of the structure is equivalent to that of Seismic Category I structures. Non-Category I structures that can interact with Seismic Category I structures include the Turbine Building, Radwaste Building, Service Building, Control Building Annex and the stack on the Reactor Building roof.

RAI 03.07.02-14

QUESTION:

(Follow-up Question to RAI 03.07.02-2)

The applicant has provided an incomplete response in Appendix Section 3H.6.5.2, "Seismic System Analysis" as provided in enclosures to responses to RAI 03.07.01-11 & 13 as well as in the same section of the FSAR, Rev 3. More specifically, the applicant is requested to provide the following information in regards to "Seismic Analysis Methods."

1. The finite element model referenced in Figure 3H.6-40 (this figure is not yet available in the response to RAI 03.07.01-13).

2. Method used to model the backfill material in the SSI analysis.

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

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

5. The analysis method used to model concrete cracking.

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

RESPONSE:

1. Figure 3H.6-40 was provided as part of the Supplement 1 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/2009) and for convenience is reproduced below. For additional information and figures for the Soil-Structure Interaction (SSI) analysis model of the Ultimate Heat Sink (UHS) and Reactor Service Water (RSW) Pump House, please see the response to RAI 03.07.02-16 submitted concurrently with this response.

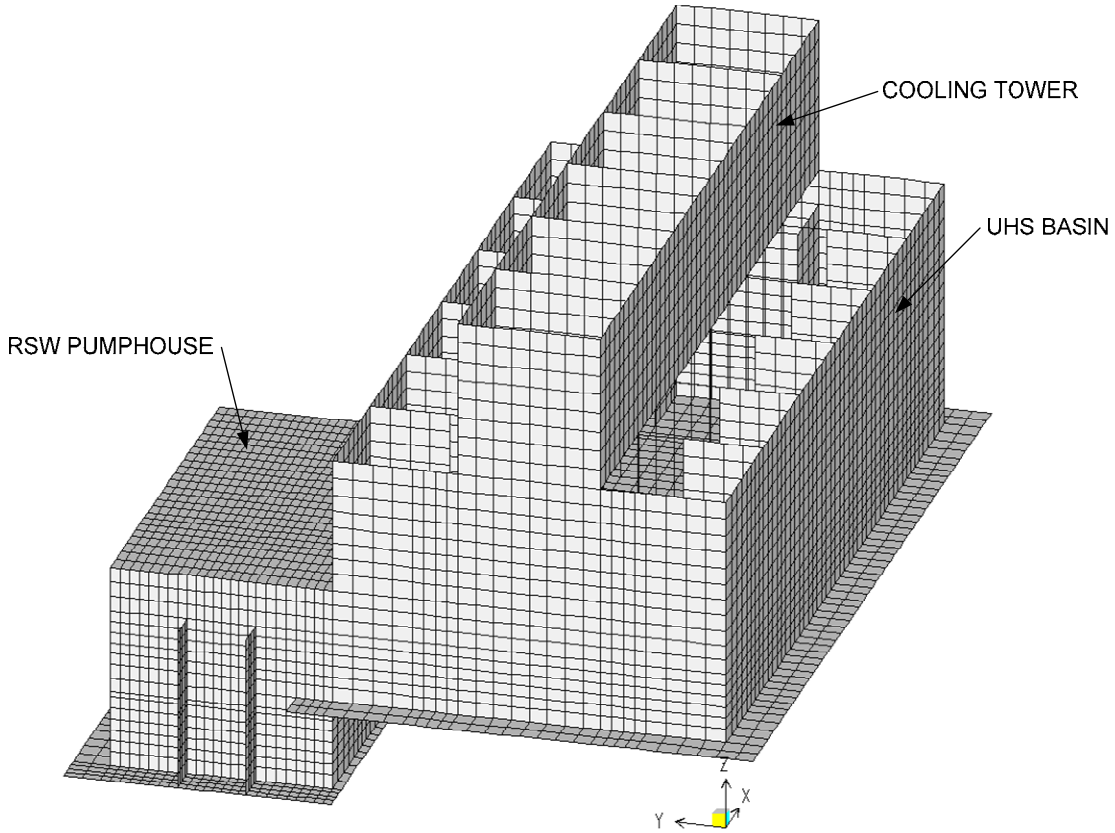


Figure 3H.6-40: SAP Finite Element Model for UHS and RSW Pump House Design

2. As stated in COLA Tier 2, Part2, Section 3H.6.5.2.4, in order to account for the backfill placed adjacent to the walls, an additional set of three SSI analyses (for best estimate, upper bound, and lower bound soil properties) was performed by modeling backfill as the soil horizon above the foundation level in the SASSI2000 model. The responses obtained from this set of SSI analyses and the analyses using in-situ soil as the horizon were enveloped. The strain compatible properties for the backfill material were calculated as explained in response to RAI 03.07.02-17, item 3 (see letter U7-C-STP-NRC-100035, dated 2/4/2010).

3. For soil below the ground water table, the compression wave velocities were calculated as described in the response to RAI 03.07.01-17, item 2 (see letter U7-C-STP-NRC-100035, dated 2/4/2010).

4. The analysis method used to obtain the seismic forces and moments for design evaluations is described in the response to RAI 03.07.02-15, item 11, submitted concurrently with this response.

5. As stated in response to RAI 03.07.02-4 (see letter U7-C-STP-NRC-090136, dated September 15, 2009) and the COLA mark-up submitted for Section 3H.6.5.2.3 with response to RAI 03.07.01-3 in the same letter, one SSI case analyzed addresses concrete cracking. For this case, the section modulus of the cracked concrete was based on 50% of the uncracked

section modulus. Results of this analysis were enveloped with the results of other SSI analyses for use in design.

6. As stated in response to RAI 03.07.02-5 (see letter U7-C-STP-NRC-090136, dated September 15, 2009) and the COLA mark-up submitted for Section 3H.6.5.2.4 with response to RAI 03.07.01-3 in the same letter, one SSI case analyzed addresses side-soil wall separation. For this case, the method recommended in Section 3.3.1.9 of ASCE 4-98 was used. Results of this analysis were enveloped with the results of other SSI analyses for use in design.

No COLA change is required for this response.

RAI 03.07.02-15

QUESTION:

(Follow-up Question to RAI 03.07.02-3)

The response to RAI 03.07.02-3 refers to the response to RAI 03.07.01-13. However, the responses to 03.07.01-13 are either incomplete or not available. Therefore, the applicant is requested to provide the missing information in Section 3H.6 of FSAR for this review to be completed. More specifically, the applicant is requested to provide the following:
UHS Basin and RSW Pump House:

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

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

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

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

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

6. A description for what and how many frequencies the model was analyzed in SASSI2000 and what frequency cutoff was used.

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

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

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

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

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

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

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

RSW Piping Tunnel:

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

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

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

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

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

6. Describe the SSI analysis from which the accelerations are obtained to establish the SSI forces for the analysis of the RSW piping tunnel (see the last bullet in Section 3H.6.6.2.2).

RESPONSE:

UHS Basin and RSW Pump House:

1. The fixed-base dominant frequencies and mass participation factors are provided in Table 3H.6-3 as part of Supplement 1 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/2009).

2. Seismic accelerations and displacements are provided in Table 3H.6-4 as part of Supplement 1 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/2009).

3. The seismic analysis of the UHS basin and enclosed cooling tower as well as RSW pump house for each unit was performed using a three-dimensional finite element model presented in Figures 3H.6-15 provided as part of Supplement 1 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/2009) and 3H.6-15a provided with response to RAI 03.07.02-16 being submitted concurrently. These figures are reproduced below for ready reference.

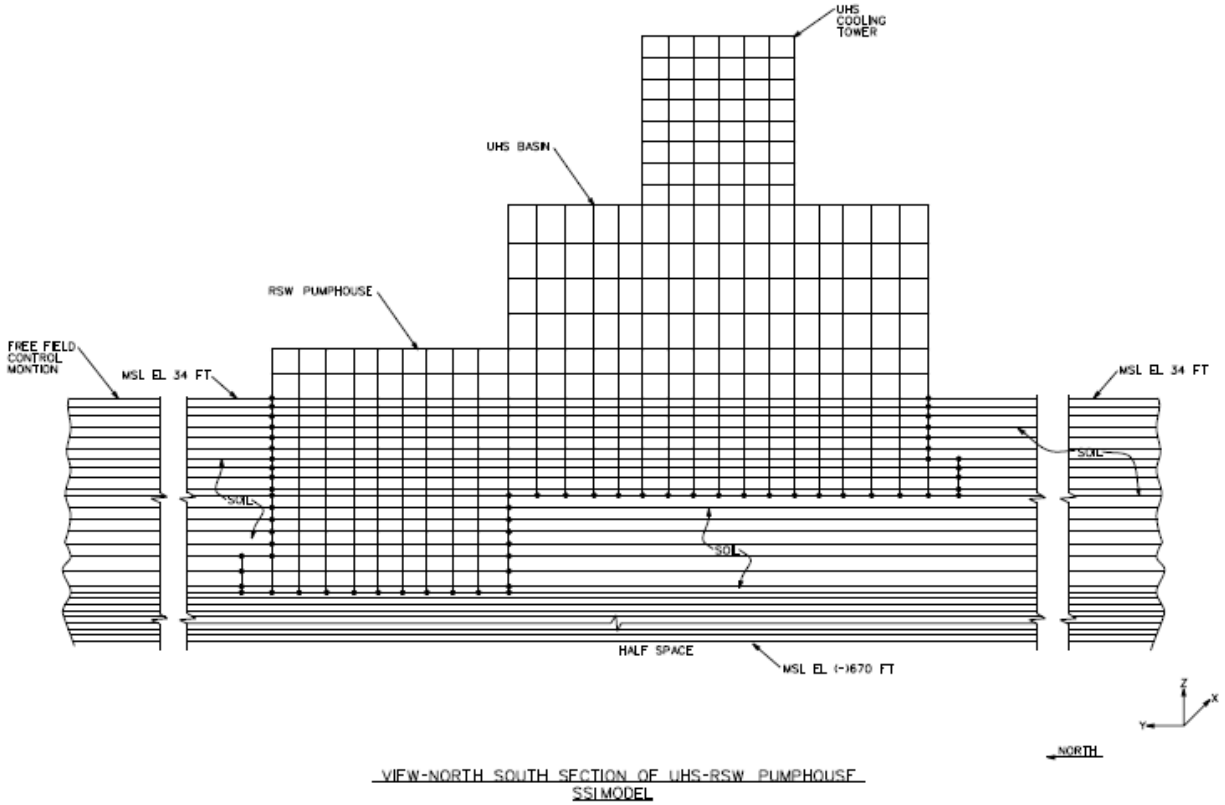


Figure 3H.6-15: SASSI2000 Model of UHS and RSW Pump House

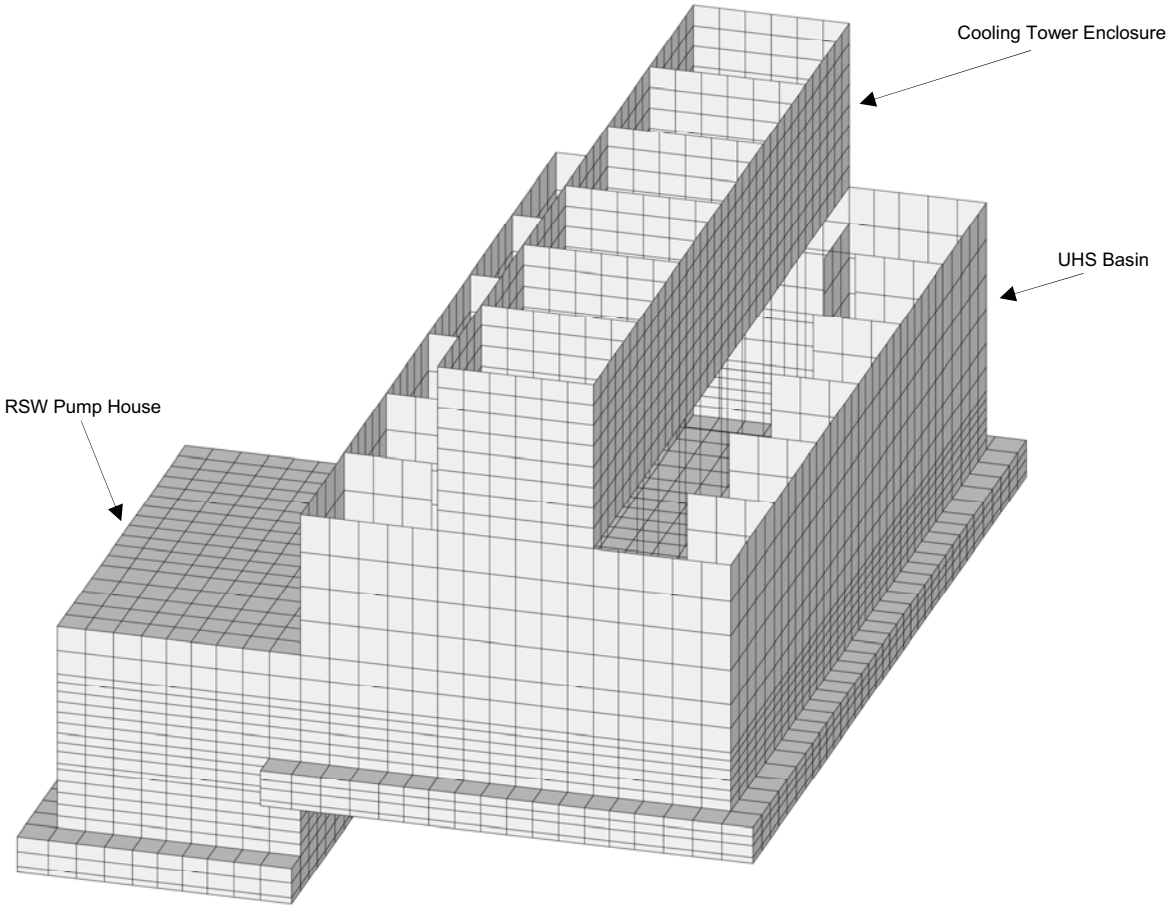


Figure 3H.6-15a: SSI Model (structure only)

The material properties for concrete elements of the model are presented in COLA Part 2, Tier 2, Section 3H.6.4.4.1. Uncracked concrete section was used for member stiffness. Another case with cracked concrete section properties was analyzed. The section modulus of the cracked concrete was based on 50% of the uncracked section modulus. For structural steel elements the Young's Modulus of 29 $x10^6$ psi and Poisson's ratio of 0.3 was used. The model consists primarily of plate elements that represent the reinforced concrete walls, buttresses, and foundation as well as the walls and slabs of the basin, cooling towers, and pump house. Beam elements were used to represent concrete columns and beams. Finally, solid elements were used to represent the basin and pump houses basemat. The analysis was performed in the frequency domain using SASSI2000 program. The input time histories were defined at a time step of 0.005 seconds. The same time step was used for generation of the in-structure response spectra.

The mass of the structures was represented primarily by the density of the plate, beam, and solid elements comprising the model. The dead load of the structures and major equipment (fans and pumps) was included along with a 50 psf load to account for the attached piping, grating, electrical cable trays and conduits, HVAC duct work etc. In addition, 25% of the floor live load was also included. The OBE damping values consistent with Regulatory Guide 1.61 were used. The impulsive water mass was calculated using the procedure described in Commentary Subsection C3.5.4 of ASCE 4-98, and was included in the model.

Solid elements representing the base slabs provide the proper interface with soil layers though the nodes do not have rotational degrees of freedom. Therefore columns not connected as shell elements are extended into the slab and beam elements are connected to each layer of solid elements. Additionally, walls represented as plate elements are connected to solid base slabs elements in order to model rotation and moment continuity properly.

The fixed-base analysis was completed using SAP2000. A modal analysis was conducted using the eigenvector option to compute the frequencies and mass participation factors.

4. Input motion is defined at grade in the free-field. All motions were applied to the model using SASSI2000. Vertically propagating plane shear waves are employed for the horizontal X and Y directions, while vertically propagating compression waves were used for the Z direction.

   In each analysis case, the response is calculated separately for each input direction. The combined response of all three directions is determined via square-root-sum-of-the-squares (SRSS). The final response is calculated by enveloping the responses in each direction for all SSI analysis cases.

5. As noted in item (4) above, input motions were defined at grade in the free-field.

6. Frequencies at which transfer functions are calculated are determined for various analysis cases including lower-bound, mean, upper-bound, fixed base, lower bound backfill, mean backfill, upper bound backfill, cracked, and the case with soil separation. The number of frequencies analyzed varies by case and are listed in the following tables. Cut-off frequencies range from 16 Hz to 29 Hz for various soil conditions and SSI analysis cases. The lowest cut-off frequency, 16 Hz, meets the ASCE 4-98 Section C3.3.3.4 recommended cut-off frequency limit of no less than 10-12 Hz, or twice the highest predominant frequency of coupled soil-structure system for the direction unconsidered.

Frequencies for SASSI Analysis for Lower-Bound, Mean, Upper-Bound, and Fixed Base Analysis Cases (Hz.)

Number	Lower Bound (LB)			Mean (BE)			Upper Bound (UB)			Fixed Base (FB)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
1	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
2	0.488	0.488	0.488	0.244	0.244	0.244	0.488	0.488	0.488	0.977	0.977	0.977
3	0.977	0.977	0.977	0.488	0.488	0.488	0.977	0.977	0.977	1.953	1.953	1.953
4	1.221	1.221	1.221	0.732	0.732	0.732	1.465	1.465	1.465	2.930	2.930	2.930
5	1.465	1.465	1.465	0.977	0.977	0.977	1.709	1.709	1.709	3.906	3.906	3.906
6	1.709	1.709	1.709	1.465	1.465	1.465	1.953	1.953	1.953	4.883	4.883	4.883
7	1.831	1.831	1.831	1.709	1.709	1.709	2.197	2.197	2.197	5.859	5.859	5.859
8	2.075	2.075	2.075	1.953	1.953	1.953	2.441	2.441	2.441	6.836	6.836	6.836
9	2.441	2.441	2.441	2.197	2.197	2.197	2.930	2.930	2.930	7.812	7.812	7.812
10	2.563	2.563	2.563	2.441	2.441	2.441	3.174	3.174	3.174	8.789	8.789	8.789
11	2.686	2.686	2.686	2.930	2.930	2.930	3.418	3.418	3.418	9.766	9.766	9.766
12	2.930	2.930	2.930	3.174	3.174	3.174	3.662	3.662	3.662	10.740	10.740	10.740
13	3.174	3.174	3.174	3.418	3.418	3.418	3.906	3.906	3.906	11.720	11.720	11.720
14	3.662	3.662	3.662	3.662	3.662	3.662	4.395	4.395	4.395	12.700	12.700	12.700
15	3.906	3.906	3.906	3.906	3.906	3.906	4.883	4.883	4.883	13.670	13.670	13.670
16	4.395	4.395	4.395	4.150	4.150	4.150	5.127	5.127	5.127	14.650	14.650	14.650
17	4.639	4.639	4.639	4.395	4.395	4.395	5.371	5.371	5.371	15.620	15.620	15.620
18	4.883	4.883	4.883	4.639	4.639	4.639	5.859	5.859	5.859	16.600	16.600	16.600
19	5.371	5.371	5.371	4.883	4.883	4.883	6.348	6.348	6.348	17.580	17.580	17.580
20	5.859	5.859	5.859	5.371	5.371	5.371	6.836	6.836	6.836	18.550	18.550	18.550
21	6.348	6.348	6.348	5.615	5.615	5.615	7.324	7.324	7.324	19.530	19.530	19.530
22	6.836	6.836	6.836	5.859	5.859	5.859	7.812	7.812	7.812	20.510	20.510	20.510
23	7.324	7.324	7.324	5.981	5.981	5.981	8.301	8.301	8.301	21.480	21.480	21.480
24	7.568	7.568	7.568	6.104	6.104	6.104	8.545	8.545	8.545	22.460	22.460	22.460
25	8.057	8.057	8.057	6.348	6.348	6.348	8.789	8.789	8.789	23.440	23.440	23.440

Frequencies for SASSI Analysis for Lower-Bound, Mean, Upper-Bound, and Fixed Base Analysis Cases (Hz.) (Cont'd)

Number	Lower Bound (LB)			Mean (BE)			Upper Bound (UB)			Fixed Base (FB)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
26	8.301	8.301	8.301	6.592	6.592	6.592	9.033	9.033	9.033	24.410	24.410	24.410
27	8.789	8.789	8.789	6.836	6.836	6.836	9.277	9.277	9.277	25.390	25.390	25.390
28	9.033	9.033	9.033	7.080	7.080	7.080	9.521	9.521	9.766	26.370	26.370	26.370
29	9.229	9.229	9.229	7.202	7.202	7.202	10.250	9.766	10.250	27.340	27.340	27.340
30	9.277	9.277	9.277	7.324	7.324	7.324	10.740	10.250	10.740	28.320	28.320	28.320
31	9.351	9.351	9.351	7.568	7.568	7.568	11.230	10.740	11.230	29.300	29.300	29.300
32	9.521	9.521	9.521	7.812	7.812	7.812	11.720	11.230	11.720	30.270	30.270	30.270
33	9.766	9.766	9.766	8.057	8.057	8.057	12.210	11.720	12.210	31.250	31.250	31.250
34	10.250	10.250	10.250	8.301	8.301	8.301	12.700	12.210	12.700	32.230	32.230	32.230
35	10.740	10.740	10.740	8.789	8.789	8.789	13.180	12.700	13.180	-	-	-
36	10.990	10.990	10.990	9.033	9.033	9.033	13.670	13.180	13.670	-	-	-
37	11.230	11.230	11.230	9.155	9.155	9.155	13.920	13.670	13.920	-	-	-
38	11.720	11.720	11.720	9.277	9.277	9.277	14.400	13.920	14.400	-	-	-
39	11.960	11.960	11.960	9.766	9.766	9.766	14.650	14.400	14.650	-	-	-
40	12.210	12.210	12.210	10.250	10.250	10.250	14.890	14.650	14.890	-	-	-
41	12.450	12.450	12.450	10.500	10.500	10.500	15.140	14.890	15.140	-	-	-
42	12.700	12.700	12.700	10.740	10.740	10.740	15.620	15.140	15.620	-	-	-
43	13.180	13.180	13.180	10.990	10.990	10.990	16.110	15.620	16.110	-	-	-
44	13.670	13.670	13.670	11.230	11.230	11.230	16.600	16.110	16.600	-	-	-
45	14.160	14.160	14.160	11.720	11.720	11.720	17.090	16.600	17.090	-	-	-
46	14.280	14.280	14.280	11.840	11.840	11.840	17.580	17.090	17.580	-	-	-
47	14.330	14.330	14.330	11.960	11.960	11.960	17.820	17.580	17.820	-	-	-
48	14.650	14.650	14.650	12.210	12.210	12.210	18.070	17.820	18.070	-	-	-
49	14.940	14.890	14.940	12.700	12.700	12.700	18.550	18.070	18.550	-	-	-
50	15.620	14.940	15.620	13.180	13.180	13.180	18.800	18.550	18.800	-	-	-
51	16.110	15.620	16.110	13.670	13.670	13.670	19.040	18.800	19.040	-	-	-
52	16.600	16.110	16.600	13.790	13.790	13.790	19.290	19.040	19.290	-	-	-
53	-	16.600	-	13.920	13.920	13.920	19.530	19.290	19.530	-	-	-
54	-	-	-	14.160	14.160	14.400	20.020	19.530	20.020	-	-	-

Frequencies for SASSI Analysis for Lower-Bound, Mean, Upper-Bound, and Fixed Base Analysis Cases (Hz.) (Cont'd)

Number	Lower Bound (LB)			Mean (BE)			Upper Bound (UB)			Fixed Base (FB)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
55	-	-	-	14.400	14.400	14.650	20.260	20.020	20.260	-	-	-
56	-	-	-	14.650	14.650	14.890	20.530	20.260	20.530	-	-	-
57	-	-	-	14.890	14.890	15.140	21.000	20.530	21.000	-	-	-
58	-	-	-	15.140	15.140	15.620	21.480	21.000	21.480	-	-	-
59	-	-	-	15.620	15.620	16.110	21.970	21.480	21.970	-	-	-
60	-	-	-	16.110	16.110	16.600	22.460	21.970	22.460	-	-	-
61	-	-	-	16.600	16.600	17.090	22.950	22.460	22.950	-	-	-
62	-	-	-	17.090	17.090	17.580	23.440	22.950	23.440	-	-	-
63	-	-	-	17.580	17.580	18.070	23.930	23.440	23.930	-	-	-
64	-	-	-	18.070	18.070	18.550	24.410	23.930	24.410	-	-	-
65	-	-	-	18.550	18.550	19.040	24.900	24.410	25.390	-	-	-
66	-	-	-	19.040	19.040	19.530	25.390	25.390	-	-	-	-
67	-	-	-	19.530	19.530	20.260	-	-	-	-	-	-
68	-	-	-	20.260	20.260	-	-	-	-	-	-	-

Frequencies for SASSI Analysis Lower Bound Backfill, Mean Backfill, Upper Bound Backfill, Cracked and Separated Analysis Cases (Hz.)

Number	Lower Bound Backfill (LBBF)			Mean Backfill (BEBF)			Upper Bound Backfill (UBBF)			Cracked (CR)			Separated (SEP)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
1	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
2	0.244	0.244	0.244	0.488	0.488	0.488	0.488	0.488	0.488	0.244	0.244	0.244	0.488	0.488	0.488
3	0.732	0.732	0.732	0.977	0.977	0.977	0.977	0.977	0.977	0.488	0.488	0.488	0.977	0.977	0.977
4	0.977	0.977	0.977	1.465	1.465	1.465	1.465	1.465	1.465	0.732	0.732	0.732	1.221	1.221	1.221
5	1.221	1.221	1.221	1.709	1.709	1.709	1.709	1.709	1.709	0.977	0.977	0.977	1.465	1.465	1.465
6	1.465	1.465	1.465	1.953	1.953	1.953	1.953	1.953	1.953	1.221	1.221	1.221	1.709	1.709	1.709
7	1.709	1.709	1.709	2.197	2.197	2.197	2.197	2.197	2.197	1.465	1.465	1.465	1.953	1.953	1.953
8	1.953	1.953	1.953	2.441	2.441	2.441	2.441	2.441	2.441	1.709	1.709	1.709	2.197	2.197	2.197
9	2.197	2.197	2.197	2.686	2.686	2.686	2.930	2.930	2.930	1.953	1.953	1.953	2.441	2.441	2.441
10	2.441	2.441	2.441	2.930	2.930	2.930	3.174	3.174	3.174	2.197	2.197	2.197	2.686	2.686	2.686
11	2.686	2.686	2.686	3.174	3.174	3.174	3.418	3.418	3.418	2.441	2.441	2.441	2.930	2.930	2.930
12	2.930	2.930	2.930	3.418	3.418	3.418	3.662	3.662	3.662	2.686	2.686	2.686	3.174	3.174	3.174
13	3.174	3.174	3.174	3.662	3.662	3.662	3.906	3.906	3.906	2.930	3.174	2.930	3.418	3.418	3.418
14	3.418	3.418	3.418	3.906	3.906	3.906	4.150	4.150	4.150	3.174	3.418	3.174	3.662	3.662	3.662
15	3.662	3.662	3.662	4.395	4.395	4.395	4.395	4.395	4.395	3.418	3.662	3.418	3.931	3.931	3.931
16	4.102	4.102	4.102	4.883	4.883	4.883	4.883	4.883	4.883	3.662	3.906	3.662	4.150	4.150	4.150
17	4.150	4.150	4.150	5.371	5.371	5.371	5.127	5.127	5.127	3.906	4.150	3.906	4.395	4.395	4.395
18	4.395	4.395	4.395	5.859	5.859	5.859	5.371	5.371	5.371	4.150	4.395	4.150	4.883	4.883	4.883
19	4.639	4.639	4.639	6.348	6.348	6.348	5.859	5.859	5.859	4.395	4.639	4.395	5.371	5.371	5.371
20	4.883	4.883	4.883	6.592	6.592	6.592	6.348	6.348	6.348	4.639	4.883	4.639	5.859	5.859	5.859
21	5.371	5.371	5.371	6.836	6.836	6.836	6.836	6.836	6.836	4.883	5.127	4.883	6.104	6.104	6.104
22	5.859	5.859	5.859	7.324	7.324	7.324	7.324	7.324	7.324	5.127	5.371	5.127	6.348	6.348	6.348
23	6.104	6.104	6.104	7.812	7.812	7.812	7.812	7.812	7.812	5.371	5.615	5.371	6.836	6.836	6.836
24	6.348	6.348	6.348	8.057	8.057	8.057	8.301	8.301	8.301	5.615	5.859	5.615	7.324	7.324	7.324
25	6.836	6.836	6.836	8.301	8.301	8.301	8.789	8.789	8.789	6.104	6.104	5.859	7.812	7.812	7.812

Frequencies for SASSI Analysis Lower Bound Backfill, Mean Backfill, Upper Bound Backfill, Cracked and Separated Analysis Cases (Hz.) (Cont'd)

Number	Lower Bound Backfill (LBBF)			Mean Backfill (BEBF)			Upper Bound Backfill (UBBF)			Cracked (CR)			Separated (SEP)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
26	7.324	7.324	7.324	8.789	8.789	8.789	9.277	9.277	9.277	6.348	6.348	6.104	8.301	8.301	8.301
27	7.812	7.812	7.812	9.277	9.277	9.277	9.521	9.521	9.521	6.592	6.592	6.348	8.789	8.789	8.789
28	8.301	8.301	8.301	9.766	9.766	9.766	9.766	10.010	9.766	6.836	6.836	6.592	9.277	9.277	9.277
29	8.789	8.789	8.789	10.250	10.250	10.250	10.010	10.250	10.010	7.080	7.080	6.836	9.766	9.766	9.766
30	9.277	9.277	9.277	10.740	10.740	10.740	10.250	10.740	10.250	7.324	7.324	7.080	10.250	10.250	10.250
31	9.766	9.766	9.766	11.230	11.230	11.230	10.740	11.230	10.740	7.568	7.568	7.324	10.740	10.740	10.740
32	10.250	10.250	10.250	11.470	11.470	11.470	11.230	11.720	11.230	7.812	7.812	7.568	10.990	10.990	10.990
33	10.740	10.740	10.740	12.210	12.210	12.210	11.720	12.210	11.720	8.057	8.057	7.812	11.230	11.230	11.230
34	10.990	10.990	10.990	12.720	12.720	12.720	12.210	12.700	12.210	8.301	8.301	8.057	11.470	11.470	11.470
35	11.230	11.230	11.230	13.180	13.180	13.180	12.700	13.180	12.700	8.545	8.545	8.301	11.720	11.720	11.720
36	11.720	11.720	11.720	13.670	13.670	13.670	13.180	13.670	13.180	8.789	8.789	8.545	12.210	12.210	12.210
37	11.960	11.960	11.960	14.160	14.160	14.160	13.670	14.160	13.670	9.033	9.033	8.789	12.700	12.700	12.700
38	12.210	12.210	12.210	14.400	14.400	14.400	14.160	14.650	14.160	9.277	9.277	9.033	13.180	13.180	13.180
39	12.450	12.450	12.450	14.650	14.650	14.650	14.650	15.110	14.650	9.521	9.521	9.277	13.430	13.430	13.430
40	12.700	12.700	12.700	14.890	14.890	14.890	15.110	15.620	15.110	9.766	9.766	9.521	13.670	13.670	13.670
41	12.740	12.740	12.740	15.140	15.140	15.140	15.620	15.870	15.620	10.250	10.250	9.766	13.920	13.920	13.920
42	12.890	12.890	12.890	15.380	15.380	15.380	15.870	16.360	15.870	10.740	10.740	10.250	14.160	14.160	14.160
43	12.940	12.940	12.940	15.620	15.620	15.870	16.360	16.600	16.360	11.230	11.230	10.740	14.400	14.400	14.400
44	13.180	13.180	13.180	15.870	15.870	16.110	17.090	17.090	17.090	11.720	11.720	11.230	14.650	14.650	14.650
45	13.310	13.310	13.310	16.110	16.110	16.600	17.580	17.580	17.580	12.210	12.210	11.720	14.890	14.890	14.890
46	13.430	13.430	13.430	16.600	16.600	16.850	18.070	18.070	18.070	12.700	12.700	12.210	15.380	15.380	15.140
47	13.670	13.670	13.670	16.850	16.850	17.090	18.550	18.550	18.550	13.180	13.180	12.700	15.620	15.620	15.380
48	13.920	13.920	13.920	17.090	17.090	17.330	18.800	18.800	18.800	13.670	13.670	13.180	15.870	15.870	15.620
49	14.160	14.160	14.160	17.330	17.330	17.580	19.040	19.040	19.040	14.160	14.160	13.670	16.110	16.110	15.870
50	14.210	14.210	14.210	17.580	17.580	18.070	19.530	19.530	19.530	14.400	14.400	14.160	16.600	16.600	16.110
51	14.280	14.280	14.280	18.070	18.070	18.550	20.020	20.020	20.020	14.890	14.650	14.400	17.090	17.090	16.600
52	14.430	14.430	14.430	18.550	18.550	19.040	20.510	20.510	20.510	15.620	14.890	14.650	17.580	17.580	17.090
53	14.530	14.530	14.530	19.040	19.040	19.560	20.750	20.750	20.750	15.870	15.140	14.890	18.070	18.070	17.580

Frequencies for SASSI Analysis Lower Bound Backfill, Mean Backfill, Upper Bound Backfill, Cracked and Separated Analysis Cases (Hz.) (Cont'd)

Number	Lower Bound Backfill (LBBF)			Mean Backfill (BEBF)			Upper Bound Backfill (UBBF)			Cracked (CR)			Separated (SEP)		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
54	14.650	14.650	14.650	19.560	19.560	20.020	21.000	21.000	21.000	16.360	15.380	15.380	18.550	18.550	18.070
55	14.770	14.770	14.770	20.020	20.020	20.510	21.480	21.480	21.480	16.600	15.620	15.620	19.040	19.040	18.550
56	14.890	14.890	14.890	20.510	20.510	21.000	21.970	21.970	21.970	17.090	15.870	15.870	19.530	19.530	19.040
57	15.140	15.140	15.140	21.000	21.000	21.480	22.460	22.460	22.460	17.330	16.110	16.110	-	-	19.530
58	15.620	15.620	15.620	21.480	21.480	21.970	22.710	22.710	22.710	17.580	16.360	16.360	-	-	-
59	16.110	16.110	16.110	21.970	21.970	22.460	22.950	22.950	23.190	18.070	16.600	16.600	-	-	-
60	16.360	16.360	16.360	22.460	22.460	22.950	23.190	23.190	23.440	18.550	17.090	17.090	-	-	-
61	16.600	16.600	16.600	22.950	22.950	23.440	23.440	23.440	23.930	19.040	17.330	17.330	-	-	-
62	16.700	16.700	16.700	23.440	23.440	-	23.930	23.930	24.410	19.530	17.580	17.580	-	-	-
63	16.750	16.750	16.750	-	-	-	24.410	24.410	24.900	-	18.070	18.550	-	-	-
64	16.800	16.800	16.800	-	-	-	24.900	24.900	25.390	-	18.550	19.040	-	-	-
65	16.850	17.090	16.850	-	-	-	25.390	25.390	25.880	-	19.040	19.530	-	-	-
66	17.090	17.580	17.090	-	-	-	25.880	25.880	26.370	-	19.530	-	-	-	-
67	17.580	18.070	17.580	-	-	-	26.370	26.370	27.340	-	-	-	-	-	-
68	18.070	18.550	18.070	-	-	-	27.340	27.340	28.320	-	-	-	-	-	-
69	18.550	-	18.550	-	-	-	28.320	28.320	-	-	-	-	-	-	-

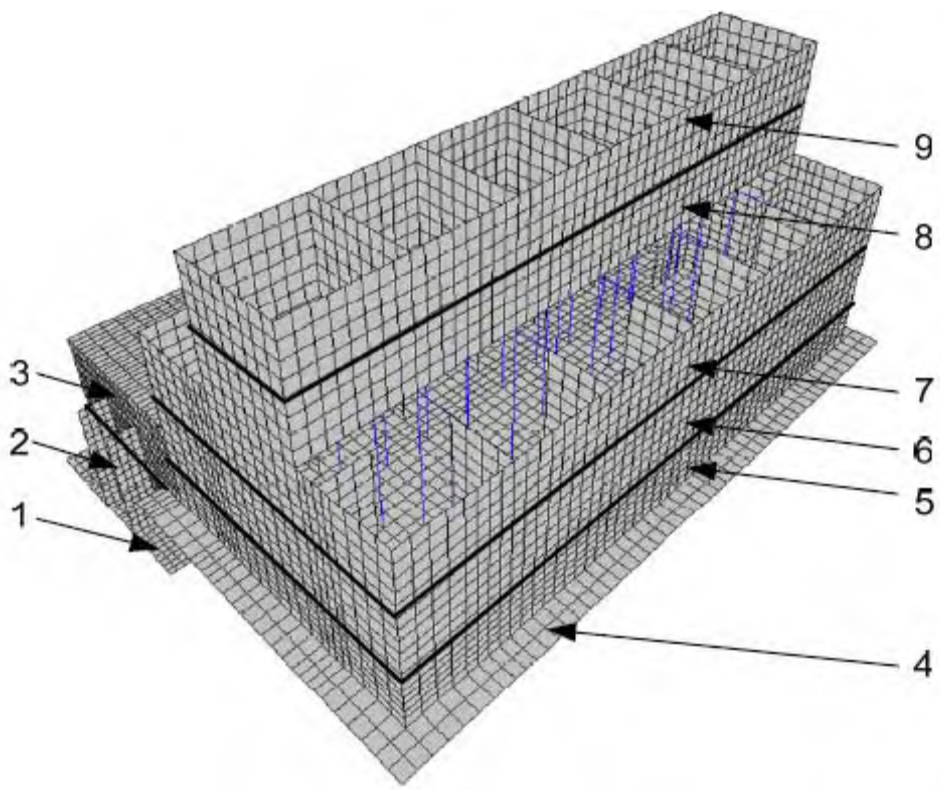
7. Figure 3H.6-15 showing the north-south cross-section of the three-dimensional finite element model of both structure and soil layers was provided as part of the Supplement 1 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208, dated 11/19/2009) and for convenience was reproduced in part 3 above. For additional information and figures, please see the response to RAI 03.07.02-16 submitted concurrently with this response.

8. The ground water effects were treated as described in the response to RAI 03.07.01-17 (see letter U7-C-STP-NRC-100035, dated 2/3/2010).

9. The descriptions of the time step, number of acceleration points, and duration of motion including duration of quiet zone used in the input motion for the SASSI analysis, are provided in the response to RAI 03.07.01-15 (see letter U7-C-STP-NRC-100035, dated 2/3/2010).

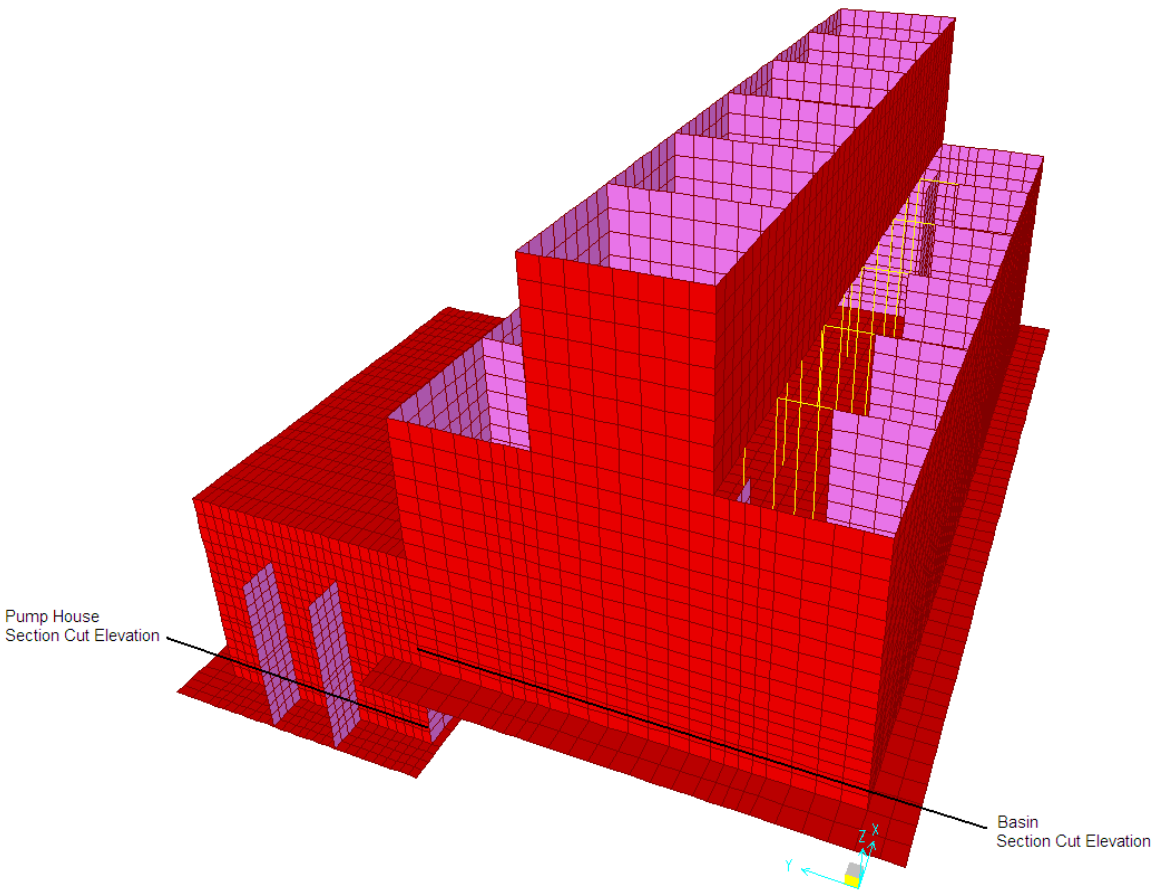
10. A separate equivalent static analysis was performed to obtain seismic forces and moments. Please see item 11 below for more information.

11. An equivalent static analysis using SAP2000 FEM was used to obtain seismic forces and moments. In order to determine seismic loads used in load combinations nodal accelerations in the global X, Y, and Z directions are averaged by group. Groups are shown in the figure below:

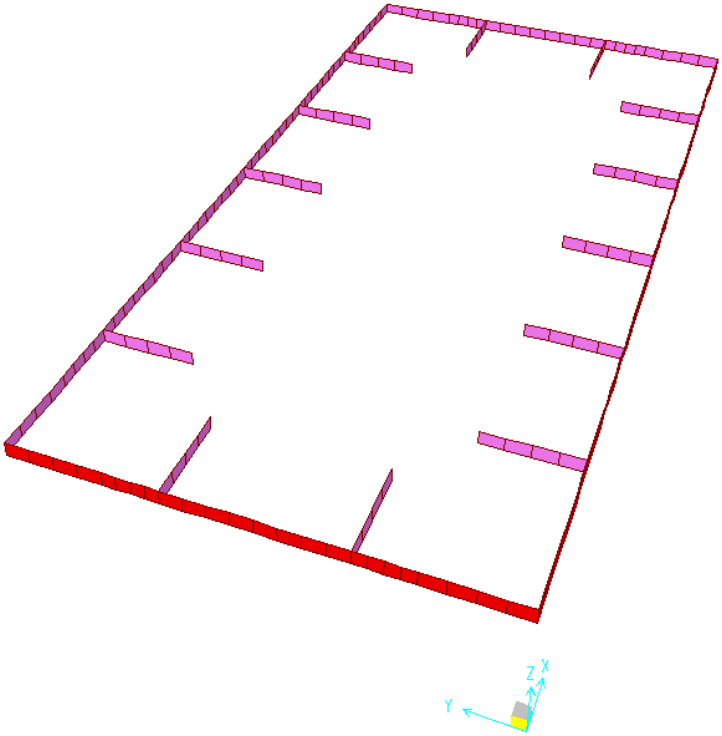


The mass of the structure, equipment weights, seismic live loads, and hydrodynamic forces are normalized to 1g in the model. Depending on their location in the structure, these loads are factored by the group acceleration and the seismic loads from the three orthogonal directions are combined by square-root-sum-of-the-squares (SRSS) method. Please see the following figures and tables demonstrating the conservatism of the equivalent static method used.

SAP2000 Seismic Design Model Section Cut Elevations



UHS Basin Section Cut View

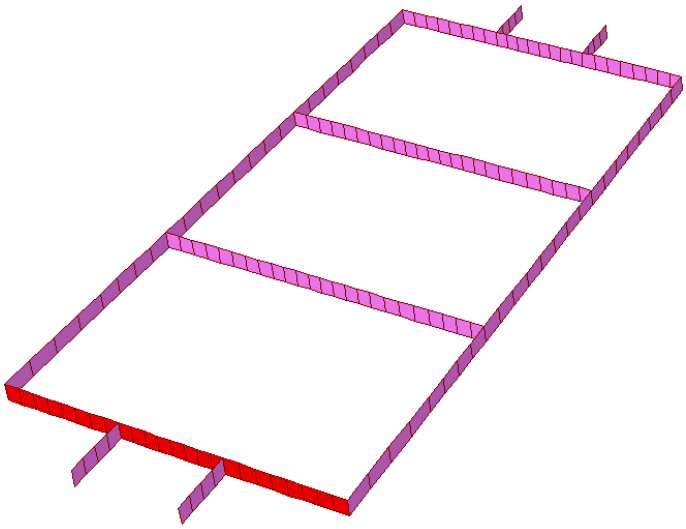


SAP2000 vs SSI Model UHS Basin Section Cut Seismic Force and Moment Comparison Table (Units: Kips, Kip-ft):

	FX (E-W)	FY (N-S)	FZ (Vertical)	MX	MY	MZ
SAP2000 Basin section cut forces due to X-dir (E-W) seismic load:	38744	1135	1117	72015	2580102	9719
SAP2000 Basin section cut forces due to Y-dir (N-S) seismic load:	562	37684	10373	1588284	525623	420299
SAP2000 Basin section cut forces due to Z-dir (Vertical) seismic load:	50	911	14627	57754	39076	109979
SAP2000 Basin section cut SRSS seismic design forces:	38748	37712	17967	1590964	2633388	434559
Enveloped SSI peak Basin section cut forces:	24605	25224	12984	953117	1445198	not reported
Ratio SAP2000/SSI Basin section cut forces	1.57	1.50	1.38	1.67	1.82	n/a

Pump House Section Cut

For soil modeling purposes, the SSI model used frame elements to represent the pump house buttresses. The SAP2000 design model used shell elements to represent the pump house buttresses.



SAP2000 vs SSI Model Pump House Section Cut Seismic Force and Moment Comparison Table (Units: Kips, Kip-ft):

	FX (E-W)	FY (N-S)	FZ (Vertical)	MX	MY	MZ
SAP2000 PH section cut forces due to X-dir (E-W) seismic load:	18372	3379	4162	47419	464946	263448
SAP2000 PH section cut forces due to Y-dir (N-S) seismic load:	3076	24072	20805	357288	143893	172844
SAP2000 PH section cut forces due to Z-dir (Vertical) seismic load:	270	1385	10546	86951	35671	770
SAP2000 PH section cut SRSS design forces:	18630	24348	23693	370762	488009	315088
SAP2000 PH section cut forces due Equivalent Static Seismic Soil on PH	333	2929	994	5960	4228	24069
SAP2000 PH section cut seismic design forces:	18963	27277	24688	376722	492237	339157
Enveloped SSI peak PH section cut forces:	17922	22490	13291	253012	309953	not reported
Ratio SAP2000/SSI PH section cut forces	1.06	1.21	1.86	1.49	1.59	n/a

All elements in the UHS/RSW pump house model are shell or frame elements. These element types are assigned to structural members based on the behavior of that structural member. Different element types (shell and frame) are directly connected to each other in SAP2000, and the mass and stiffness of each element is determined in the analysis program.

The UHS basin and pump house foundation mats are modeled as 10 ft thick shell elements. In order to locate the nodes at the mid-depth of the foundation mats, five foot deep zero-mass shell elements are used to connect the nodes at the bottom of the walls and buttresses of the basin and pump house to the foundation mats. Also, zero-mass frame elements are used to connect the nodes at the bottom of the columns to the basin foundation mat. In order to capture the stiffness of the mat, the modulus of elasticity for both the zero-mass shell element and the zero-mass frame elements is 100 times that of the reinforced concrete.

All basin walls, basin buttresses, pump house walls, and pump house buttresses are modeled as six foot thick shell elements. The pump house interior walls are modeled as four foot thick shell elements. The pump house operating floor and roof are modeled using 19.75 in thick shell elements, which is the total thickness of the slab, including the decking, minus half the decking thickness.

The pump house operating floor and roof beams are modeled as frame elements. The beams have releases such that they only increase the stiffness in the vertical direction. These beams are included only to capture their weight and to represent their effect on the vertical stiffness of the slabs. UHS columns supporting the cooling towers are modeled as 5'x5' and 5'x12' frame elements. Beams in the north-south direction in the UHS basin are modeled as 2'x4.5' frame elements. Exterior north and south cooling tower enclosure walls are modeled as two foot thick shell elements. The exterior east and west cooling tower enclosure walls are modeled as six foot thick shell elements. The interior cooling tower enclosure walls are modeled as two foot thick shell elements.

Soil springs for both the UHS basin and RSW Pump House foundation mats in the seismic model are supported with area springs with uniform spring constants as follows:

- UHS basin foundation
  - Vertical direction (global Z) = 80 kips/ft/ft^2
  - East-West direction (global X) = 30 kips/ft/ft^2
  - North-South direction (global Y) = 33 kips/ft/ft^2

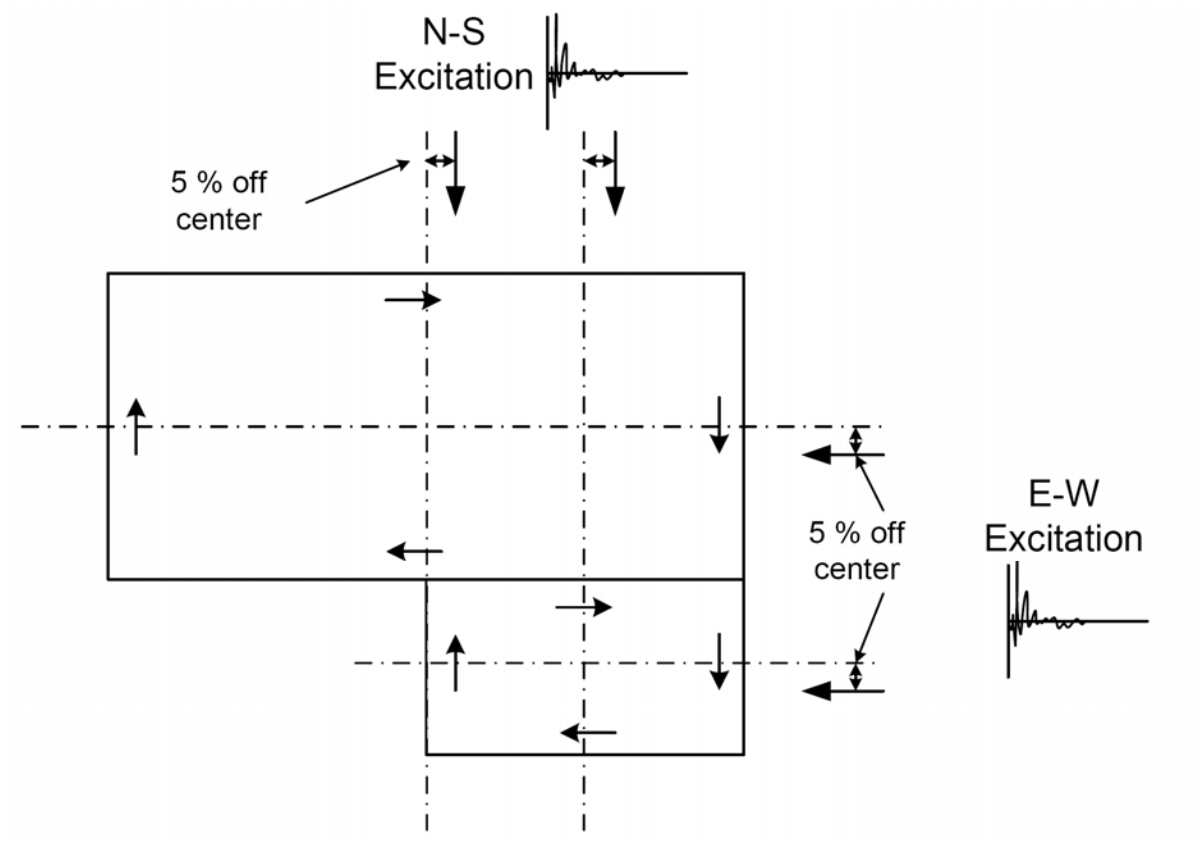
- RSW Pump House foundation
  - Vertical direction (global Z) = 170 tons/ft/ft^2
  - East-West direction (global X) = 104 kips/ft/ft^2
  - North-South direction (global Y) = 112 kips/ft/ft^2

For detailed information regarding the methodology used for determination of soil spring constants under static and dynamic loading, please see the response to RAI 03.08.04-23 submitted concurrently with this response.

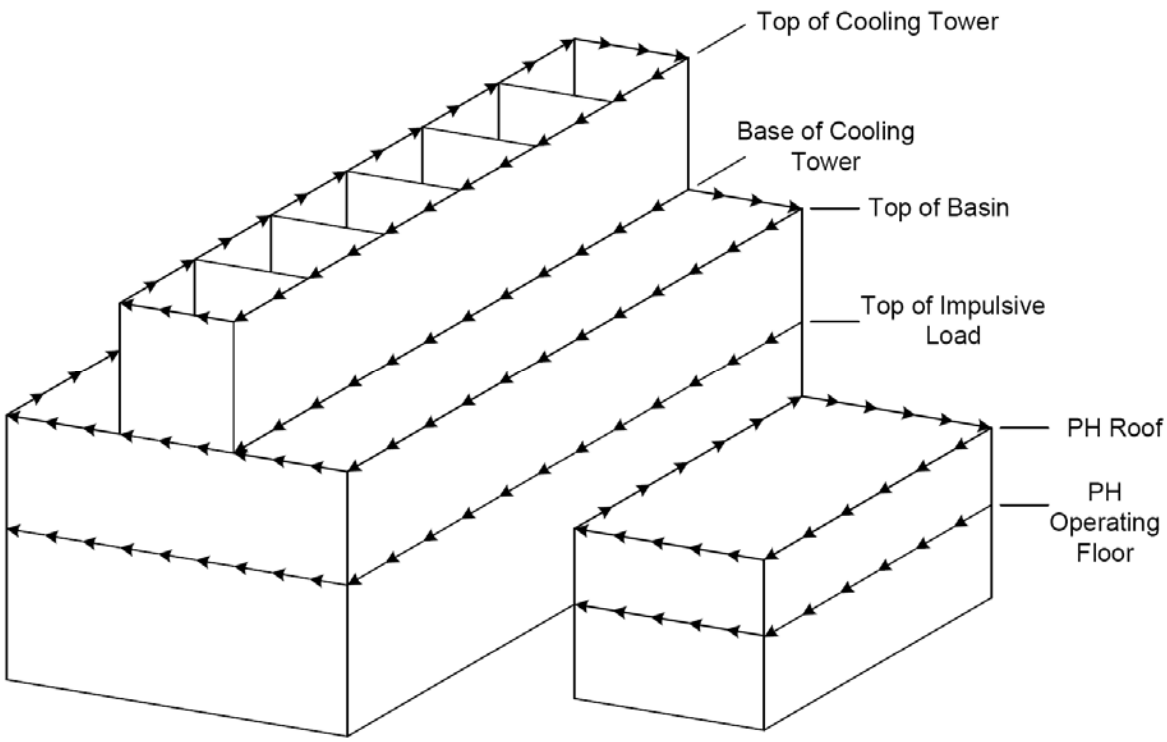
Hydrodynamic loads representing impulsive and convective (sloshing) effects of the water within the UHS basin are included. Additionally, a vertical hydrodynamic pressure resulting from vertical excitation is included.

Dynamic soil loads are calculated in accordance with section 3.5.3.2.2 of ASCE 4-98 and compared to the envelope soil pressures from the Soil-Structure Interaction analysis, and the maximum pressures are used in the SAP2000 model.

Accidental eccentricity load used in the analysis accounts for a torsional moment resulting from a 5% eccentricity in plan dimension between the centers of mass and rigidity. Masses are determined based on structure, equipment and live loads. These masses are used in conjunction with the seismic accelerations in both the global X and Y directions to obtain an equivalent static torsional moment.



The figure below shows how the clockwise torsional moments due to both X and Y excitations are applied at the operating floor and roof slabs of the RSW Pump House, at the top of the impulsive load on the basin walls, at the top of the basin walls, and at the base and top of the cooling tower walls. The counterclockwise torsional moments are applied in the same manner in the opposite direction. This method conservatively assumes that ground excitations in both directions occur simultaneously. The torsional moments are applied in the SAP model as nodal forces to capture both the in-plane shear and moment on the walls. In all seismic load combinations, the resulting demand from accidental eccentricity is additive to the demand for other loads.



12. Numerous walls in the UHS and RSW Pump House are soil-retaining walls including, the UHS Basin east, south and west walls, the eastern portion of the UHS Basin north wall, as well as the west, north, east, and lower portion of the south RSW Pump House Walls. Maximum displacements are shown for a variety of nodes at numerous elevations throughout the UHS Basin and RSW Pump House.

Summary of Enveloping Maximum Displacement Relative to Input Motion at Free-Field Grade Level

Location	SAP Node No.	SASSI Node No.	Displacement Relative to Input Motion (in.): Envelope of all Analysis Cases		
			East-West (X)	North-South (Y)	Vertical (Z)
Bottom of PH walls					
	663	1163	0.18	0.20	0.08
	843	1527	0.21	0.20	0.10
	860	1561	0.22	0.19	0.11
	680	1197	0.18	0.19	0.07
Mid-level of PH walls					
	11920	14995	0.16	0.14	0.07
	11863	15101	0.17	0.11	0.09
	11823	15015	0.16	0.11	0.08
	11766	14851	0.16	0.11	0.05
PH roof					
	5511	16429	0.16	0.13	0.08
	5690	16608	0.16	0.12	0.10
	5707	16625	0.17	0.11	0.11
	5528	16446	0.16	0.13	0.06
	5626	16544	0.16	0.09	0.06
	5621	16539	0.16	0.11	0.06
	5632	16550	0.16	0.10	0.07
Bottom of UHS basin walls					
	3397	8546	0.15	0.14	0.11
	3989	9753	0.16	0.15	0.08
	4023	9821	0.16	0.12	0.13
	3431	8614	0.15	0.12	0.12
Mid-level of UHS basin walls					
	5778	16815	0.63	0.14	0.10
	5832	16869	0.16	0.20	0.07
	5779	16816	0.65	0.15	0.10
	5728	16765	0.16	0.34	0.11
Top of UHS basin walls					
	6180	17263	0.16	0.13	0.12
	6410	17493	0.15	0.13	0.08
	6444	17527	0.17	0.19	0.13
	6214	17297	0.16	0.19	0.13

Summary of Enveloping Maximum Displacement Relative to Input Motion at Free-Field Grade Level (Cont'd)

Location	SAP Node No.	SASSI Node No.	Displacement Relative to Input Motion (in.): Envelope of all Analysis Cases		
			East-West (X)	North-South (Y)	Vertical (Z)
Bottom of cooling tower walls					
cell 1	6258	17341	1.31	0.13	0.12
	6330	17413	1.29	0.13	0.12
	6336	17419	1.58	0.34	0.09
	6264	17347	1.59	0.38	0.09
cell 3	6270	17353	1.67	0.51	0.12
	6342	17425	1.65	0.45	0.10
	6348	17431	1.66	0.60	0.11
	6276	17359	1.68	0.68	0.12
cell 6	6288	17371	1.60	0.78	0.10
	6360	17443	1.58	0.78	0.09
	6366	17449	1.29	0.19	0.13
	6294	17377	1.31	0.19	0.13
Mid-level of cooling tower walls					
cell 1	6823	17956	1.72	0.13	0.11
	6847	17980	1.62	0.32	0.07
	6824	17957	2.04	0.44	0.08
	6775	17908	1.64	0.35	0.09
cell 3	6825	17958	2.20	0.57	0.09
	6859	17992	1.65	0.76	0.11
	6826	17959	2.05	0.71	0.09
	6787	17920	1.67	0.76	0.13
cell 6	6828	17961	2.04	0.91	0.08
	6877	18010	1.62	0.67	0.08
	6829	17962	1.72	0.24	0.11
	6805	17938	1.64	0.66	0.09

Summary of Enveloping Maximum Displacement Relative to Input Motion at Free-Field Grade Level (Cont'd)

Location	SAP Node No.	SASSI Node No.	Displacement Relative to Input Motion (in.): Envelope of all Analysis Cases		
			East-West (X)	North-South (Y)	Vertical (Z)
Top of cooling tower walls					
cell 1	7208	18341	1.75	0.14	0.12
	7280	18413	1.73	0.14	0.11
	7286	18419	1.67	0.50	0.09
	7214	18347	1.69	0.49	0.10
cell 3	7220	18353	1.66	0.63	0.13
	7292	18425	1.64	0.64	0.11
	7298	18431	1.64	0.79	0.12
	7226	18359	1.66	0.79	0.13
cell 6	7238	18371	1.69	0.99	0.11
	7310	18443	1.67	0.99	0.10
	7316	18449	1.73	0.30	0.12
	7244	18377	1.75	0.30	0.12
PH operating floor					
	3989	9753	0.16	0.15	0.08
	4188	10155	0.18	0.16	0.10
	4205	10189	0.18	0.14	0.11
	4006	9787	0.16	0.13	0.07
	4119	10015	0.17	0.15	0.08
	4124	10025	0.17	0.14	0.09
	4130	10037	0.17	0.13	0.09

13. Under section 3.1.6.3 "Building Model Hydrodynamic Mass Effects" of ASCE 4-98 it is indicated that fluids contained in basins within a structure shall be modeled to represent both impulsive and convective (sloshing) effects. The impulsive mass may be uniformly distributed over a height equal to twice the distance from the bottom of the basin to the center of mass as determined for the simplified case of a single impulsive mass. Chapter 6 of Nuclear Reactors and Earthquakes TID-7024 outlines a methodology for creating a dynamic model of fluid motion in a tank which includes an impulsive mass acting at the centroid of the impulsive pressure prism.

    According to Section 3.1.6.3 (d) of ASCE 4-98, for water depths less than 50 ft, the entire water mass may be lumped at the foundation mat of the basin. For water depths greater than 50 ft, the effects due to the compressibility of water shall be determined on the basis of engineering mechanics principles. The water depth in the UHS basin is 71 ft. Based on a compression wave velocity (v_c) of water equal to 4800 ft/sec, the vertical frequency (f_v) of 71 ft of water is given by:

    $$f_v = v_c/4\text{h} = 17 \text{ Hz}$$

    Since the predominant SSI frequencies are below 17 Hz the water mass is lumped at the foundation mat of the basin. When the fluid is accelerated in the horizontal direction, a certain portion of the fluid acts as if it were a solid mass in rigid contact with the walls. This portion is defined in TID-7024 as the impulsive mass. The horizontal acceleration also causes the fluid to oscillate. This is the oscillating or convective mass. Using the UHS response spectra, the acceleration values are determined for the convective masses in the global X and Y directions. Shell elements of the SAP2000 models are modeled at the wall/slab/buttress centerlines, thus pressures are adjusted based on the geometry of the area where they are applied.

    Since the buttresses baffle the impulsive water pressure, a proportionate impulsive pressure is applied to the basin walls in each of the global directions, while the remaining impulsive pressure is evenly distributed among the buttresses and the shielded portion of the basin walls. Since the impulsive force of the water is acting on both sides of the buttress, the buttresses are assigned twice the pressure as the portion of the basin walls shielded by the buttresses. This is completed for acceleration in both north/south and east/west directions.

    According to section 3.1.6.3 (c) of ASCE 4-98, the convective pressure shall be distributed over a height from the top of the water surface to the center of the equivalent oscillating mass. In order to conservatively model the resulting bending stresses on the wall, the convective pressure is assigned to the shell elements beginning at 42.1 ft above the basemat. The remaining convective pressure is distributed among the buttresses and the area of the basin walls shielded by the buttresses.

RSW Piping Tunnels:

1. Initial design of the RSW Piping Tunnels as described in the COLA Part 2, Tier 2, Section 3H.6 and prior RAI responses was performed considering the following:

   - The walls and slabs of the tunnels were sized such that the out-of-plane frequency of each element spanning between its immediate supports exceeded 33 Hz. Thus, since the tunnels (with the exception of access shafts) are fully embedded, the in-structure amplification was considered negligible.
   - The designs of the walls and slabs of the tunnels were performed considering a Zero Period Acceleration (ZPA) of 0.21g which exceeds the site-specific ZPA of 0.13g. Use of 0.21g ZPA acceleration was judged to adequately account for in-structure amplification effect.

   In order to quantify the in-structure amplification and ensure that the use of 0.21g ZPA adequately accounted for in-structure amplification, a two-dimensional SSI analysis of the RSW tunnel is performed. Attached Figures 3H.6-138 and 3H.6-139 show the resulting amplified response spectra for all the walls and slabs of the tunnels. As can be seen from these figures, the amplified ZPA of these response spectra are less than 0.21g ZPA acceleration used for the design of tunnels' walls and slabs.

2. As noted in item 1 above, 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 were determined using a conservative ZPA acceleration of 0.21g. Simple manual calculations were 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. The Elastic Solution Method per Section 3.5.3.2 of ASCE 4-98 was used to determine lateral seismic soil loads on the exterior walls. For lateral soil pressures used for design of RSW Piping Tunnels, please see Figure 3H.6-44 provided as part of Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated 12/30/2009).

   In general the walls and slabs were 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). Thus, the analysis did not use any model or soil springs. 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 were used to determine the required rebar in accordance with the requirements of ACI 349-97. For results of RSW Piping Tunnels design, please see Table 3H.6-6 provided as part of Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated 12/30/2009).

3. 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 for the RSW Tunnel 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 actual tensile axial strain is 2.864×10^{-4} in/in which is about 14% of the rebar yield strain of 2.069×10^{-3} in/in.

4. For consideration of axial strain, please see item 3 above. The transverse shear demands on the exterior walls due to out-of-plane loads were determined through manual calculations as described in item 2 above.

5. Please see item 1 above. As noted in item 1 a two dimensional SSI analysis is performed to determine in-structure amplifications.

6. Please see item 1 above.

COLA Sections 3H.6.5.3, 3H.6.6.1 and 3H.6.6.2.2 will be revised as shown below. Also, new Figures 3H.6-138 and 3H.6-139 will be added.

3H.6.5.3 Seismic Analysis of RSW Piping Tunnels

~~The RSW piping tunnel seismic analysis has been performed using an equivalent static approach, using the horizontal and vertical Input Spectra defined in Subsection 3H.6.5.1.1.1. The concrete elements of the RSW piping tunnel are sized such that the structure is rigid with a minimum frequency exceeding 33 Hz. The structure is buried inside the soil. Since the minimum structural frequency of the RSW piping tunnel exceeds 33 Hz, in-structure amplification will not take place and, therefore, the Input Spectra can be used as in-structure response spectra.~~ 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 results of the RSW Tunnel design are summarized in Table 3H.6-6.~~

3H.6.6.1 Analytical Models

The structural analysis and design of the UHS basin and the RSW pump house was performed using a finite element model (FEM). The FEM model is shown in Figure 3H.6-40. The analysis for the seismic loads was performed using equivalent static loads and the induced forces due to the X, Y, and Z seismic excitations were combined using the SRSS method of combination. For the portions of the UHS basin where liquid-tightness is required (i.e., exterior walls and basemat of the basin), in addition to satisfying ACI 349 strength requirements, the required strength was increased by the environmental durability factors noted in Subsection 3H.6.4.3.4.3 per Section 9.2.8 of ACI 350-01. Detailed stability evaluations were performed for sliding, overturning, and flotation. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for consideration of the X, Y, and Z seismic excitations. ~~The RSW piping tunnel has been analyzed using an equivalent static approach for the seismic loads, as described in Subsection 3H.6.5.3.~~

3H.6.6.2.2 RSW Piping Tunnels

~~An equivalent static analysis was performed for the RSW piping tunnels (see Subsection 3H6.5.3).~~ 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 2.9×10^{-4} in/in which is about 14% of the rebar yield strain of 2.069×10^{-3} in/in. This analysis considered the loads identified below, combined in accordance with Subsection 3H.6.4.3.4. ~~In addition, SSE strains created in the tunnel walls due to the passage of seismic waves through the soil were computed using method of ASCE 4-98 Subsection 3.5.2.1. The maximum computed strain was 2.9×10^{-4} in/in for the Input Spectrum described in Section 3H.6.5.1.1.1. This strain is less than the yield strain of the reinforcing steel.~~

- 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.
- Dynamic lateral soil pressures on the tunnel walls due to an SSE calculated using the

methodology defined in Subsection 3.5.3.2.2 of ASCE 4. Lateral soil pressures used for design of RSW Piping Tunnels are presented in Figure 3H.6-44.

- 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.

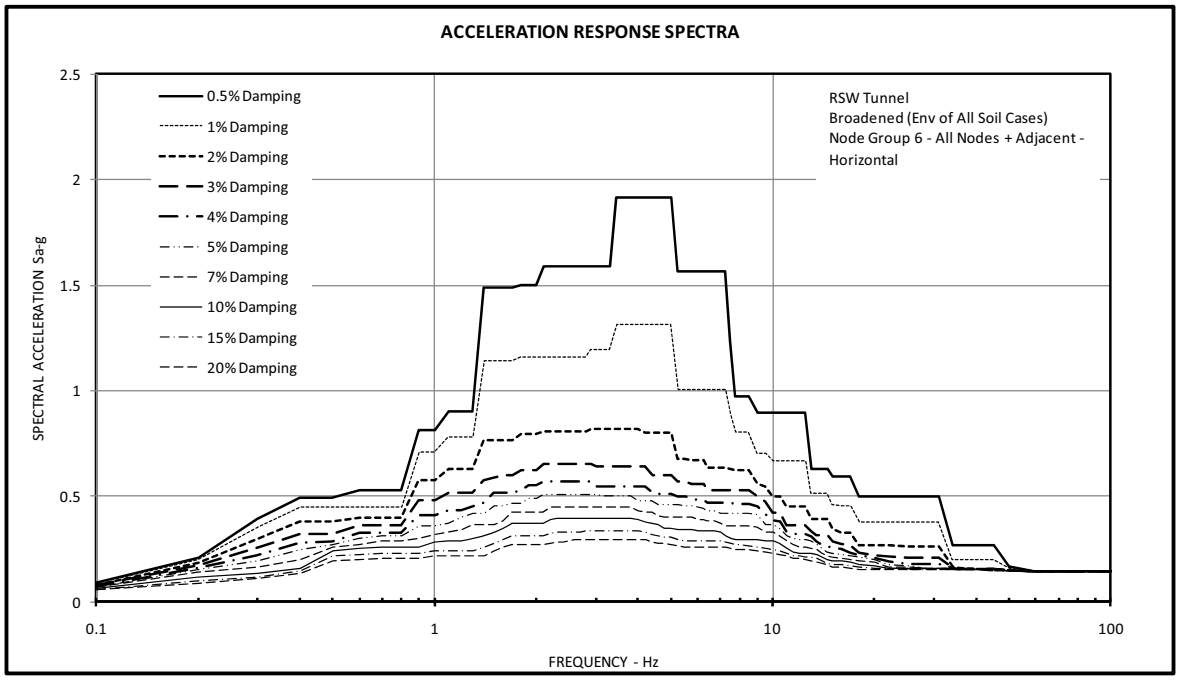


Figure 3H.6-138: Broadened Horizontal FRS for RSW Piping Tunnels

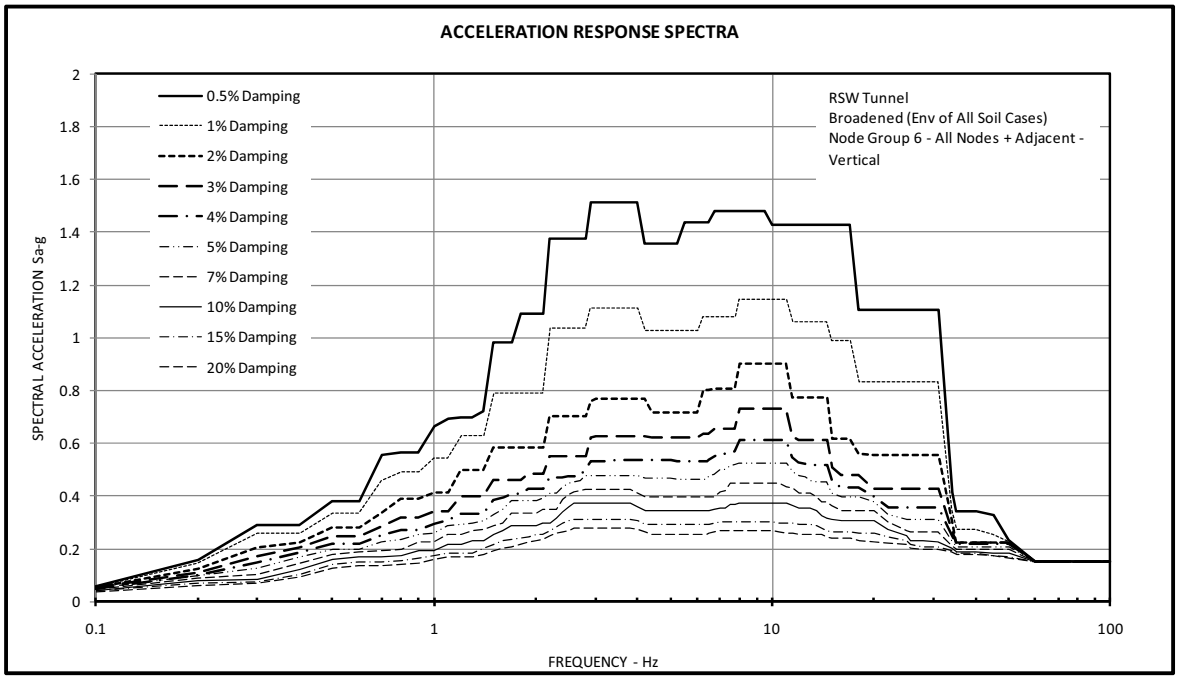


Figure 3H.6-139: Broadened Vertical FRS for RSW Piping Tunnels

RAI 03.07.02-16

QUESTION:

(Follow-up Question to RAI 03.07.02-4)

1. In the response to Item 2 of RAI 03.07.01-4, the finite element model of the UHS basin and RSW Pump House was not provided in Figure 3H.6-40 (this figure was supposed to be part of the response to RAI 03.07.01-13). The applicant is requested to ensure this information be part of Figure 3H.6-40, including a plot showing the basement slab and soil-retaining wall mesh configuration and grid sizes. The response to Item 2 of RAI 03.07.01-4 also states that, "***The model mesh size is detailed enough to model the principal features of the structure and transmit a frequency of at least 33 Hz***." The applicant is requested to a) provide the criteria and quantitative basis to show that the element sizes are sufficiently small to transmit frequencies of up to 33 Hz for the three soil cases; and b) provide a justification that the aspect ratio of the elements is sufficiently small as not to affect the solution accuracy.

2. In the response to Item 5 of RAI 03.07.01-4, the analytical model for the RSW Piping Tunnel is not provided. As such the applicant is requested to provide the analytical model used for analysis of the RSW Piping Tunnel.

RESPONSE:

1. Figure 3H.6-40 has been provided as part of the Supplement 1 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/2009) and for convenience is reproduced below. Note that this figure provides the Finite Element Model (FEM) used for design of the ultimate head sink (UHS) and reactor service water (RSW) pump house. In this FEM, the basemat is represented by shell elements, whereas in the FEM model for the soil-structure interaction (SSI) analysis (see Figure 3H.6-15a) the basemat is represented by solid elements. Solid elements are required for proper modeling of interaction between the soil and the structure in the SSI analysis.

   The following discussion provides response to parts (a) and (b) of this question.

   A mesh sensitivity analysis was performed to examine the adequacy of model mesh. This sensitivity analysis was performed by dividing each element into four elements. Sensitivity analysis results are presented for two representative walls. Figures 03.07.02-16a1 through 03.07.02-16a14 show comparison of the resulting membrane and out-of-plane forces and moments for the UHS basin west wall under 5 ksf surface loading. Similarly, Figures 03.07.02-16b1 through 03.07.02-16b14 show comparison of the membrane and out-of-plane forces and moments for the UHS fan enclosure south wall under 2 ksf surface loading. These comparisons show that the results are similar and the mesh used for the design of the UHS/RSW Pump House is acceptable.

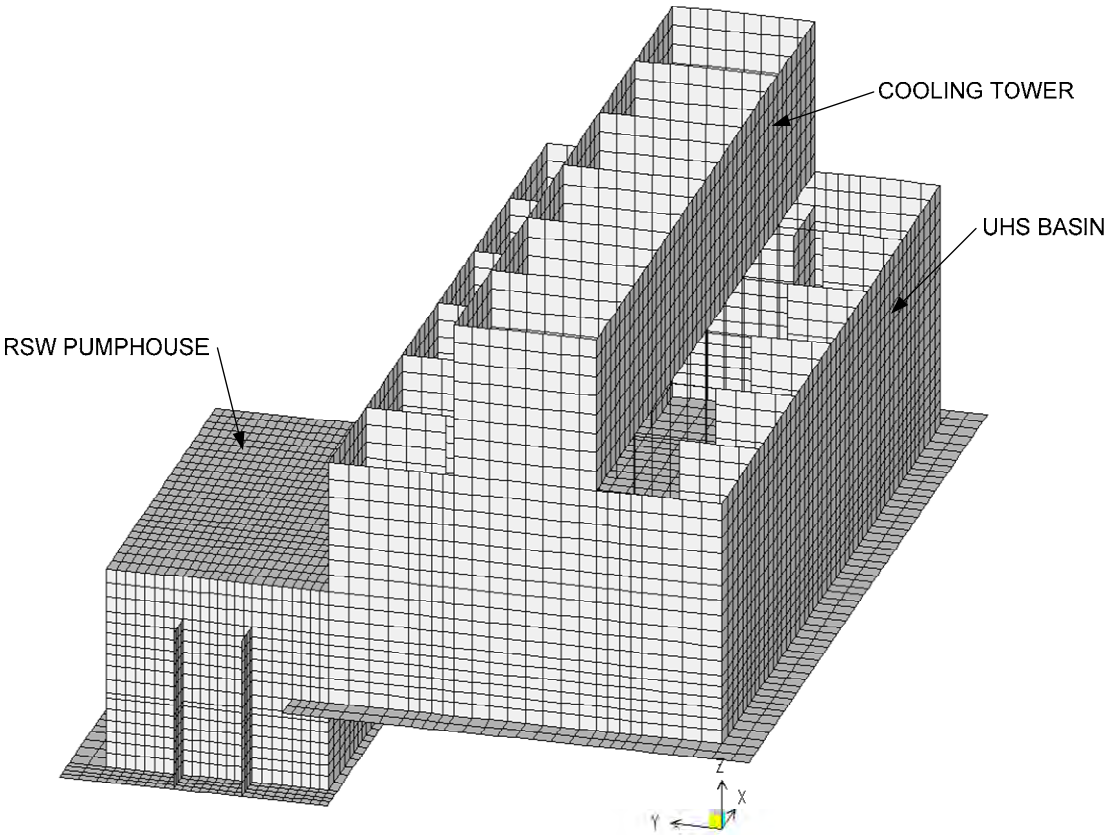
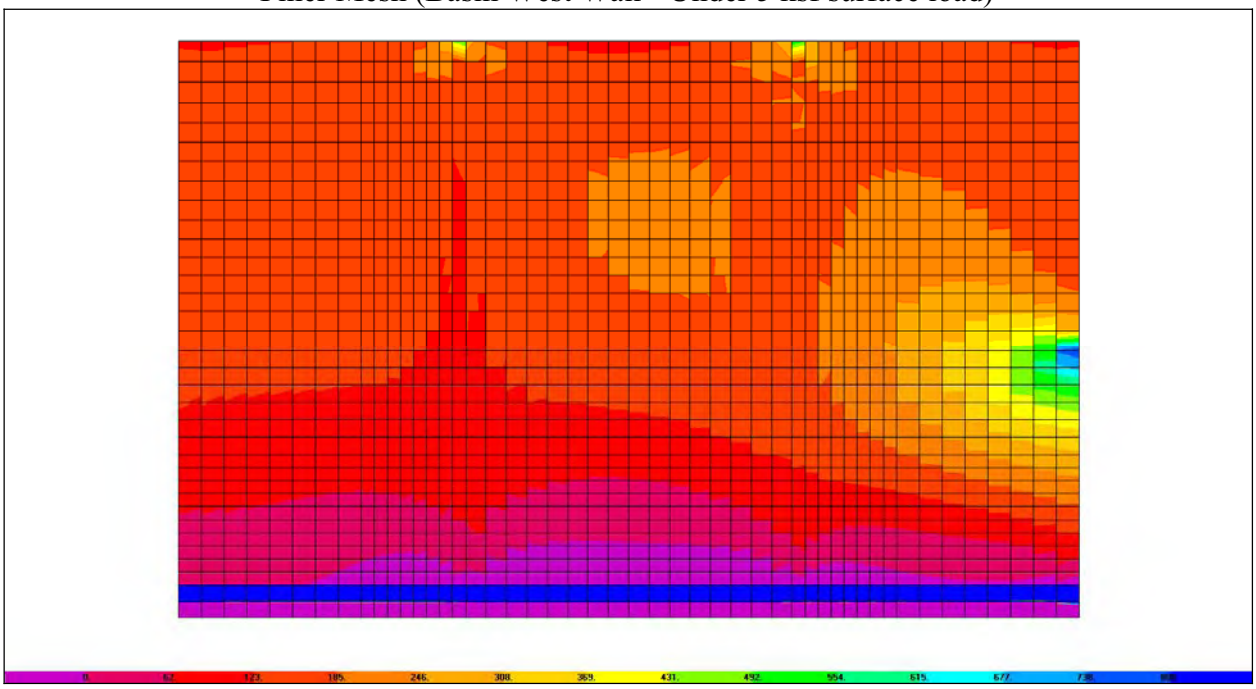


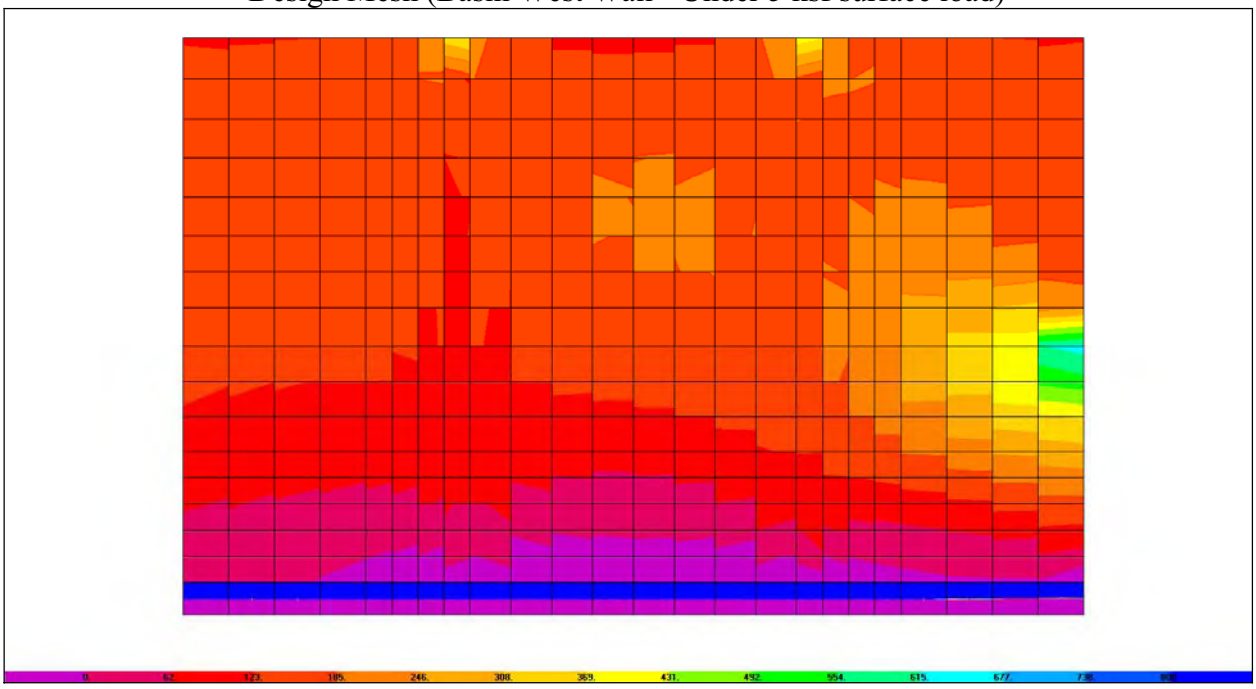
Figure 3H.6-40: SAP Finite Element Model for UHS and RSW Pump House Design

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



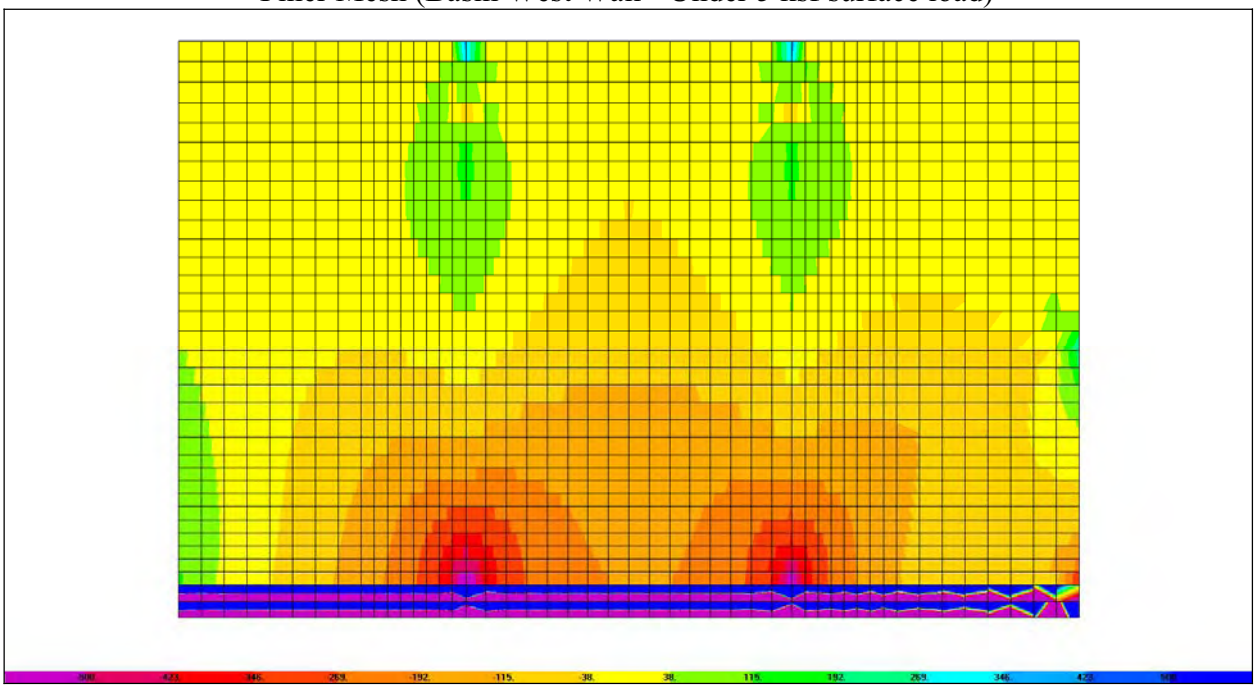
F11
Scale 0 kip/ft to 800 kip/ft
Figure 03.07.02-16a1

Design Mesh (Basin West Wall - Under 5 ksf surface load)



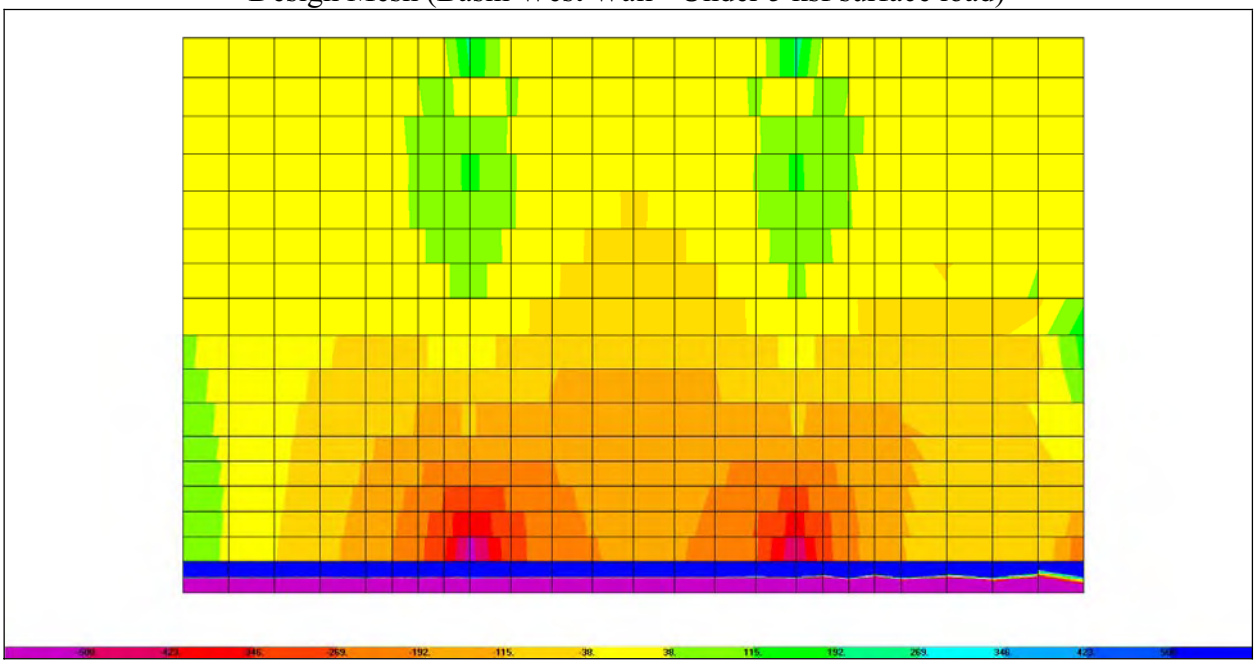
F11
Scale 0 kip/ft to 800 kip/ft
Figure 03.07.02-16a2

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



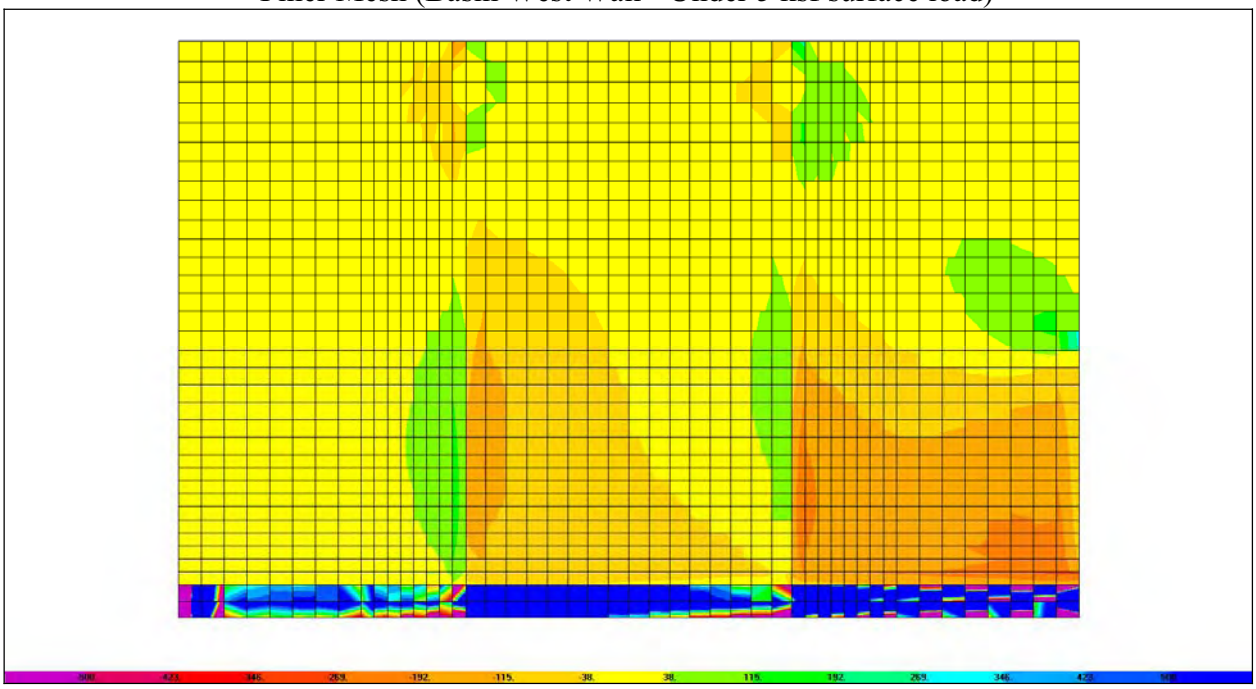
F22
Scale -500 kip/ft to 500 kip/ft
Figure 03.07.02-16a3

Design Mesh (Basin West Wall - Under 5 ksf surface load)



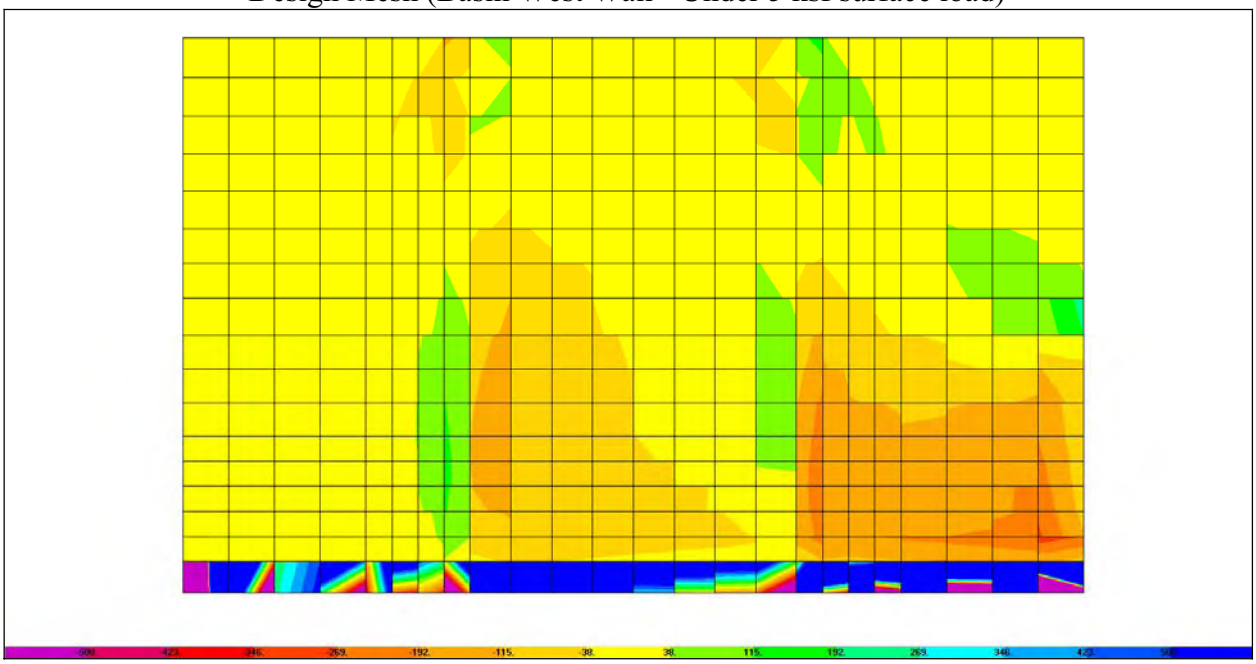
F22
Scale -500 kip/ft to 500 kip/ft
Figure 03.07.02-16a4

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



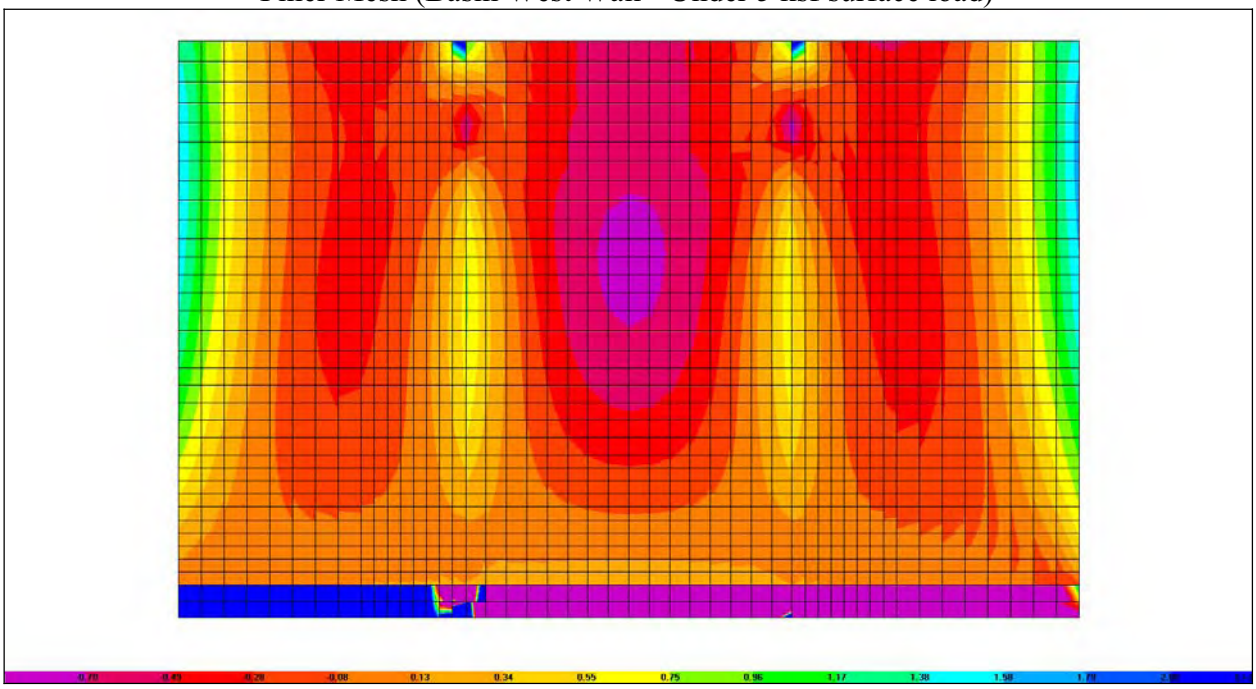
F12
Scale -500 kip/ft to 500 kip/ft
Figure 03.07.02-16a5

Design Mesh (Basin West Wall - Under 5 ksf surface load)



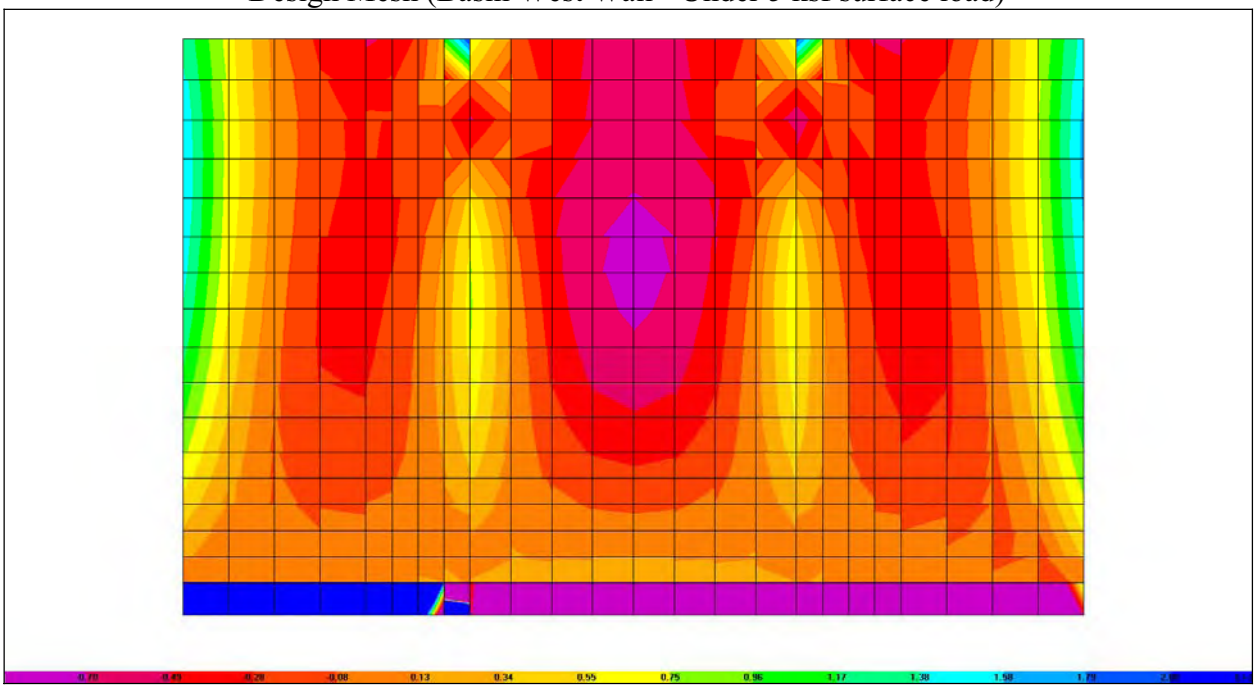
F12
Scale -500 kip/ft to 500 kip/ft
Figure 03.07.02-16a6

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



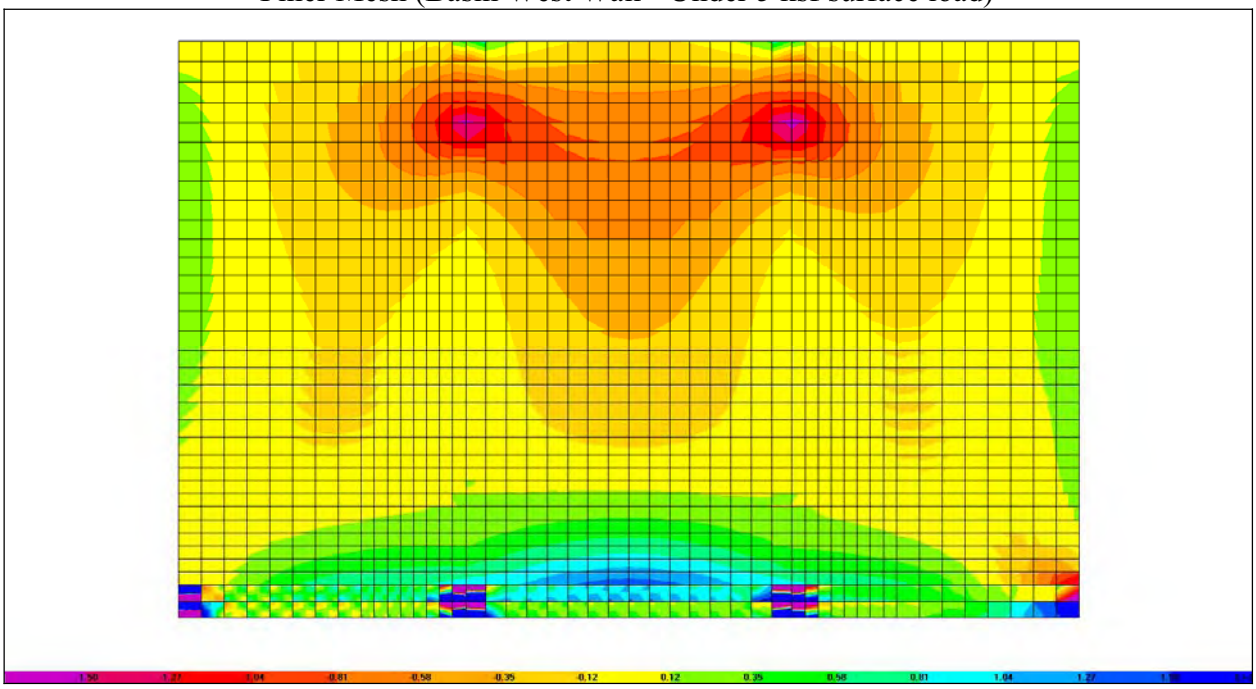
M11
Scale -700 kip-ft/ft to 2000 kip-ft/ft
Figure 03.07.02-16a7

Design Mesh (Basin West Wall - Under 5 ksf surface load)



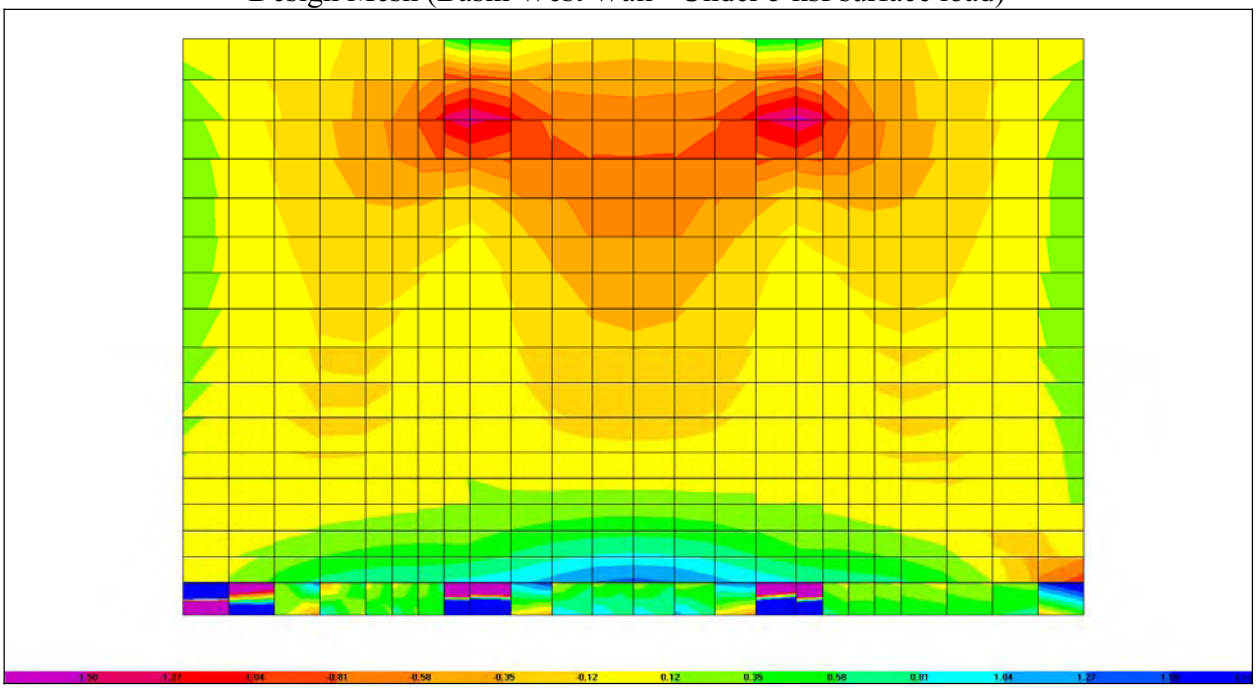
M11
Scale -700 kip-ft/ft to 2000 kip-ft/ft
Figure 03.07.02-16a8

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



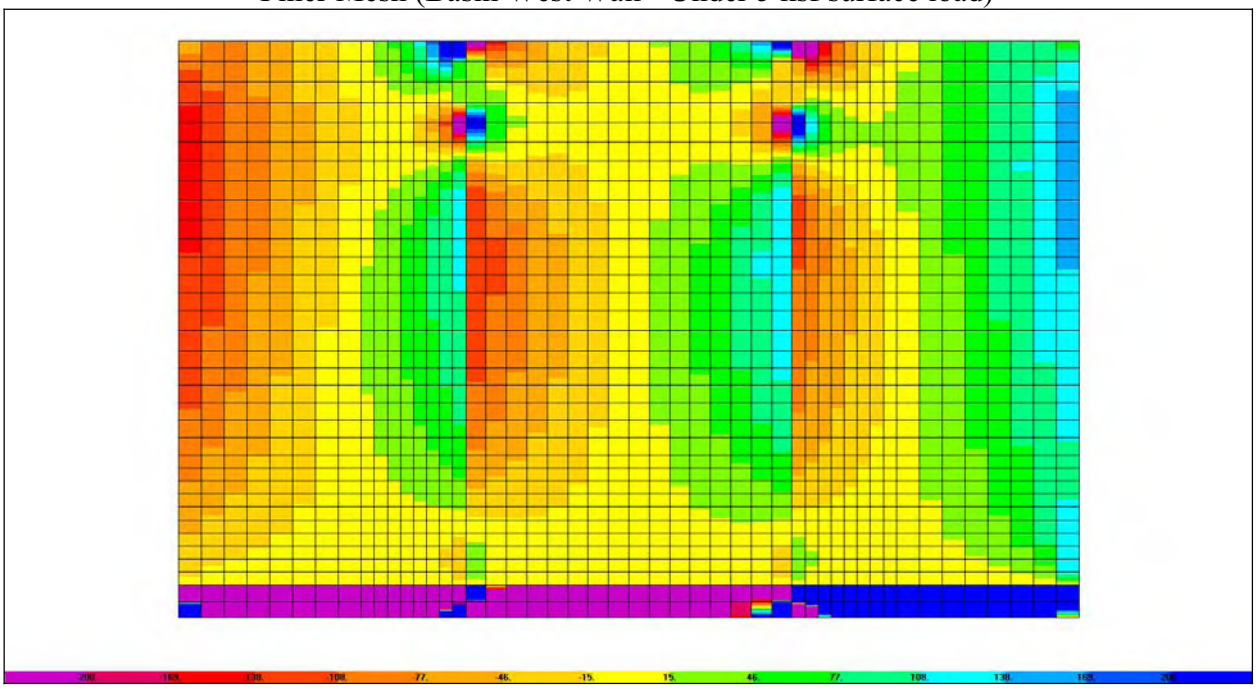
M22
Scale -1500 kip-ft/ft to 1500 kip-ft/ft
Figure 03.07.02-16a9

Design Mesh (Basin West Wall - Under 5 ksf surface load)



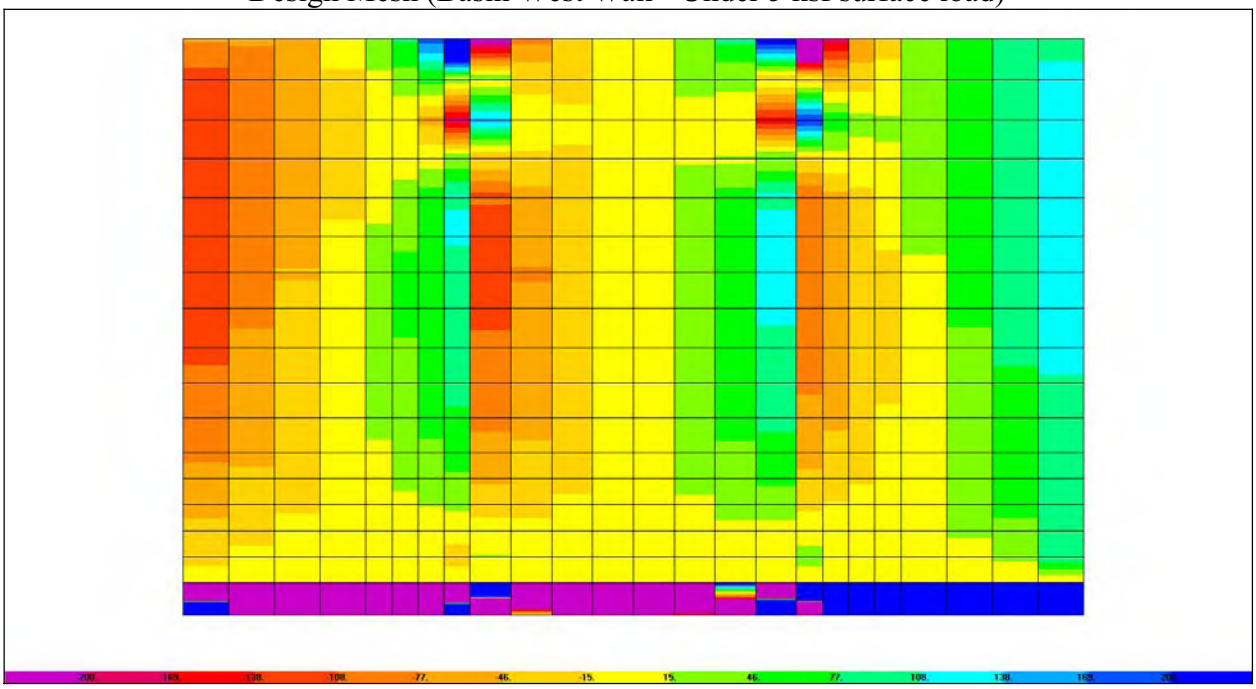
M22
Scale -1500 kip-ft/ft to 1500 kip-ft/ft
Figure 03.07.02-16a10

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



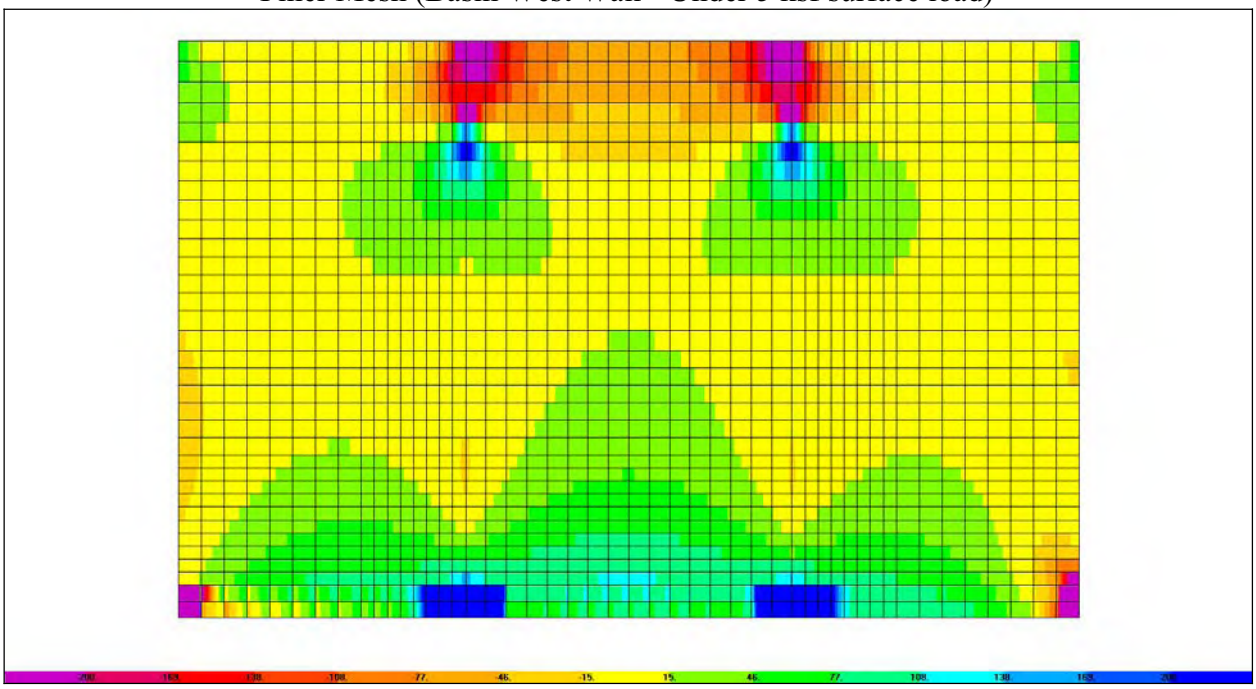
V13
Scale -200 kip/ft to 200 kip/ft
Figure 03.07.02-16a11

Design Mesh (Basin West Wall - Under 5 ksf surface load)



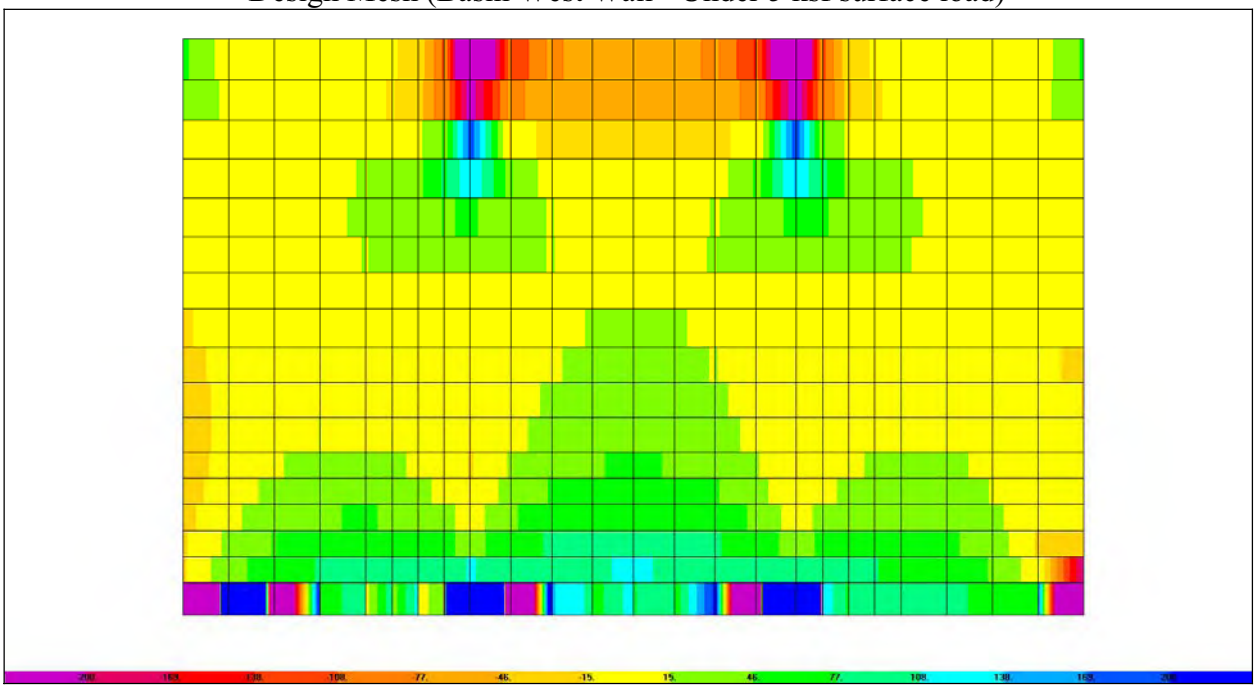
V13
Scale -200 kip/ft to 200 kip/ft
Figure 03.07.02-16a12

Finer Mesh (Basin West Wall - Under 5 ksf surface load)



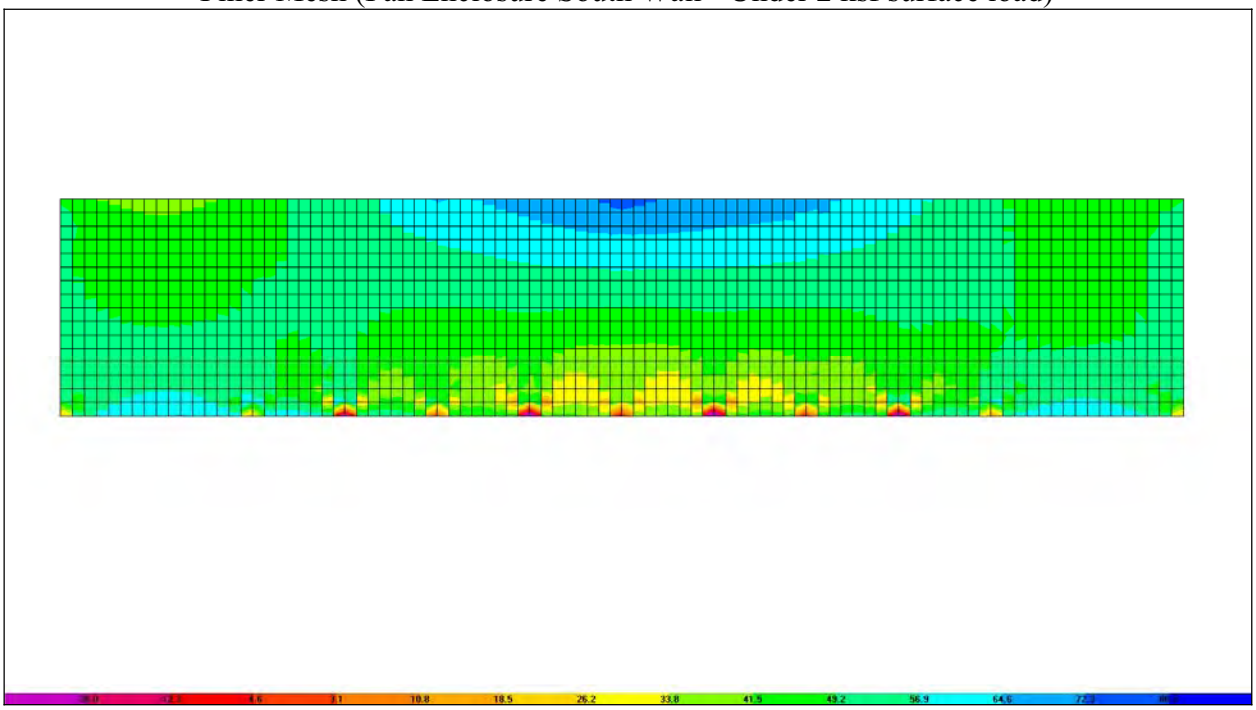
V23
Scale -200 kip/ft to 200 kip/ft
Figure 03.07.02-16a13

Design Mesh (Basin West Wall - Under 5 ksf surface load)



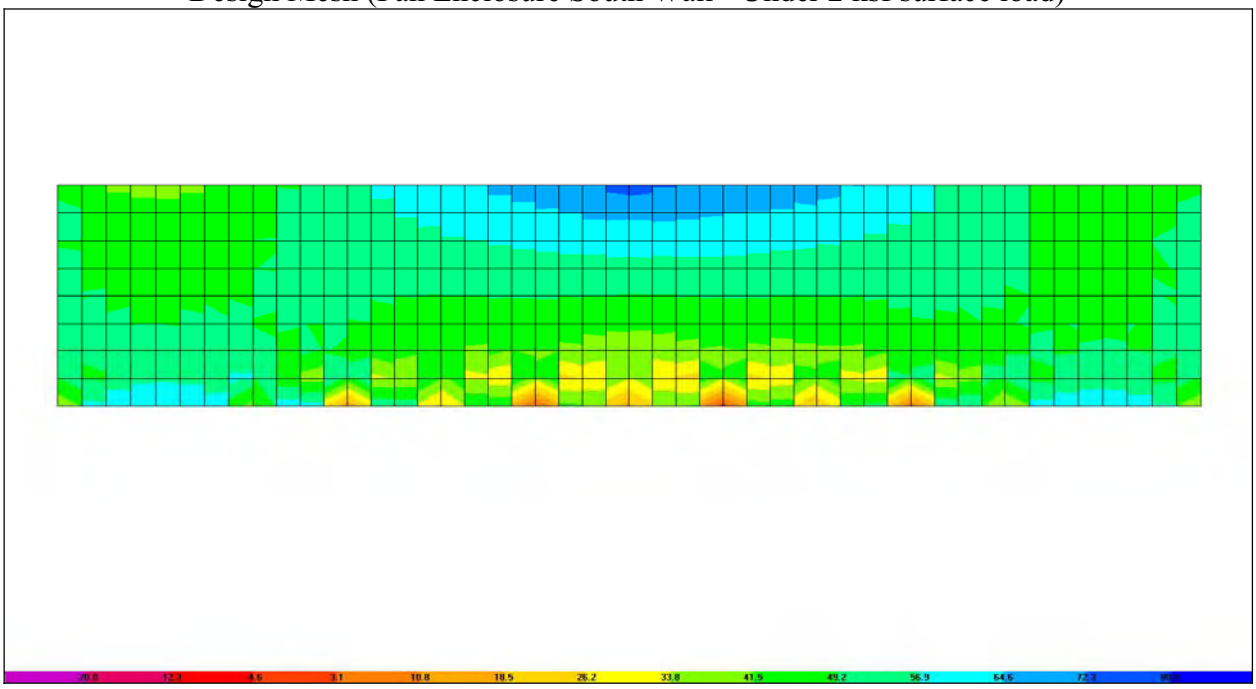
V23
Scale -200 kip/ft to 200 kip/ft
Figure 03.07.02-16a14

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



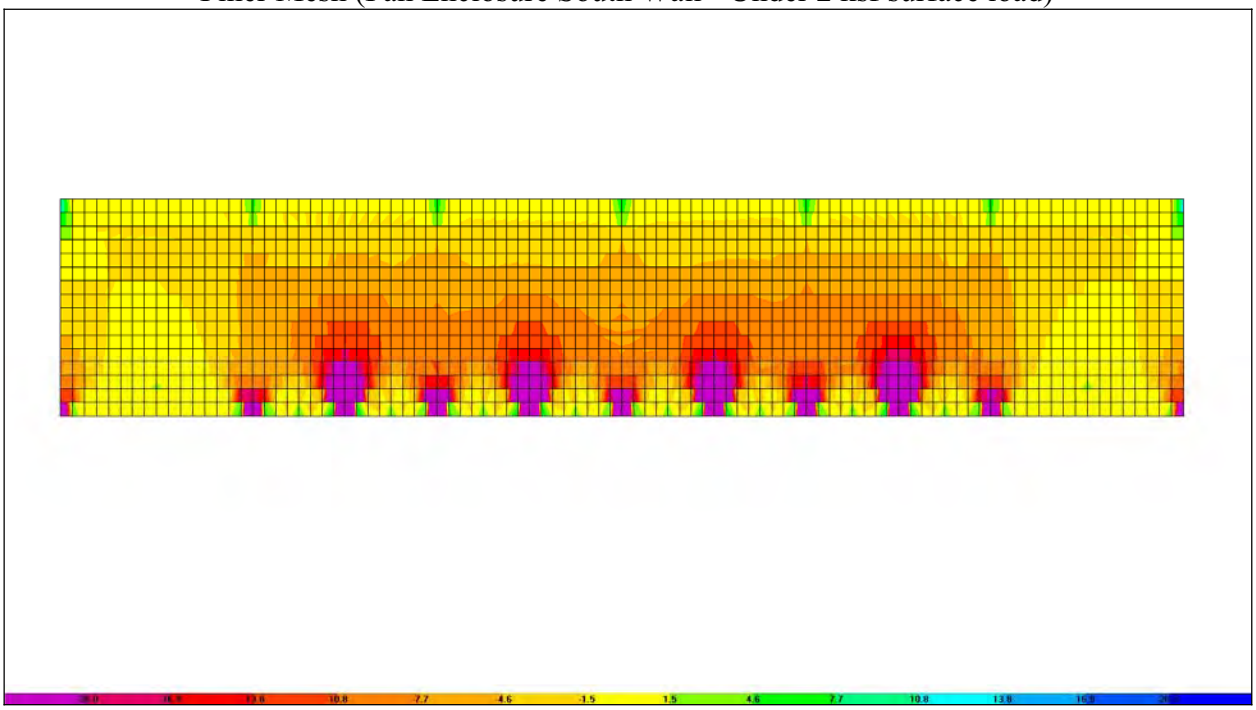
F11
Scale -20 kip/ft to 80 kip/ft
Figure 03.07.02-16b1

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



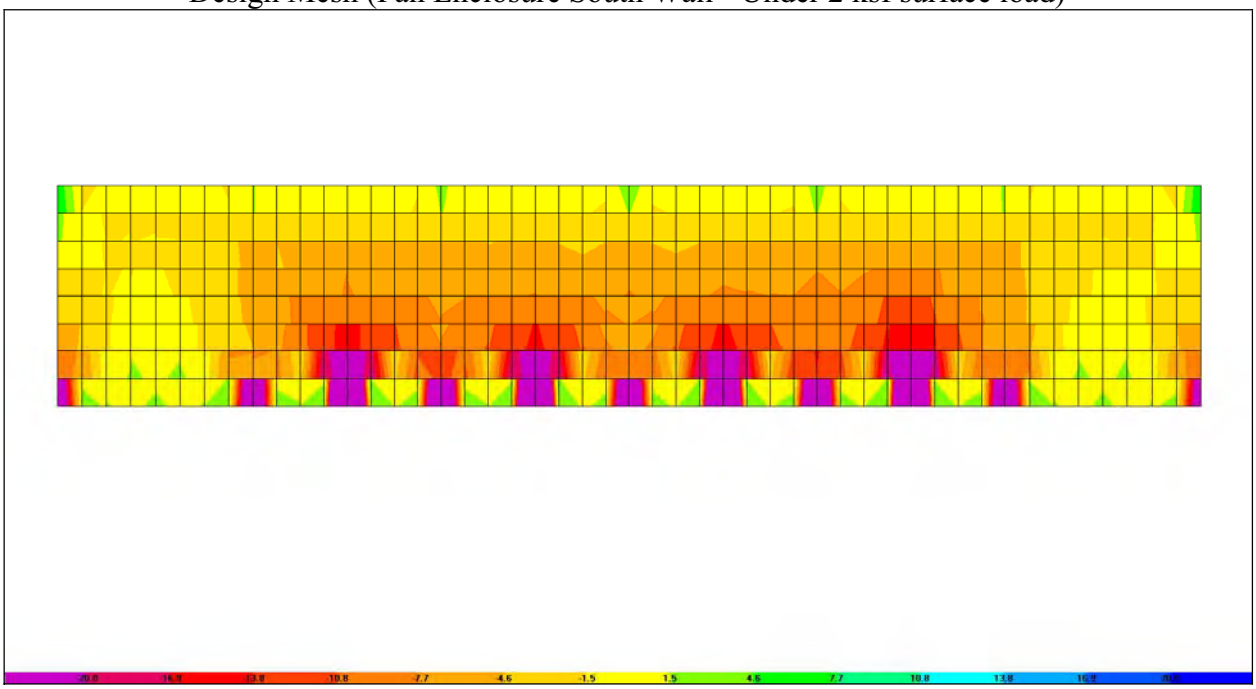
F11
Scale -20 kip/ft to 80 kip/ft
Figure 03.07.02-16b2

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



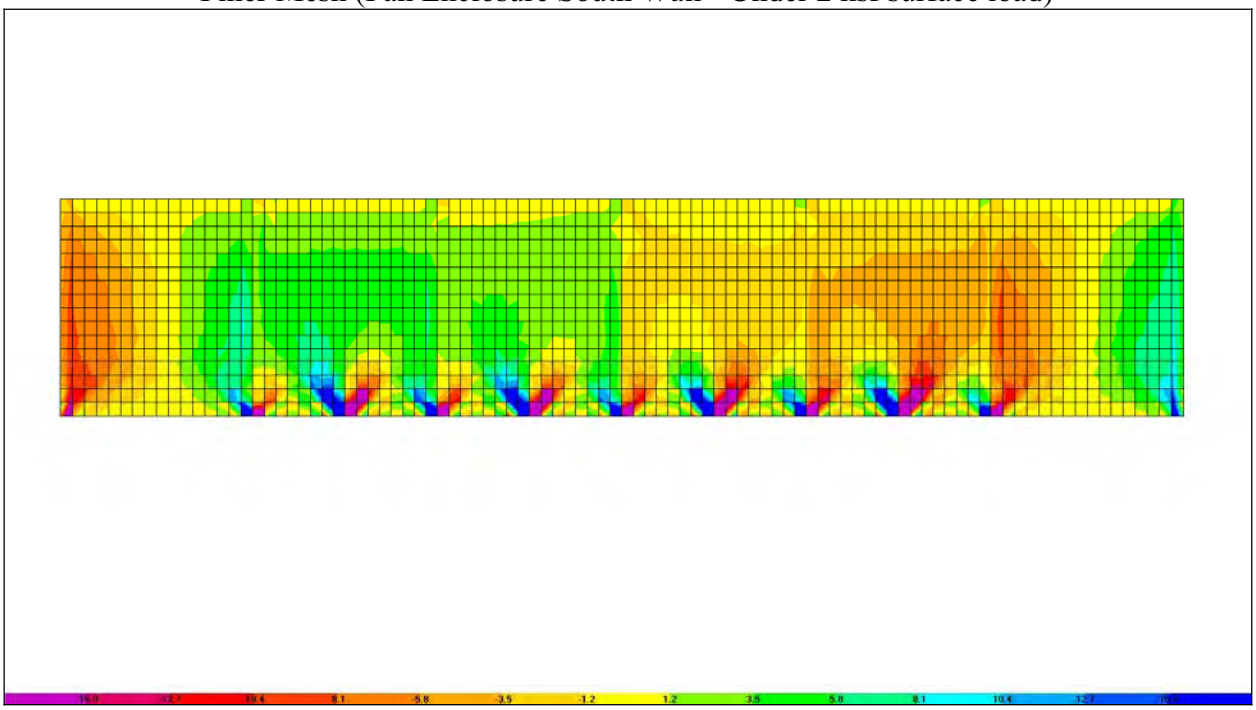
F22
Scale -20 kip/ft to 20 kip/ft
Figure 03.07.02-16b3

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



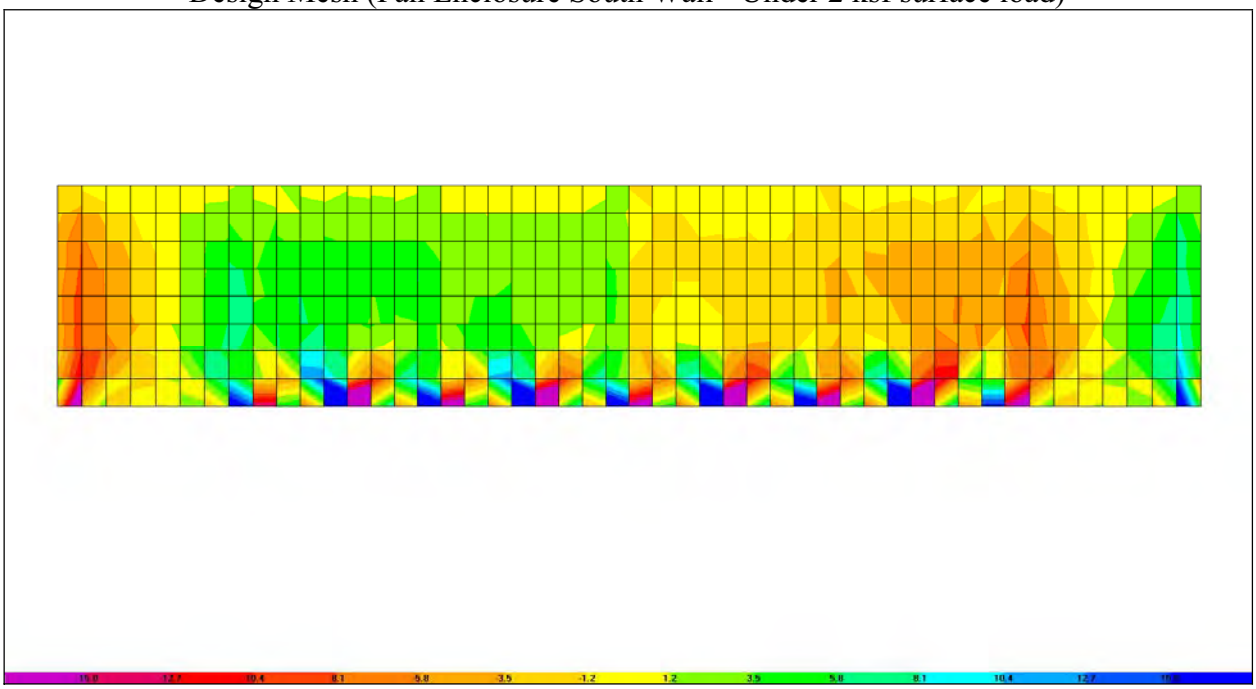
F22
Scale -20 kip/ft to 20 kip/ft
Figure 03.07.02-16b4

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



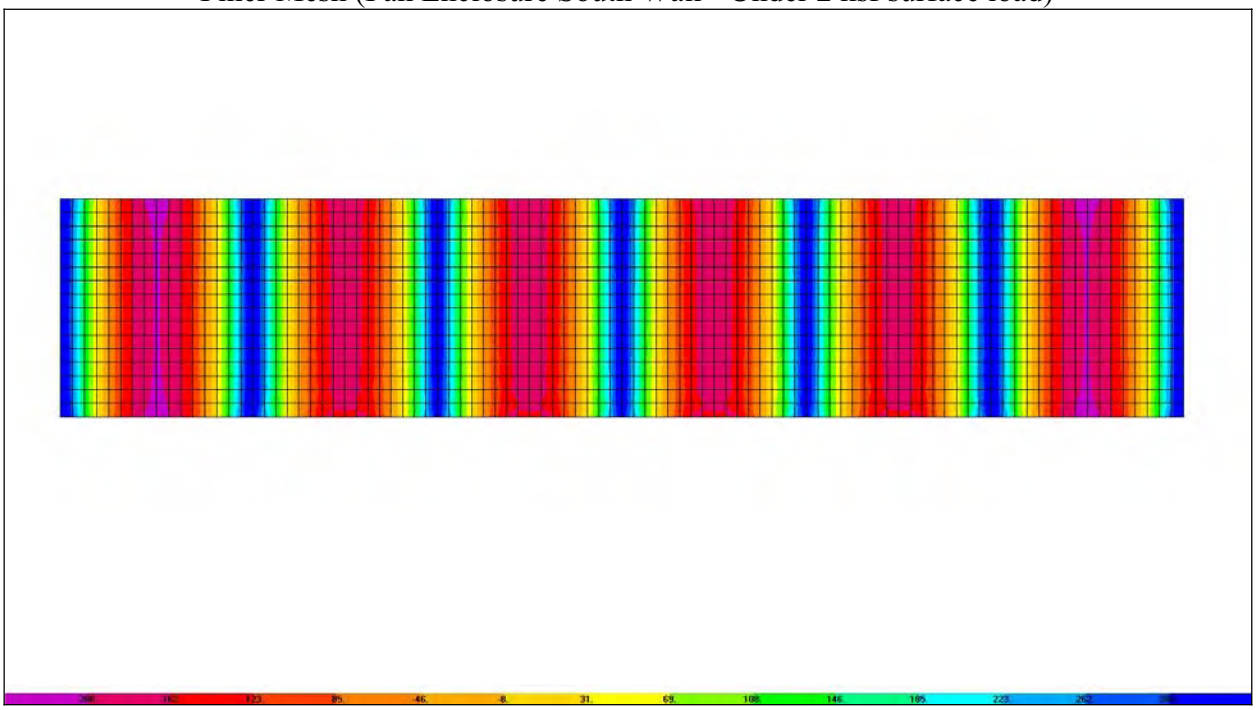
F12
Scale -15 kip/ft to 15 kip/ft
Figure 03.07.02-16b5

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



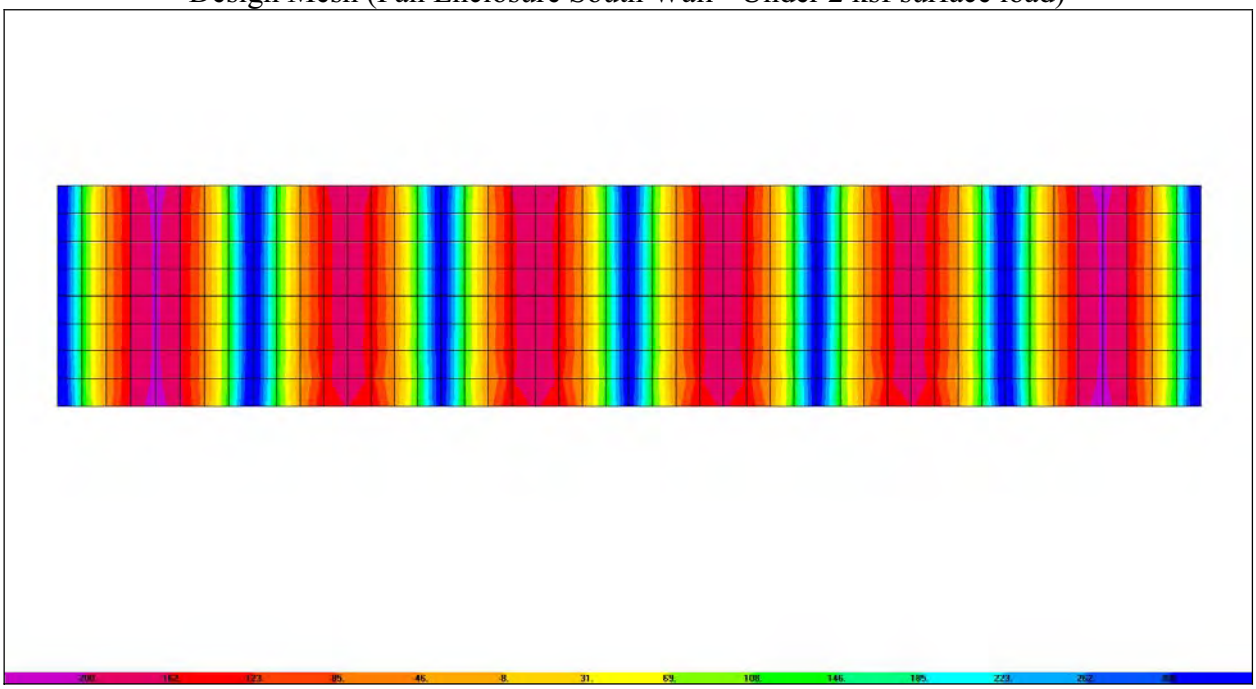
F12
Scale -15 kip/ft to 15 kip/ft
Figure 03.07.02-16b6

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



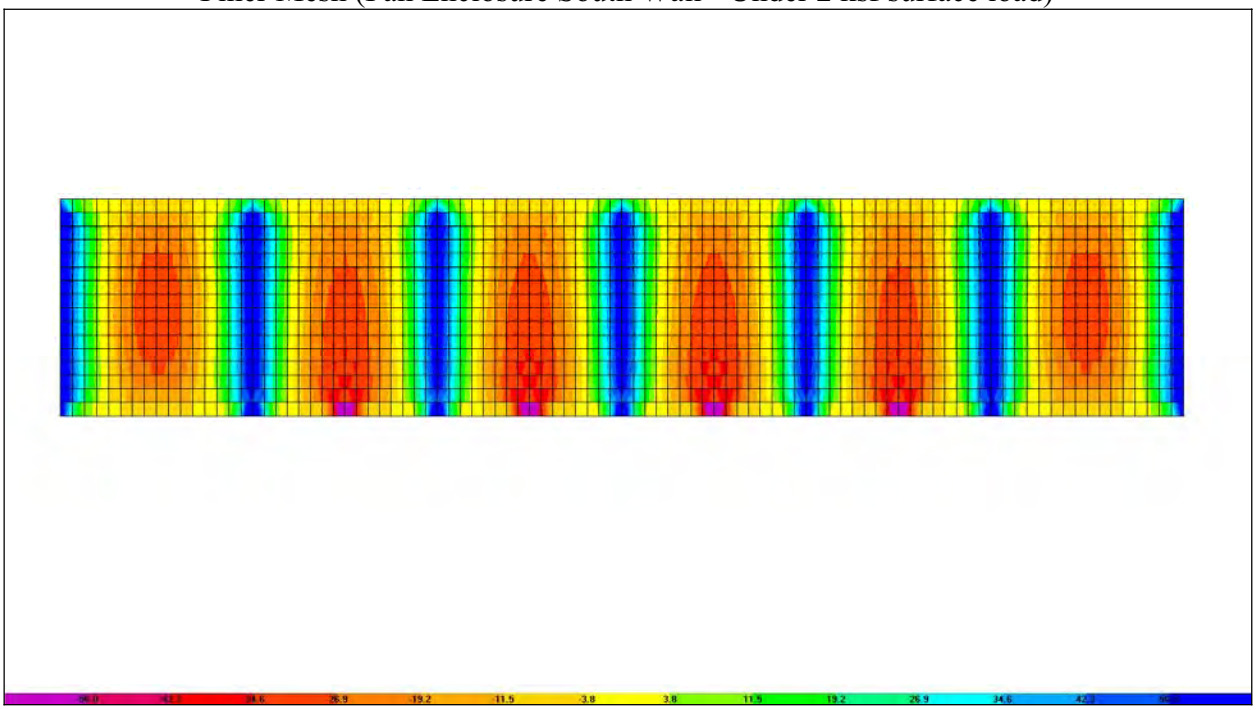
M11
Scale -200 kip-ft/ft to 300 kip-ft/ft
Figure 03.07.02-16b7

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



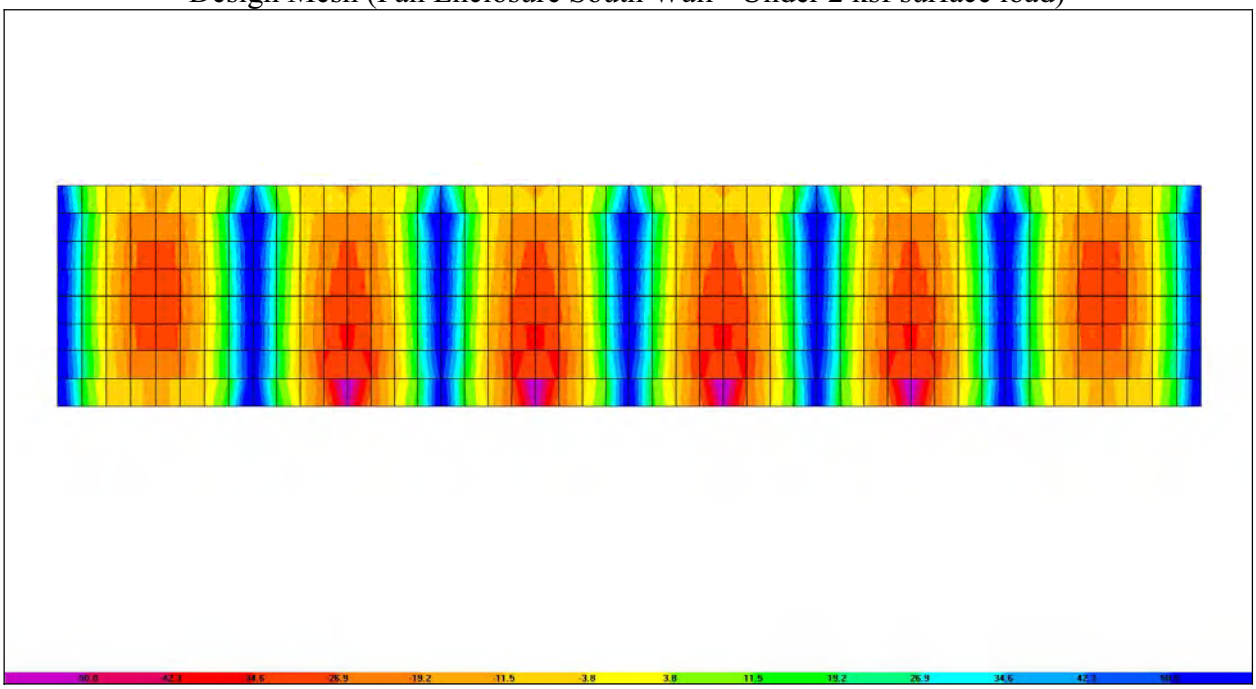
M11
Scale -200 kip-ft/ft to 300 kip-ft/ft
Figure 03.07.02-16b8

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



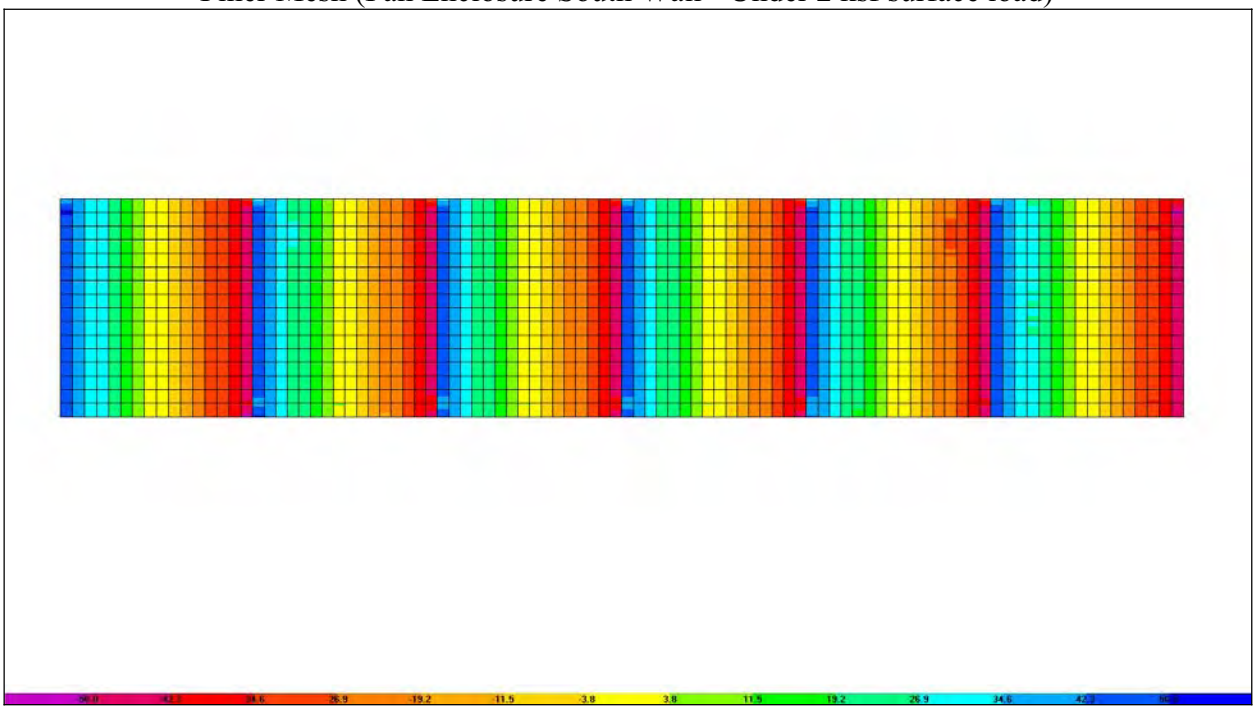
M22
Scale -50 kip-ft/ft to 50 kip-ft/ft
Figure 03.07.02-16b9

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



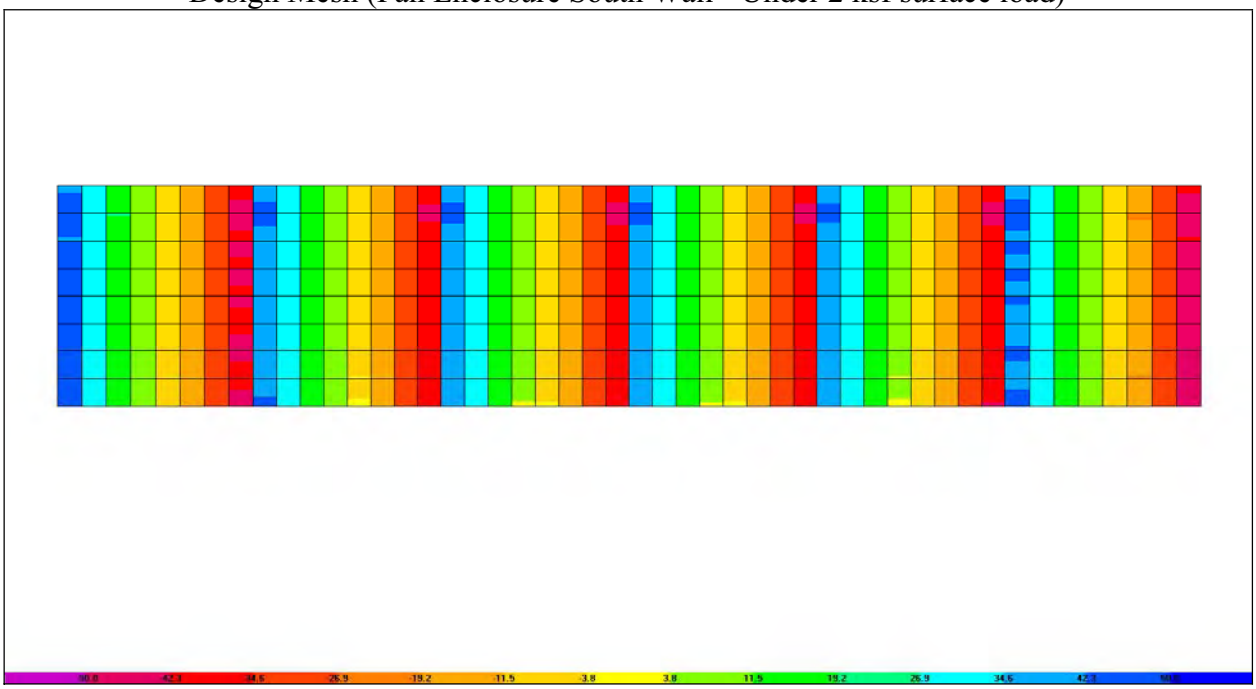
M22
Scale -50 kip-ft/ft to 50 kip-ft/ft
Figure 03.07.02-16b10

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



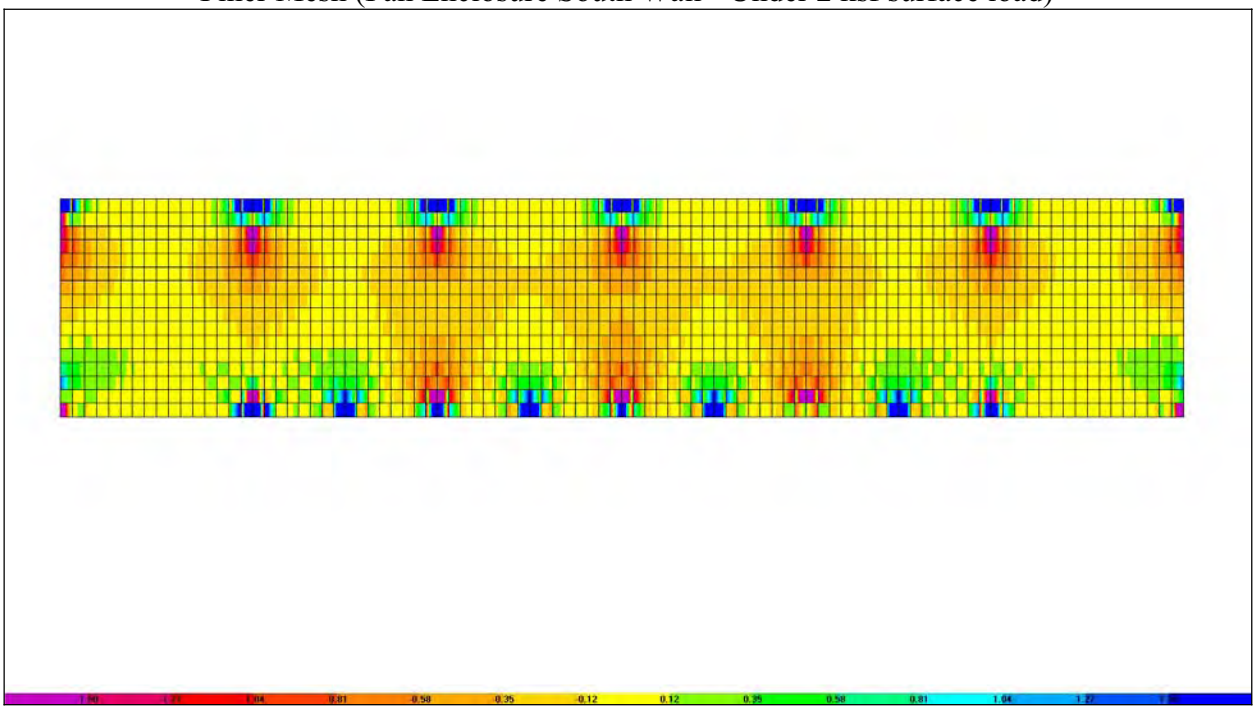
V13
Scale -50 kip/ft to 50 kip/ft
Figure 03.07.02-16b11

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



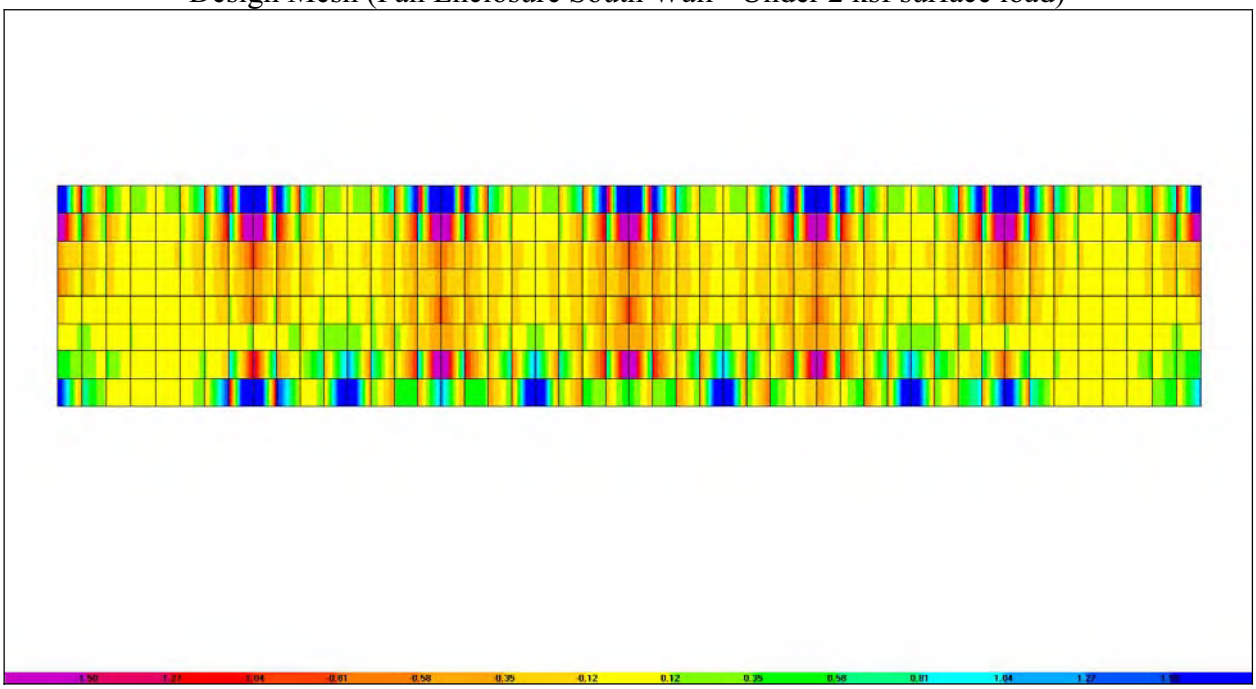
V13
Scale -50 kip/ft to 50 kip/ft
Figure 03.07.02-16b12

Finer Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



V23
Scale -1.5 kip/ft to 1.5 kip/ft
Figure 03.07.02-16b13

Design Mesh (Fan Enclosure South Wall - Under 2 ksf surface load)



V23
Scale -1.5 kip/ft to 1.5 kip/ft
Figure 03.07.02-16b14

Figures 3H.6-15a through 3H.6-15g on the following pages provide the structure model used in the SSI analysis along with the additional requested information regarding mesh configuration and grid size of the basemat and the exterior walls.

With regards to adequacy of soil layer thicknesses for transmittal frequency, please see the response to RAI 03.07.02-17 (see letter U7-C-STP-NRC-100035 dated 2/4/2010).

The structure model mesh size used in the SSI analysis is sufficiently small to transmit frequencies of at least 33 Hz for the corresponding concrete properties. The transmittal frequency (F) is defined as

$$F = V_s / (5*H)$$

Where, V_s is the shear wave velocity of concrete material and H is the maximum mesh size of the structural elements.

For a concrete structure with a shear modulus of 221538.5 ksf and density of 0.15 kcf, for transmittal frequency of at least 33 Hz, the maximum allowable mesh size is 41.8 ft. In the finite element model of the UHS basin and RSW Pump House, all mesh sizes are less than 41.8 ft. The industry practice is to keep the aspect ratio of the finite elements within a ratio of 1 to 4. In general, the aspect ratios of the finite elements in the SSI model are kept within the ratio of 1 to 2, except at the soft soil layer where the average aspect ratio is about 1 to 4.

A mesh sensitivity analysis was performed to examine model frequency and mass participation. This sensitivity analysis was performed by dividing each element into 4 elements. Figures 03.07.02-16c1 through 03.07.02-16c4 provide the frequency and mass participation for the major modes in the E-W and N-S directions. Comparison of these figures shows that the frequencies and mass participation from the two models are similar, and thus the structural model used in the SSI analysis is adequate.

Design Mesh – Mode 1
Major Mode in E-W direction
Frequency = 2.133 Hz
17.1% mass participation in E-W direction

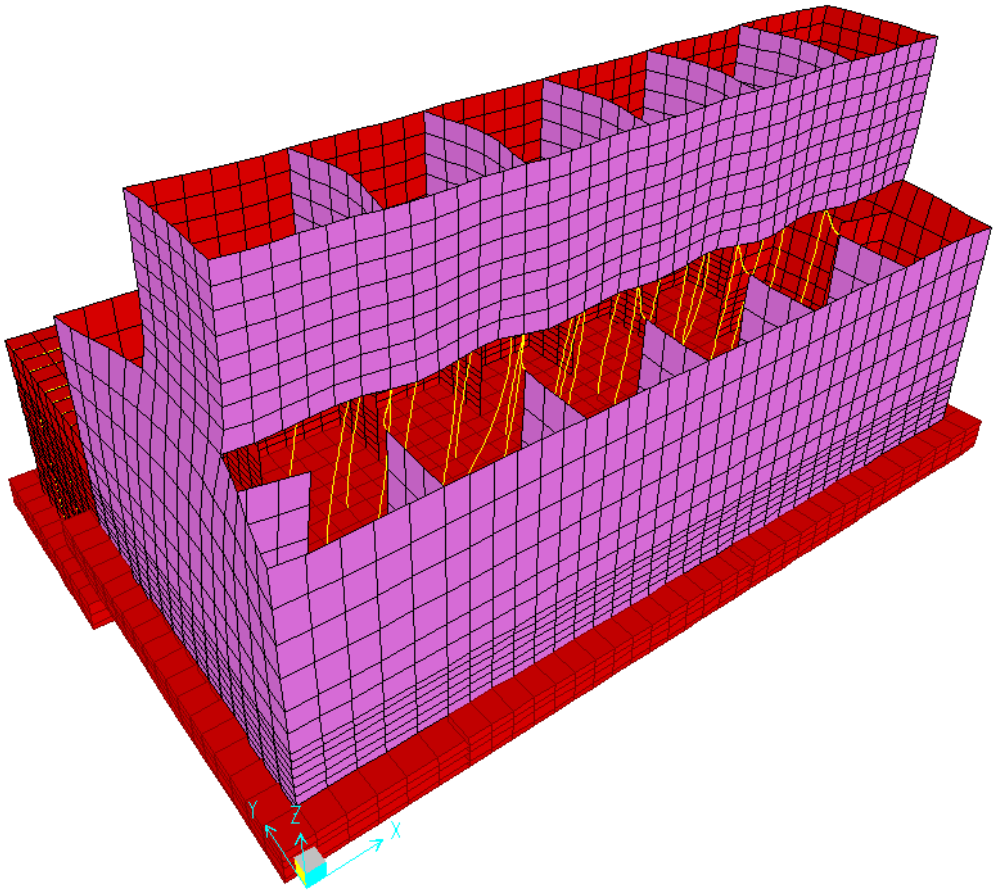


Figure 03.07.02-16c1

Refined Mesh – Mode 1
Major Mode in E-W direction
Frequency = 2.056 Hz
17.2% mass participation in E-W direction

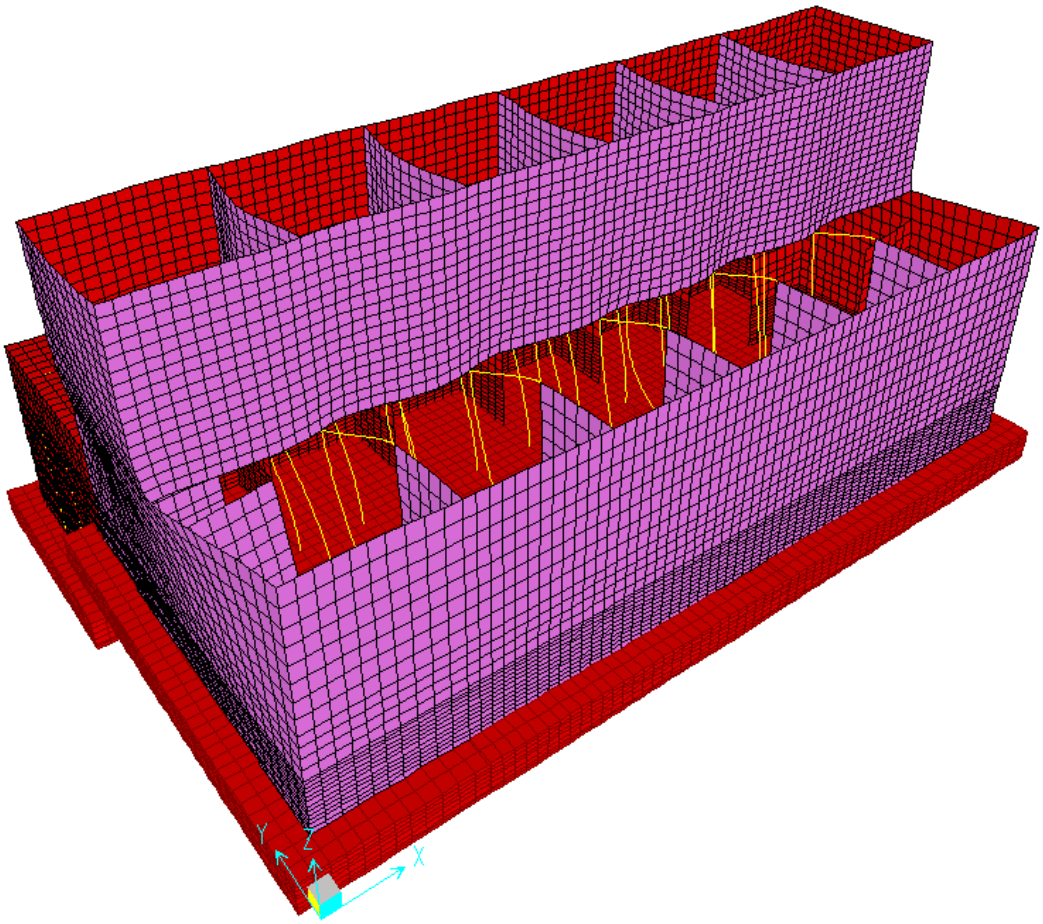


Figure 03.07.02-16c2

Design Mesh – Mode 4
Major Mode in N-S direction
Frequency = 3.187 Hz
15.4% mass participation in N-S direction

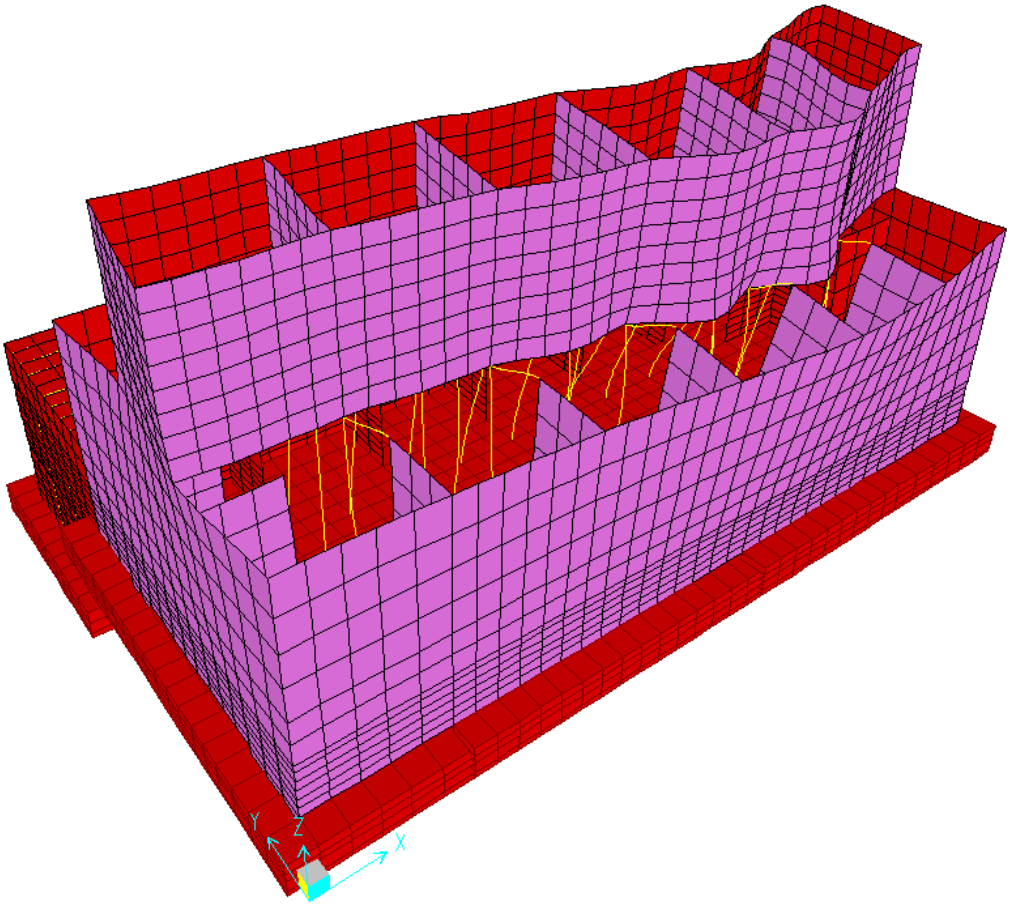


Figure 03.07.02-16c3

Refined Mesh – Mode 4
Major Mode in N-S direction
Frequency = 3.028 Hz
16.9% mass participation in N-S direction

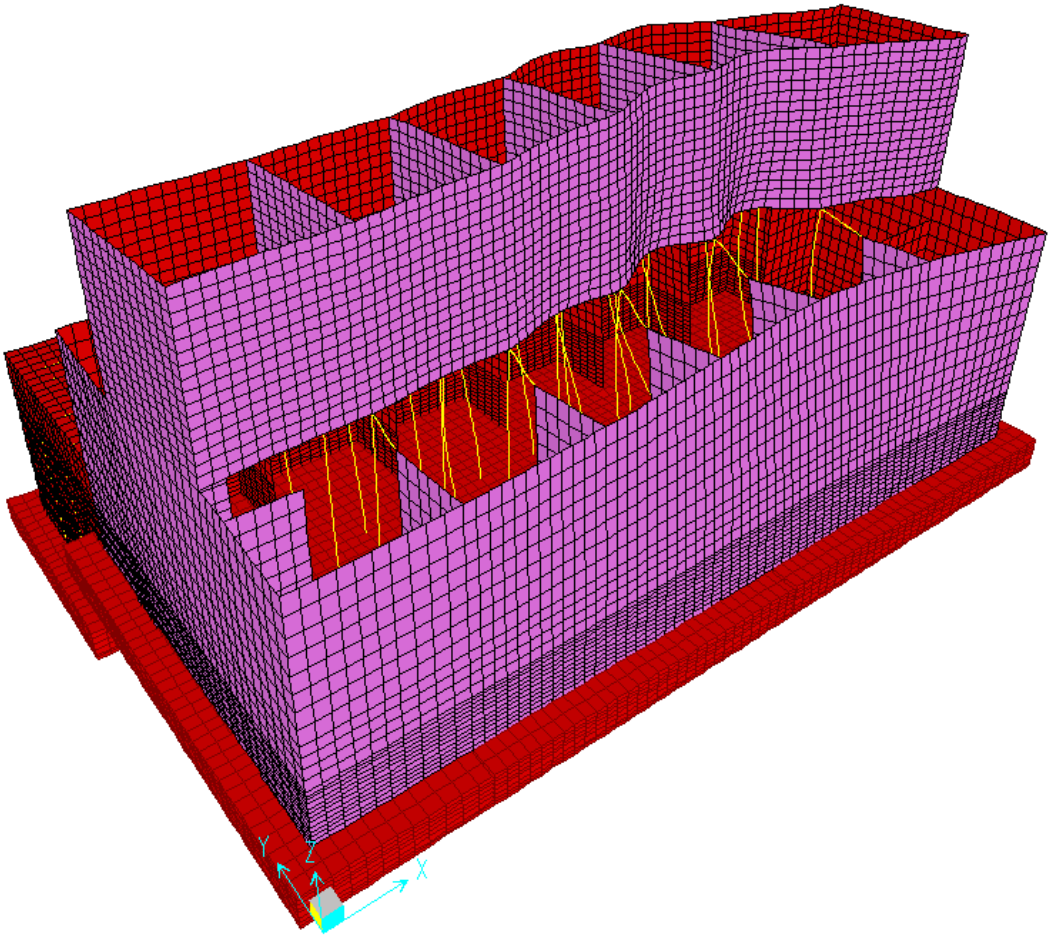


Figure 03.07.02-16c4

2. Please see the response to RAI 03.07.02-15, questions 1 through 6 for the RSW Piping Tunnel model. The response to RAI 03.07.02-15 was submitted concurrently with this response.

The STP Units 3 and 4 COLA Part 2, Tier 2 will be revised to include the following Figures 3H.6-15a thru 3H.6-15g.

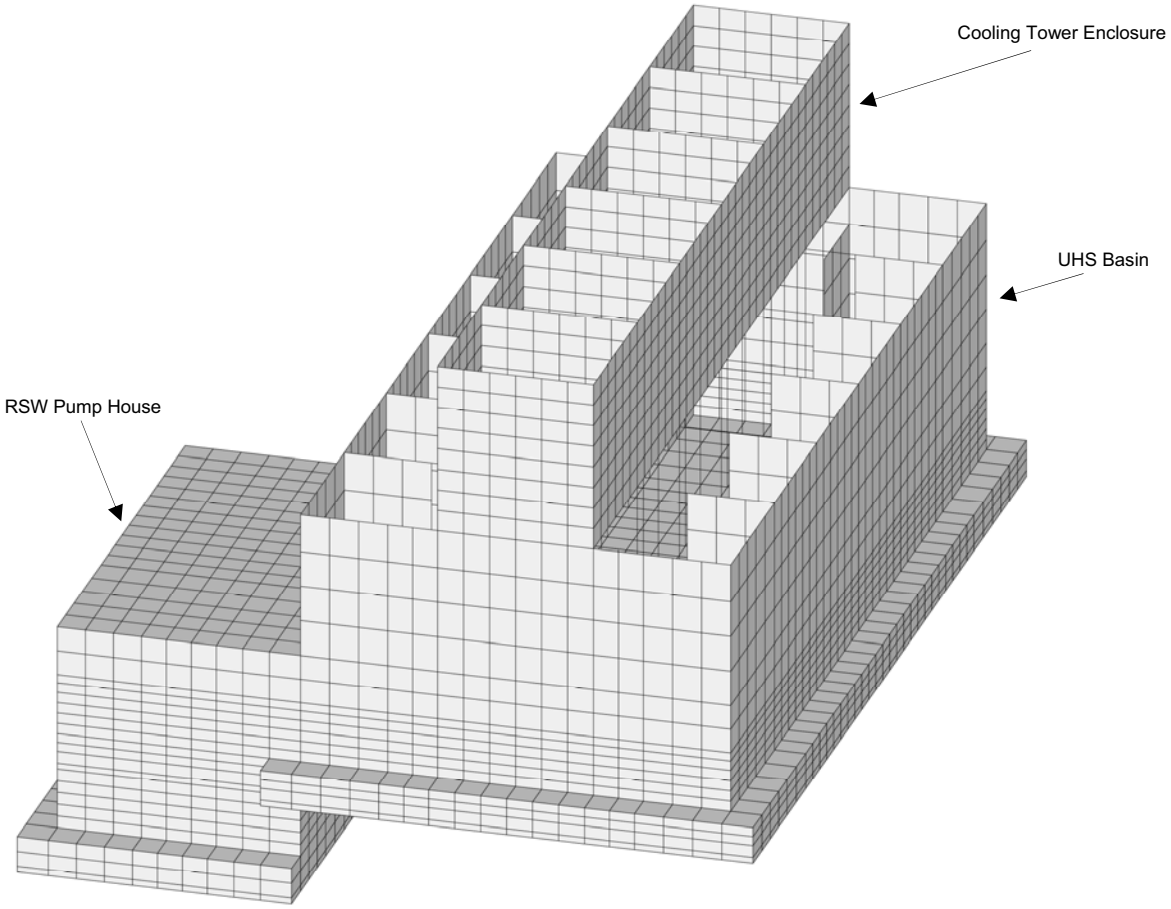
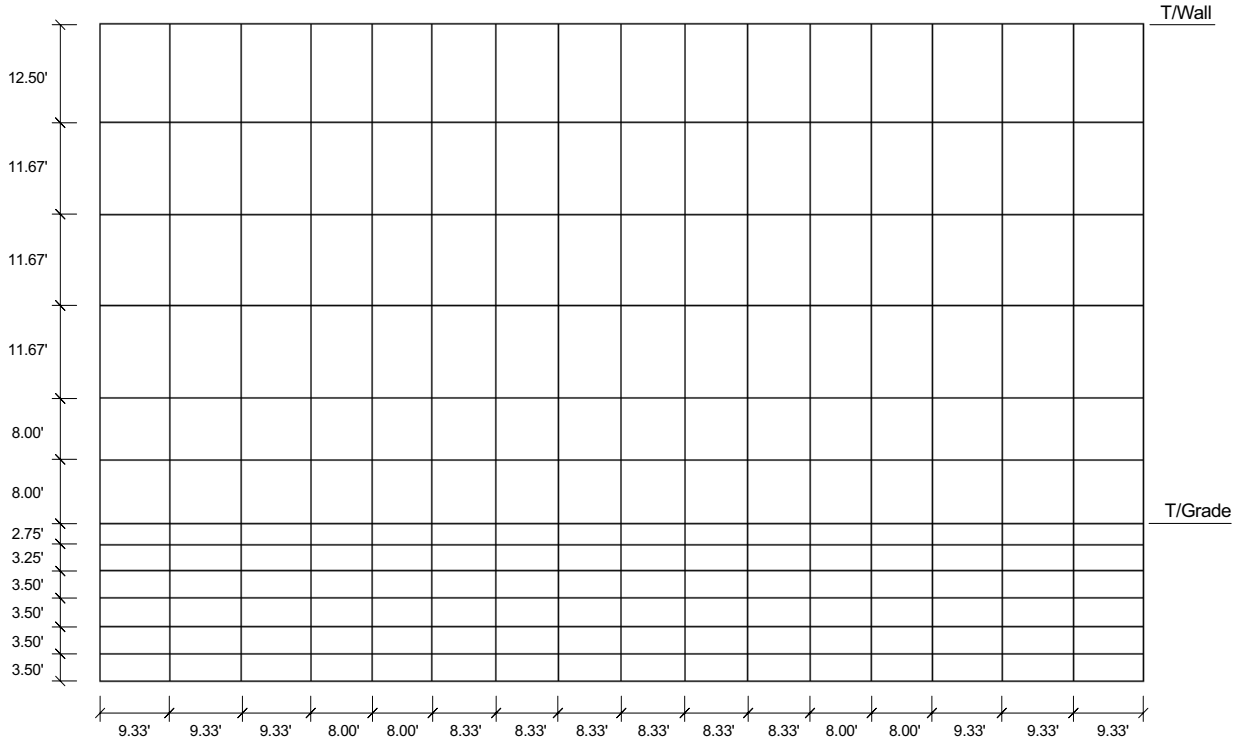
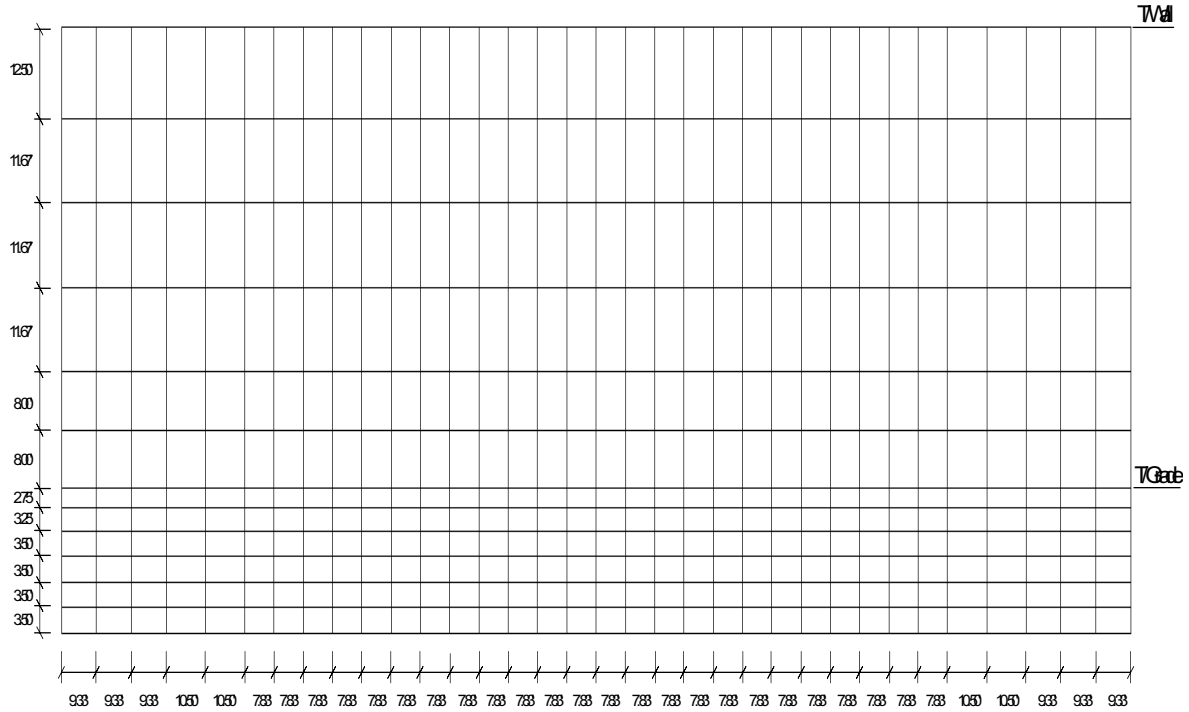


Figure 3H.6-15a: SSI Model (structure only)



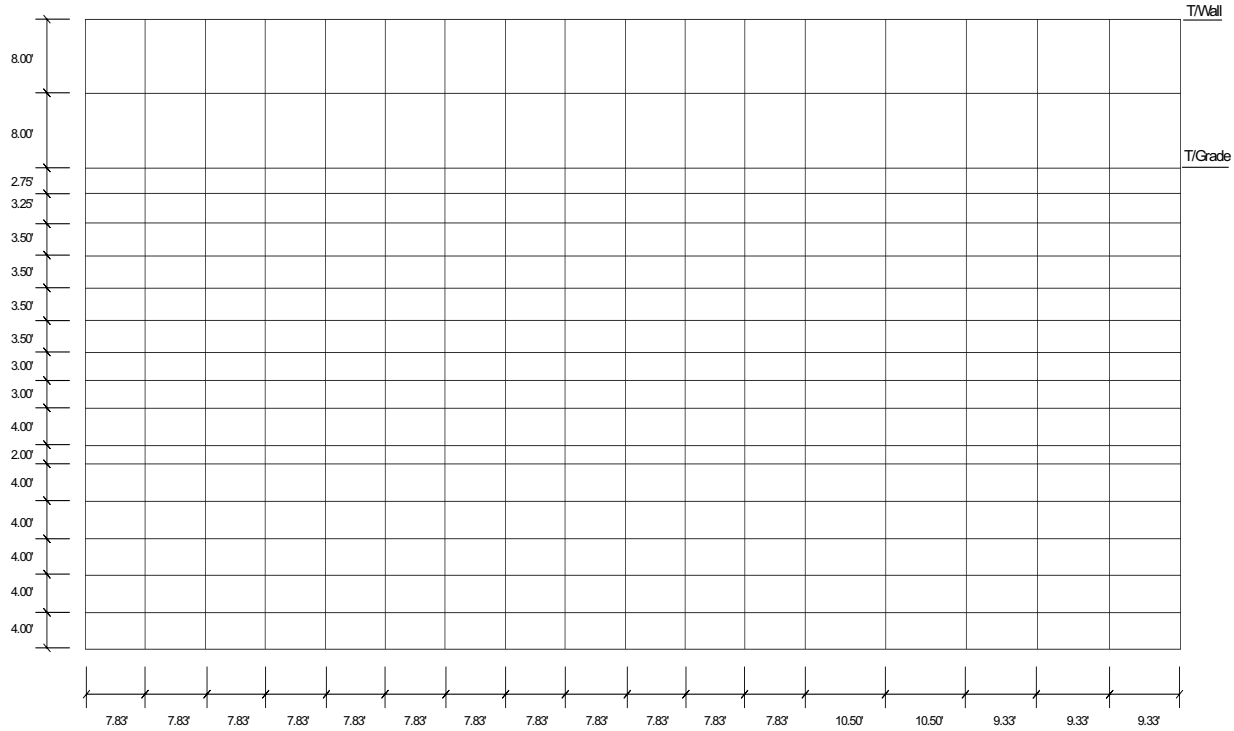
Note: Basin East and West Walls have the same mesh. The mesh is symmetrical about the vertical axis such that the view is the same whether looking at the wall from the inside or outside of the basin.

Figure 3H.6-15b: UHS Basin East and West Wall – SSI Model



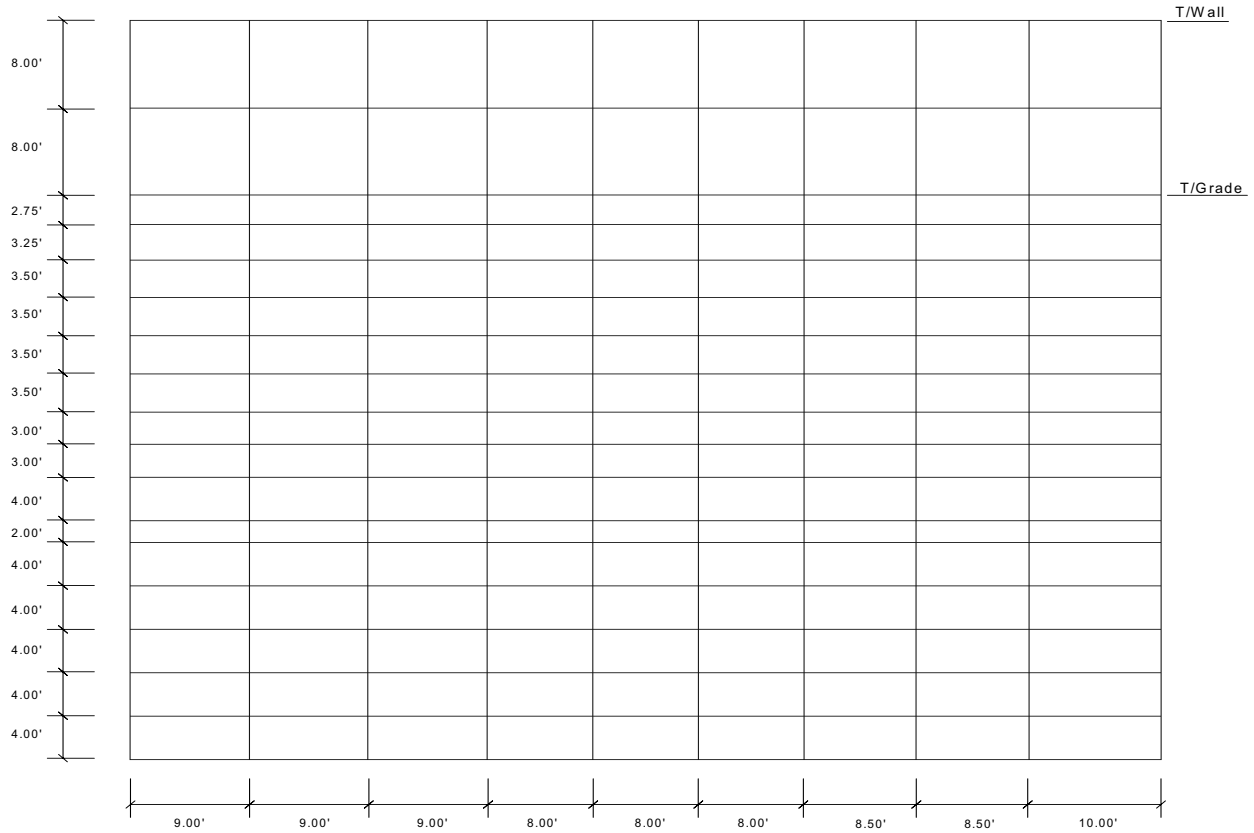
Note: Basin North and South Walls have the same mesh. The mesh is symmetrical about the vertical axis such that the view is the same whether looking at the wall from the inside or outside of the basin.

Figure 3H.6-15c: UHS Basin North and South Wall – SSI Model



Note: The view is looking south at the outside face of the RSW pump house north wall.

Figure 3H.6-15d: RSW Pump House North Wall – SSI Model



Note: The view above is looking east at the outside face of the RSW pump house west wall. The RSW pump house east wall mesh is the mirror image of the RSW pump house west wall mesh with the same dimensions.

Figure 3H.6-15e: RSW Pump House East and West Wall – SSI Model

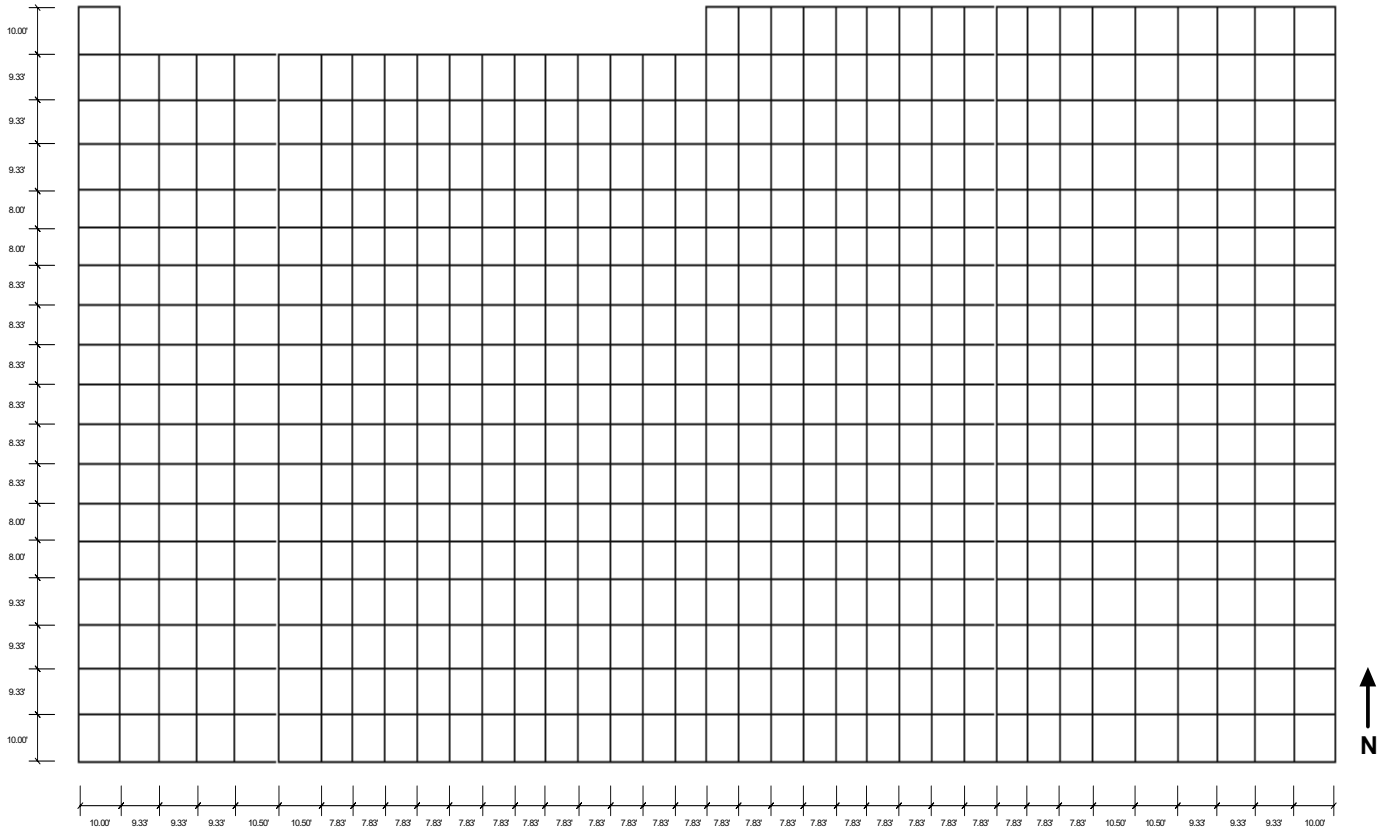


Figure 3H.6-15f: UHS Basin Basemat – SSI Model

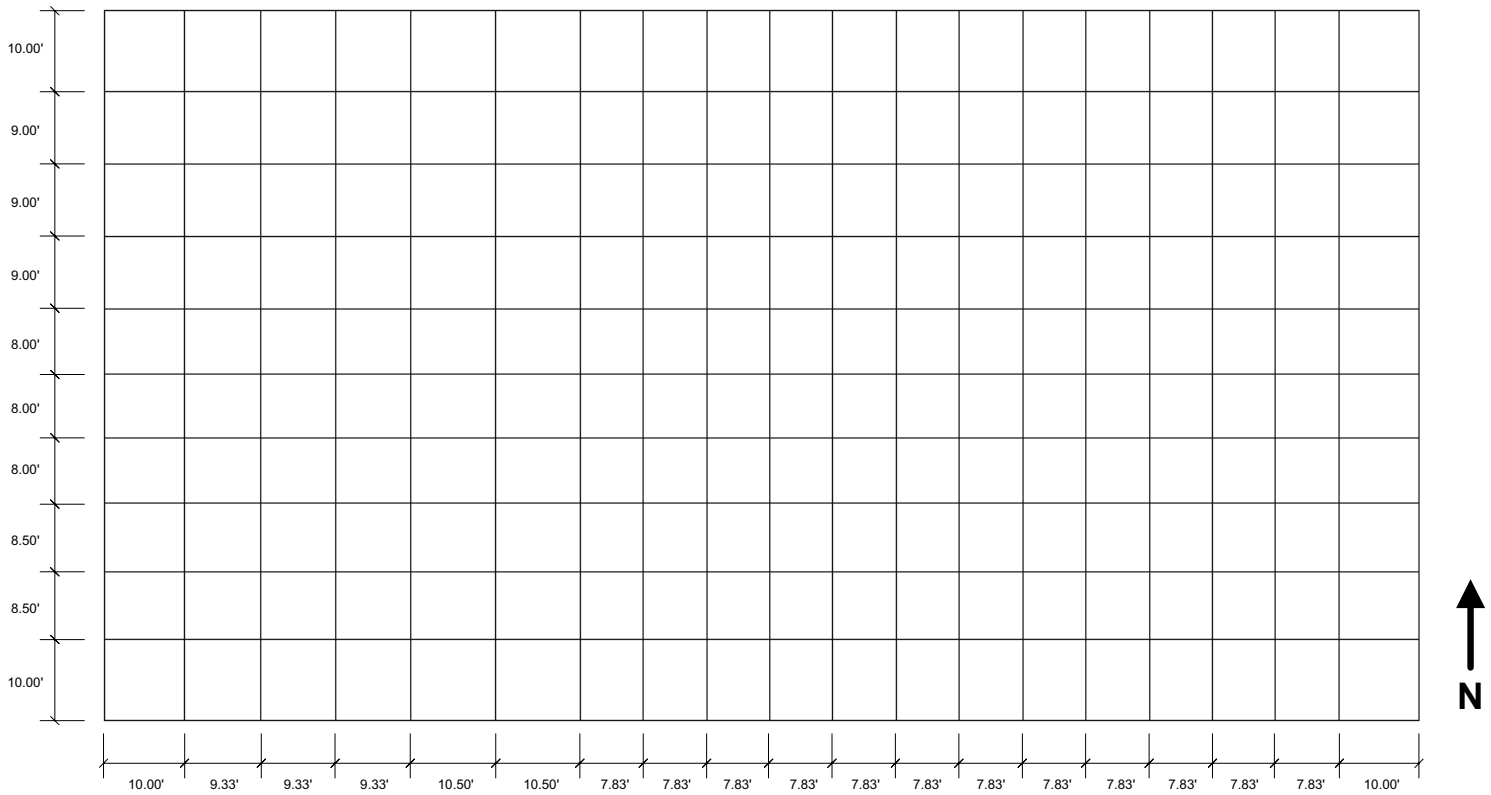


Figure 3H.6-15g: RSW Pump House Basemat – SSI Model

RAI 03.07.02-18

QUESTION:

(Follow-up Question to RAI 03.07.02-10)

1. In the second bullet of the response to RAI 03.07.02-10, the seismic loads from the input motion applied separately in three orthogonal directions should be combined following the procedure outlined in Section 2 of RG 1.92, Rev 2. According to RG 1.92 the absolute maximum values of the co-directional forces obtained from the three input motions should be sorted in decreasing values and added by applying 1.0, 0.4 and 0.4 factors to these component quantities. The applicant is requested to clarify whether the procedure used by the applicant complies with the provisions of RG 1.92, Rev 2 for combining effects caused by three spatial components of earthquake, and if not provide justification that the method used is conservative.

2. In the fifth bullet of the response to RAI 03.07.02-10, the response stated that the passive pressure in resisting the foundation sliding and overturning is not utilized. This assumption is conservative in determining the factor of safety against sliding and overturning. However, the passive soil pressure should be considered in the design of the soil-retaining walls. In addition, the magnitude and distribution of the passive soil pressure will depend on the rigidity of the wall and the amount of wall displacement and/or rotation against the soil. As such, the applicant is requested to clarify how the passive soil pressure has been calculated and considered in the wall design.

3. The applicant is requested to provide the calculated factors of safety against overturning, sliding, and floatation in the FSAR as stated in the sixth bullet of the response to RAI 03.07.02-10.

4. In the seventh and last bullet of the response to RAI 03.07.02-10, Figure 1 attached to this response shows the driving force "Es" as static and dynamic soil pressure but does not clarify the nature of the static soil pressure. The applicant is requested to clarify the nature of the driving static soil pressure in Figure 1. Also, please clarify how total at-rest soil pressure is calculated including algebraic expression. This figure with the above clarification should be included in the FSAR.

RESPONSE:

1. Application of the 100%, 40%, 40% combination rule as described in the response to RAI 03.07.02-10 (see letter U7-C-STP-NRC-090136, dated 09/15/2009), accounts for all possible combinations. The reason for exclusion of vertical downward excitation is that downward excitation will stabilize the structure and will yield higher factors of safety. The procedure outlined in Section 2 of Regulatory Guide (RG) 1.92, Revision 2 is only one of the possible combinations and it has been captured in the procedure used by STPNOC. Thus, the procedure complies with the provision of RG 1.92, Revision 2 for combining effects caused by three spatial components of earthquake.

2. As noted in the response to RAI 03.07.02-10, passive pressure is not used for resisting sliding or overturning. Full passive soil pressure can not be developed without significant structure movement (in excess of six inches). Since the Ultimate Heat Sink (UHS) structure is shown to have adequate factors of safety against sliding, overturning and floatation, there will be no significant structure movement, therefore no significant passive pressure will be mobilized. Attached Table 03.07.02-18a provides the enveloping maximum displacement relative to input motion at free field at various locations of buried walls for the Pump House (PH) and UHS basin. As can be seen the maximum displacement is only 0.22 inches. Since the movement is so small, no significant passive pressure is mobilized and therefore not used for wall design.

   The embedded walls are designed for the total lateral at rest soil pressure as shown in Figures 3H.6-41 thru 3H.6-43 provided as part of the Supplement 2 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated December 30, 2009).

3. The requested factors of safety against overturning, sliding and floatation have been provided in Table 3H.6-5 as part of the Supplement 2 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated December 30, 2009).

4. Please see Figures 3H.6-45 through 3H.6-50 provided as part of the Supplement 2 response to RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090230, dated December 30, 2009). These figures provide the details of driving and resisting lateral earth pressures used for stability evaluation of the Ultimate Heat Sink and Reactor Service Water Pump House structure. Details for computation of lateral earth pressures, including algebraic equations, are provided in COLA Section 2.5S.4.10.5.

   As requested, Figure 1 of the response to RAI 03.07.02-10 will be included in COLA Part 2, Tier 2 as new Figure 3H.6-137; see attached COLA mark-up.

Table 03.07.02-18a
Summary of Enveloping Maximum Displacement Relative to Input Motion at Free-Field Grade Level

Location	SAP Node No.	SASSI Node No.	Displacement Relative to Input Motion (in.): Envelope of all Analysis Cases		
			East-West (X)	North-South (Y)	Vertical (Z)
Bottom of PH walls					
	663	1163	0.18	0.20	0.08
	843	1527	0.21	0.20	0.10
	860	1561	0.22	0.19	0.11
	680	1197	0.18	0.19	0.07
Mid-level of PH walls					
	11920	14995	0.16	0.14	0.07
	11863	15101	0.17	0.11	0.09
	11823	15015	0.16	0.11	0.08
	11766	14851	0.16	0.11	0.05
PH roof					
	5511	16429	0.16	0.13	0.08
	5690	16608	0.16	0.12	0.10
	5707	16625	0.17	0.11	0.11
	5528	16446	0.16	0.13	0.06
	5626	16544	0.16	0.09	0.06
	5621	16539	0.16	0.11	0.06
	5632	16550	0.16	0.10	0.07
Bottom of UHS basin walls					
	3397	8546	0.15	0.14	0.11
	3989	9753	0.16	0.15	0.08
	4023	9821	0.16	0.12	0.13
	3431	8614	0.15	0.12	0.12
PH operating floor					
	3989	9753	0.16	0.15	0.08
	4188	10155	0.18	0.16	0.10
	4205	10189	0.18	0.14	0.11
	4006	9787	0.16	0.13	0.07
	4119	10015	0.17	0.15	0.08
	4124	10025	0.17	0.14	0.09
	4130	10037	0.17	0.13	0.09

COLA Section 3H.6.5.2.14 as revised in the Supplement 2 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated December 30, 2009) will be revised and new Figure 3H.6-137 will be added as shown below:

3H.6.5.2.14 Determination of Seismic Overturning Moments and Sliding Forces for Seismic Category I Structures

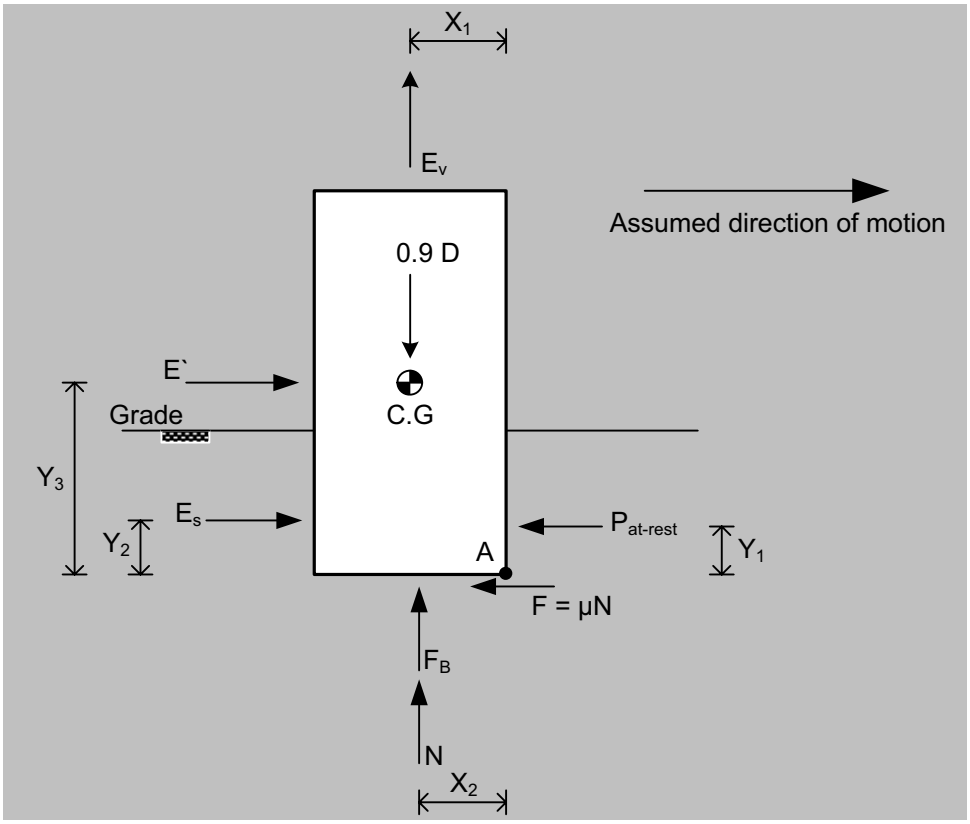
The evaluation of seismic overturning moments and sliding accounts for the simultaneous application of seismic forces in three directions using 100%, 40%, 40% combination rule as shown below:

±100% X-excitation ±40% Y-excitation +40% Z-excitation
±40% X-excitation ±100% Y-excitation +40% Z-excitation

(Note: X & Y are horizontal axes and Z is vertical axis. Positive Z is upward. Also, ±40% X-excitation ±40% Y-excitation ±100% Z-excitation is not critical.)

The resisting forces and moments due to dead load are calculated using a reduction factor of 0.90. Resisting forces and moments due to soil are based on at-rest soil pressure. The friction coefficients used for the sliding evaluation are 0.30 under the RSW Pump House and 0.40 under the UHS Basin. See Figure 3H.6-137 for formulations used for calculation of factors of safety against sliding and overturning. The calculated stability safety factors for the UHS/RSW Pump House are provided in Table 3H.6-5.

Figure 3H.6-137: Formulations Used for Calculation of Factors of Safety Against Sliding and Overturning



Factors of Safety against Sliding and Overturning about point A are calculated as follows:

$$SF_{sliding} = \frac{P_{at\_rest} + F}{E_s + E`}$$

$$SF_{OT\_A} = \frac{(P_{at\_rest})(Y_1) + (0.9D)(X_1)}{(F_B)(X_2) + (E_s)(Y_2) + (E`)(Y_3) + (E_v)(X_1)}$$

Where:

$SF_{sliding}$ = Safety factor against sliding

SF_{OT_A} = Safety factor against overturning about "A"

D = Dead load

P_{at_rest} = Total at-rest soil pressure (see Figures 3H.6-48 through 3H.6-50)

$F = \mu N$ = friction force and μ is the coefficient of friction

E_s = Static and dynamic soil pressure (see Figures 3H.6-45 through 3H.6-47)

$E`$ = Self weight excitation in the horizontal direction

E_v = Self weight excitation in the vertical direction

F_B = Buoyancy force

N = Vertical reaction $= 0.9D - F_B - E_v$

RAI 03.08.04-17

QUESTION:

Follow-up to Question 03.08.04-1 (RAI 2964)

The staff reviewed the applicant's response to Question 03.08.04-1 and needs the following additional clarification and information to complete its review:

a) In its response the applicant uses the term "at-rest seismic lateral earth pressure in non-yielding walls." In general, "at-rest" soil pressure relates to static lateral soil pressure on non-yielding walls due to the self-weight of soil including effects due to hydrostatic pressure and surcharge pressure. The dynamic soil pressure is calculated separately and added to the lateral pressure due to static loads (e.g., at-rest, hydrostatic, surcharge, etc.). Therefore, the applicant is requested to clarify the terminology of "at-rest seismic lateral earth pressure" used to describe lateral loads in the response to this RAI.

b) For the staff to conclude that the design of structures with deep foundations, such as the Reactor Building (RB) and Control Building (CB), is satisfactory for the site, the site-specific design loads are needed to compare with the design loads used for the DCD. Lateral soil pressure is one such load. Therefore, please provide the lateral soil pressures for the RB and the CB, and compare these calculated pressures with those used in the ABWR standard plant design. Please also confirm if the effects of adjacent structures are considered in computing the lateral soil pressures, and if not, provide the justification for not doing so.

RESPONSE:

a) At-rest seismic lateral earth pressure is the dynamic soil pressure for at-rest condition [i.e., the structure is not moving away from soil (active condition) or moving towards soil (passive condition)]. The total at-rest lateral earth pressure is the summation of at-rest seismic lateral earth pressure and the lateral earth pressure due to static loads such as static surcharge load, hydrostatic pressure and static soil pressure. The procedure for earth pressure calculations is described in COLA Part 2, Tier 2, Section 2.5S.4.10.5. For further clarification, please see Figures 03.08.04-17a through 03.08.04-17l provided in part (b) of this response.

b) Per Sections 3H.1.4.3.1.8 and 3H.1.5.5.3.1 of the ABWR DCD, Tier 2, the lateral soil pressures used for the design of Reactor Building (RB) walls are provided in Figure 3H.1-11 of DCD Tier 2. Also, per Section 3H.2.4.3.1.4 of DCD Tier 2, the lateral soil pressures used for the design of Control Building (CB) walls are provided in Figure 3H.2-14 of DCD Tier 2.

   The DCD Figures 3H.1-11 and 3H.2-14 provide the at-rest lateral soil pressure excluding safe shutdown earthquake (SSE) increment, H, and at-rest lateral soil pressure including SSE increment, H`. H is used in non-seismic load combinations and H` is used in seismic load combinations.

   Figures 03.08.04-17a through 03.08.04-17l on the following pages provide the following:

Figure 03.08.04-17a: DCD total at-rest lateral soil pressure excluding SSE increment, H, for design of RB walls

Figure 03.08.04-17b: DCD total at-rest lateral soil pressure including SSE increment, H`, for design of RB walls

Figure 03.08.04-17c: STP 3 & 4 total at-rest lateral soil pressure excluding SSE increment, H, for RB walls

Figure 03.08.04-17d: STP 3 & 4 total at-rest lateral soil pressure including SSE increment, H`, for RB walls

Figure 03.08.04-17e: Comparison of DCD and STP 3 & 4 lateral soil pressure H for design of RB walls

Figure 03.08.04-17f: Comparison of DCD and STP 3 & 4 lateral soil pressure H` for design of RB walls

Figure 03.08.04-17g: DCD total at-rest lateral soil pressure excluding SSE increment, H, for design of CB walls

Figure 03.08.04-17h: DCD total at-rest lateral soil pressure including SSE increment, H`, for design of CB walls

Figure 03.08.04-17i: STP 3 & 4 total at-rest lateral soil pressure excluding SSE increment, H, for CB walls

Figure 03.08.04-17j: STP 3 & 4 total at-rest lateral soil pressure including SSE increment, H`, for CB walls

Figure 03.08.04-17k: Comparison of DCD and STP 3 & 4 lateral soil pressure H for design of CB walls

Figure 03.08.04-17l: Comparison of DCD and STP 3 & 4 lateral soil pressure H` for design of CB walls

In the above figures, the DCD lateral soil pressures (Figures 03.08.04-17a, 03.08.04-17b, 03.08.04-17g and 03.08.04-17h) are based on the soil pressures shown in DCD Figures 3H.1-11 and 3H.2-14. The STP 3 & 4 lateral soil pressures are determined in accordance with COLA Part 2, Tier 2, Section 2.5S.4.10.5 considering the site-specific SSE and assuming DCD surcharge loads which include additional surcharge from adjacent structures. Actual surcharge loads are not known at this time. Final pressure calculations are prepared at the project detailed design stage, based on the actual design conditions at each structure, on a case-by-case basis. A commitment (COM 2.5S-3) exists in COLA Part 2, Tier 2, Section 2.5S.4.10.5.4 to provide the final earth pressure calculations, including actual surcharge loads, structural fill properties, and final configuration of structures, following

completion of the project detailed design in an update to the FSAR in accordance with 10CFR 50.71(e).

No COLA change is required for this response.

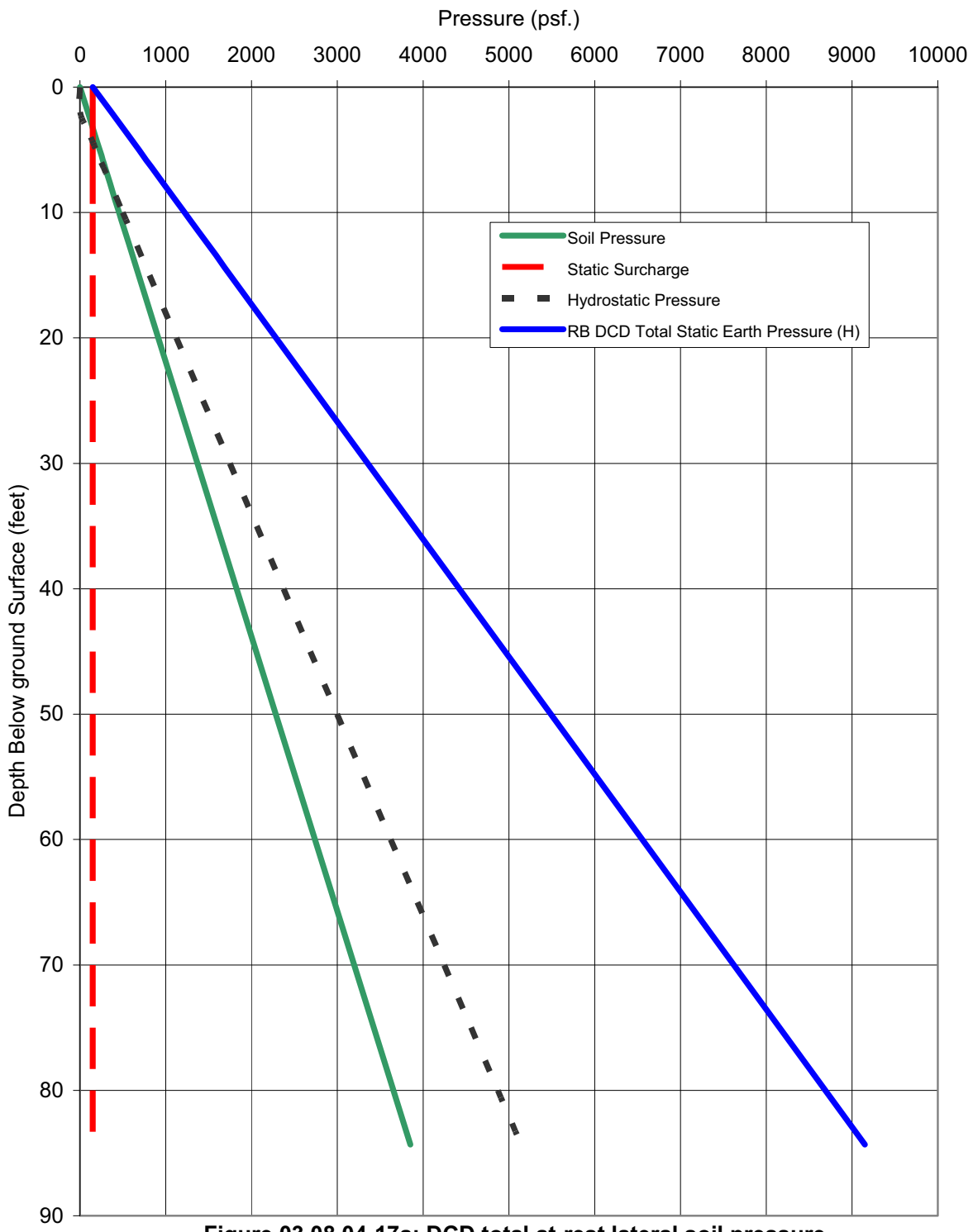


Figure 03.08.04-17a: DCD total at-rest lateral soil pressure excluding SSE increment, H, for design of RB walls

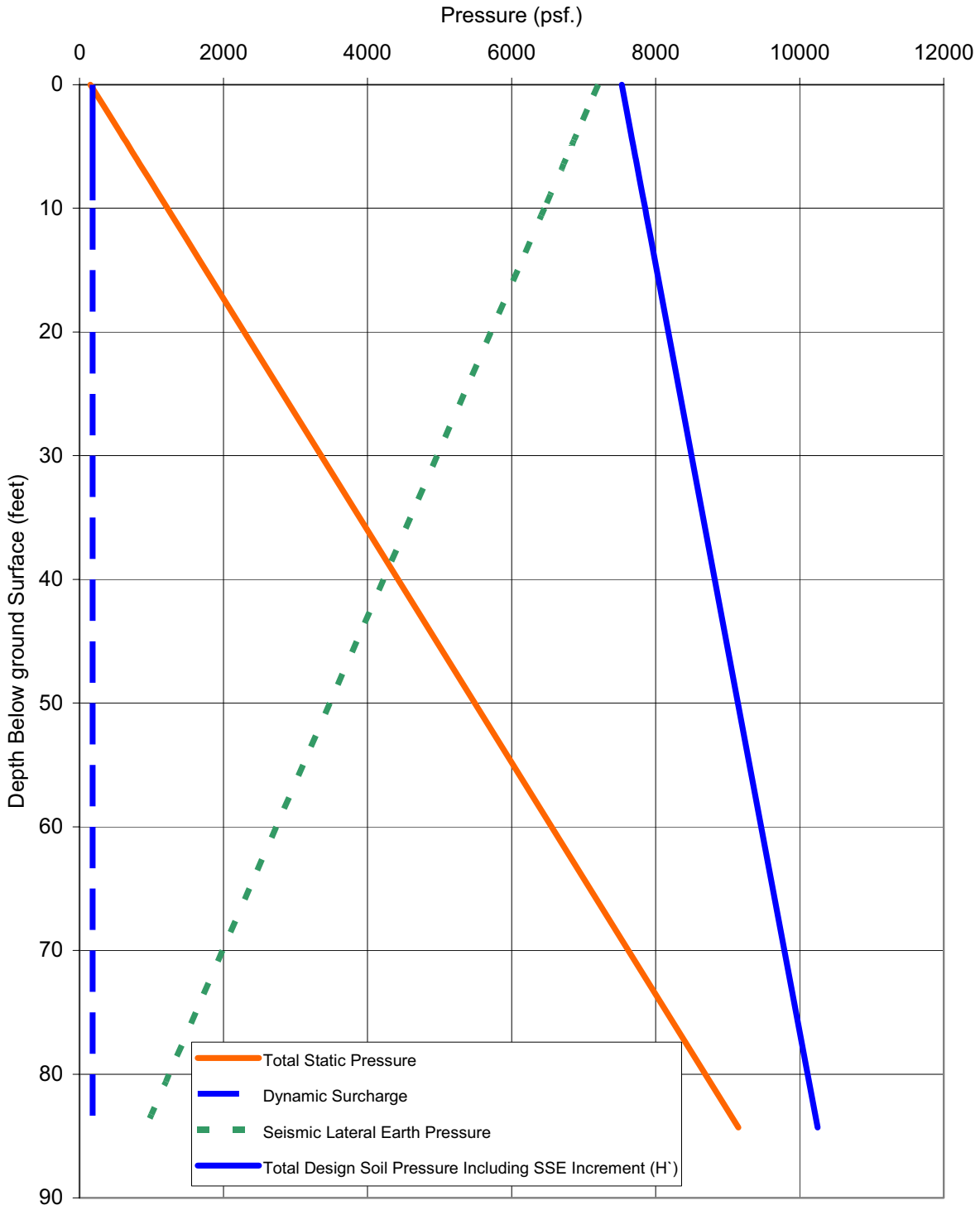


Figure 03.08.04-17b: DCD total at-rest lateral soil pressure including SSE increment, H', for design of RB walls

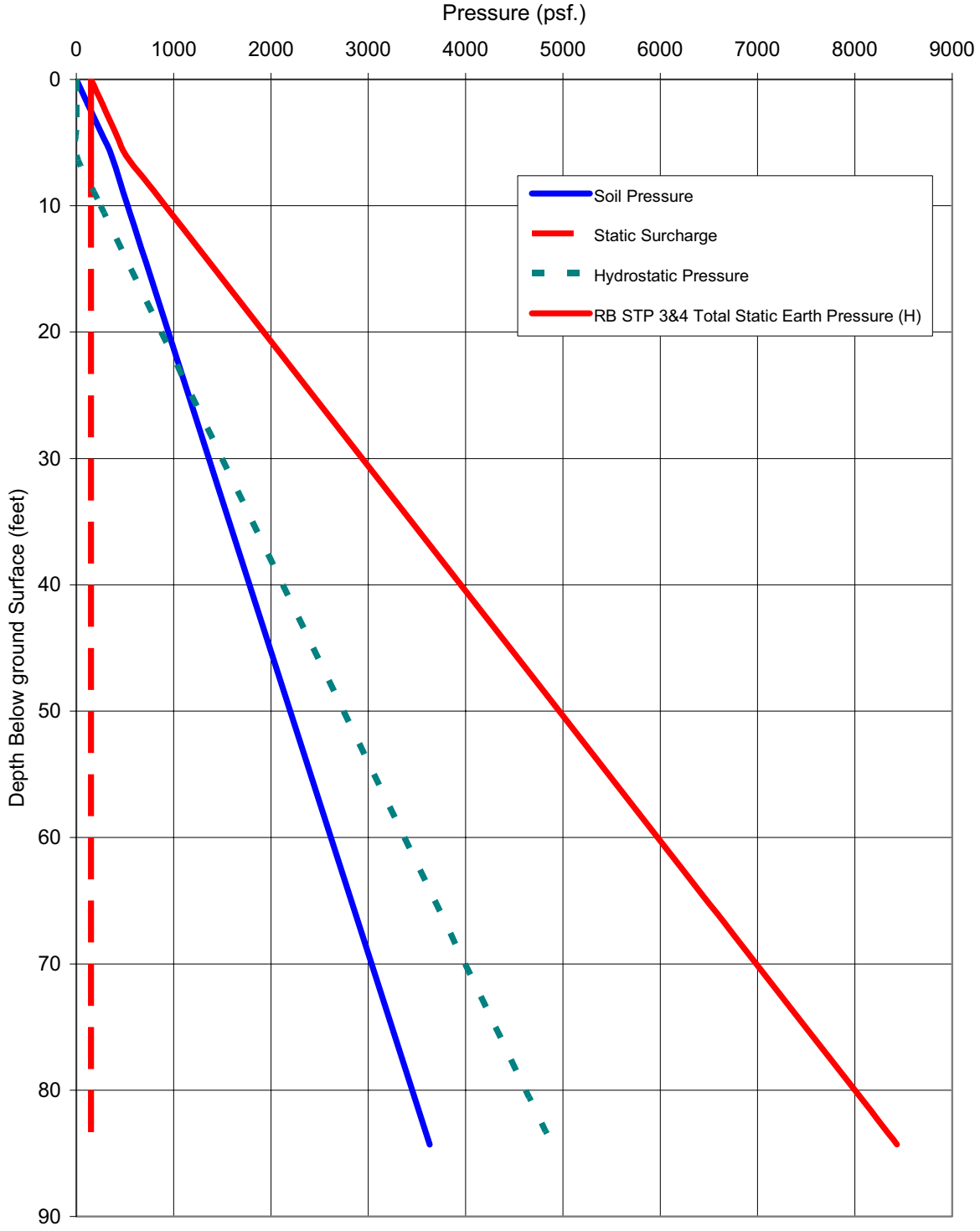


Figure 03.08.04-17c: STP 3 & 4 total at-rest lateral soil pressure excluding SSE increment, H, for RB walls

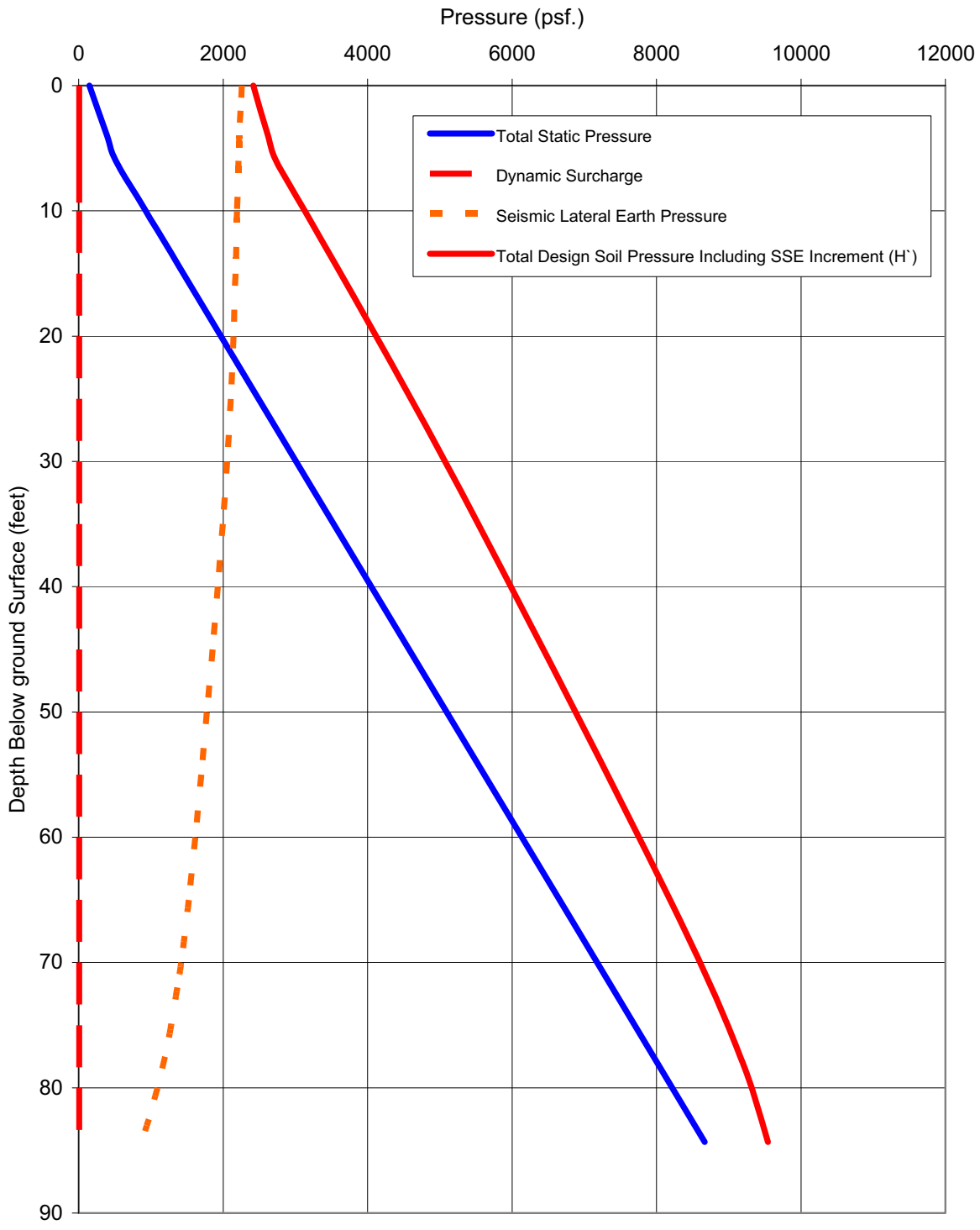


Figure 03.08.04-17d: STP 3 & 4 total at-rest lateral soil pressure including SSE increment, H', for RB walls

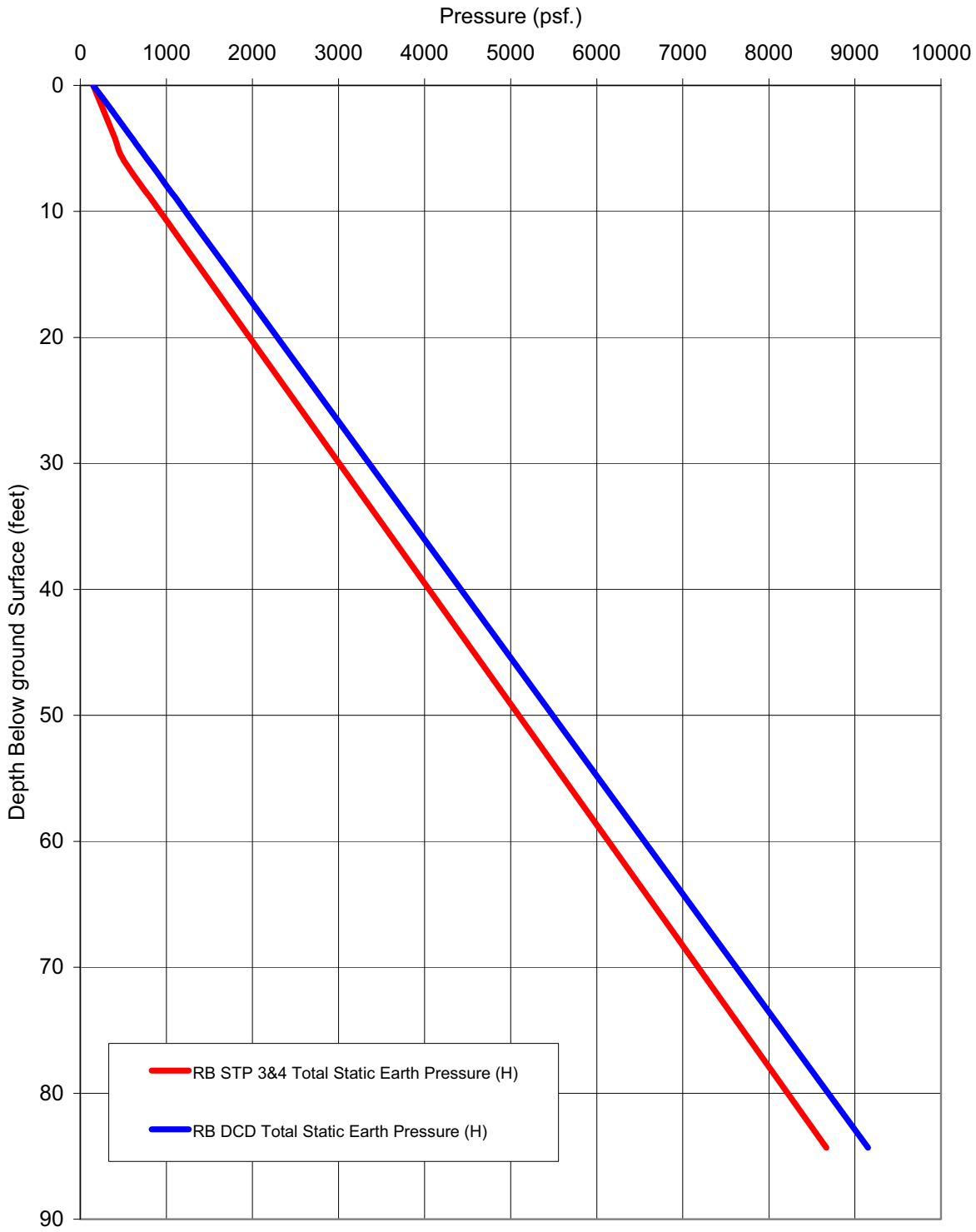


Figure 03.08.04-17e: Comparison of DCD and STP 3 & 4 lateral soil pressure H for design of RB Walls

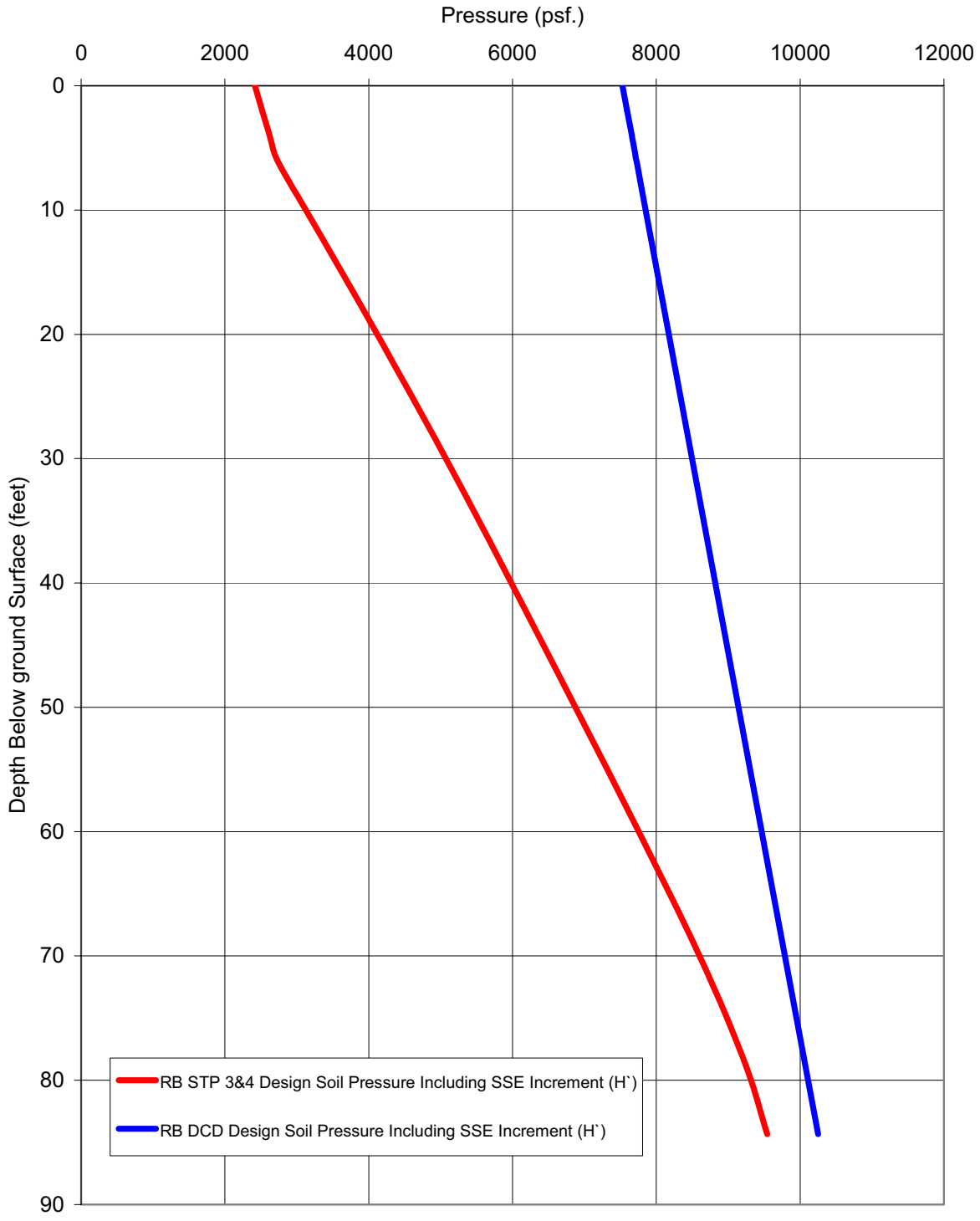


Figure 03.08.04-17f: Comparison of DCD and STP 3 & 4 lateral soil pressure H' for design of RB Walls

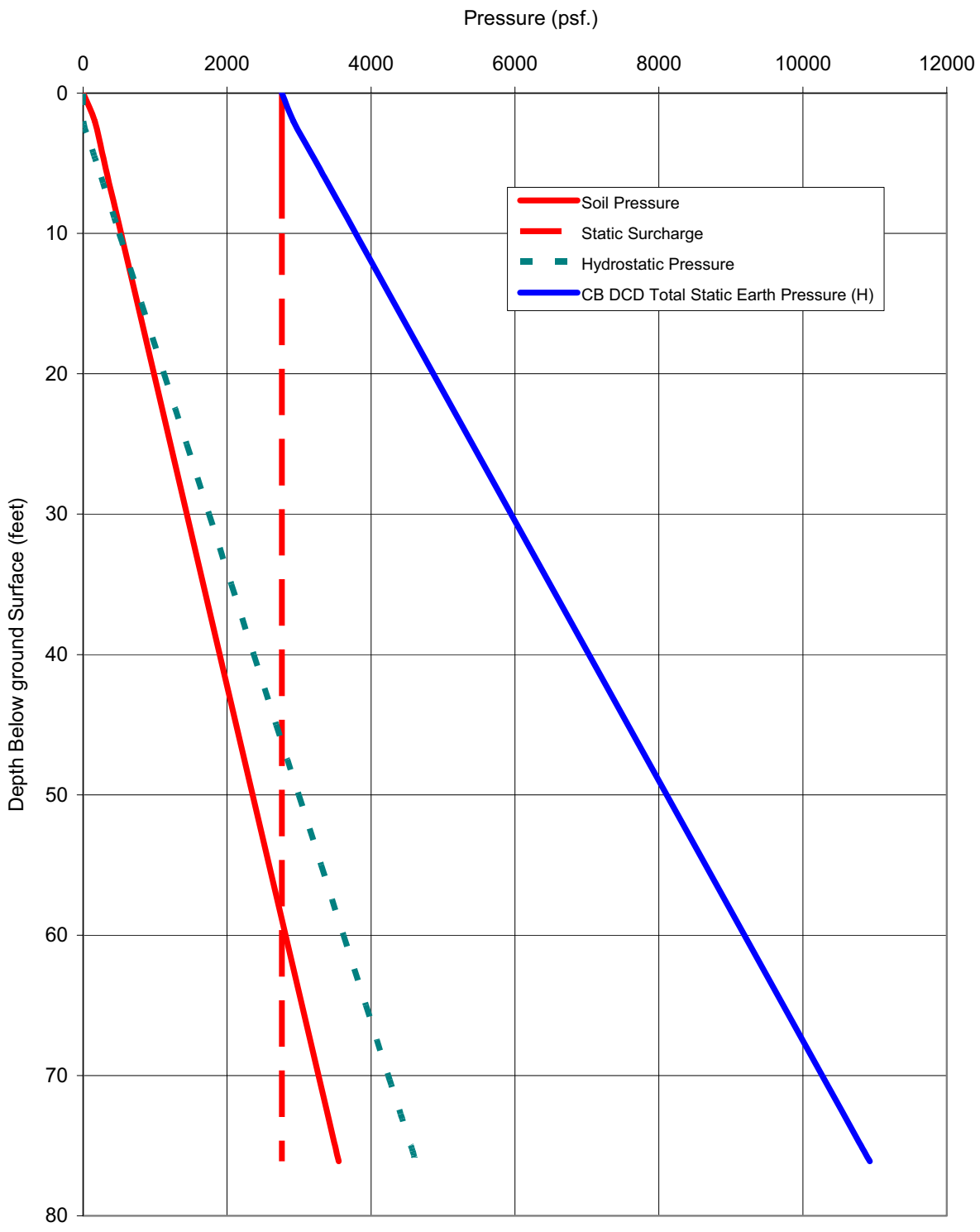


Figure 03.08.04-17g: DCD total at-rest lateral soil pressure excluding SSE increment, H, for design of CB walls

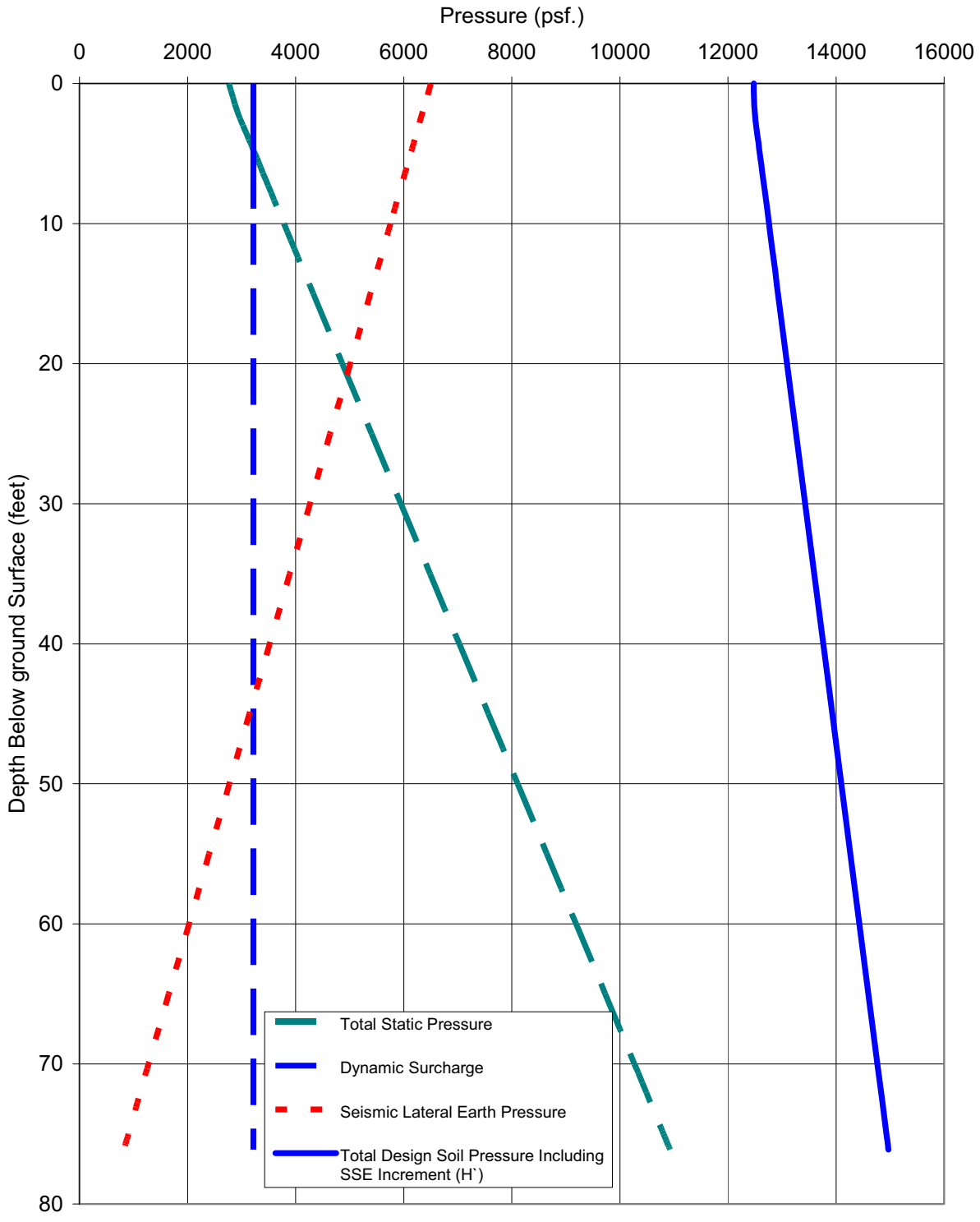


Figure 03.08.04-17h: DCD total at-rest lateral soil pressure including SSE increment, H', for design of CB walls

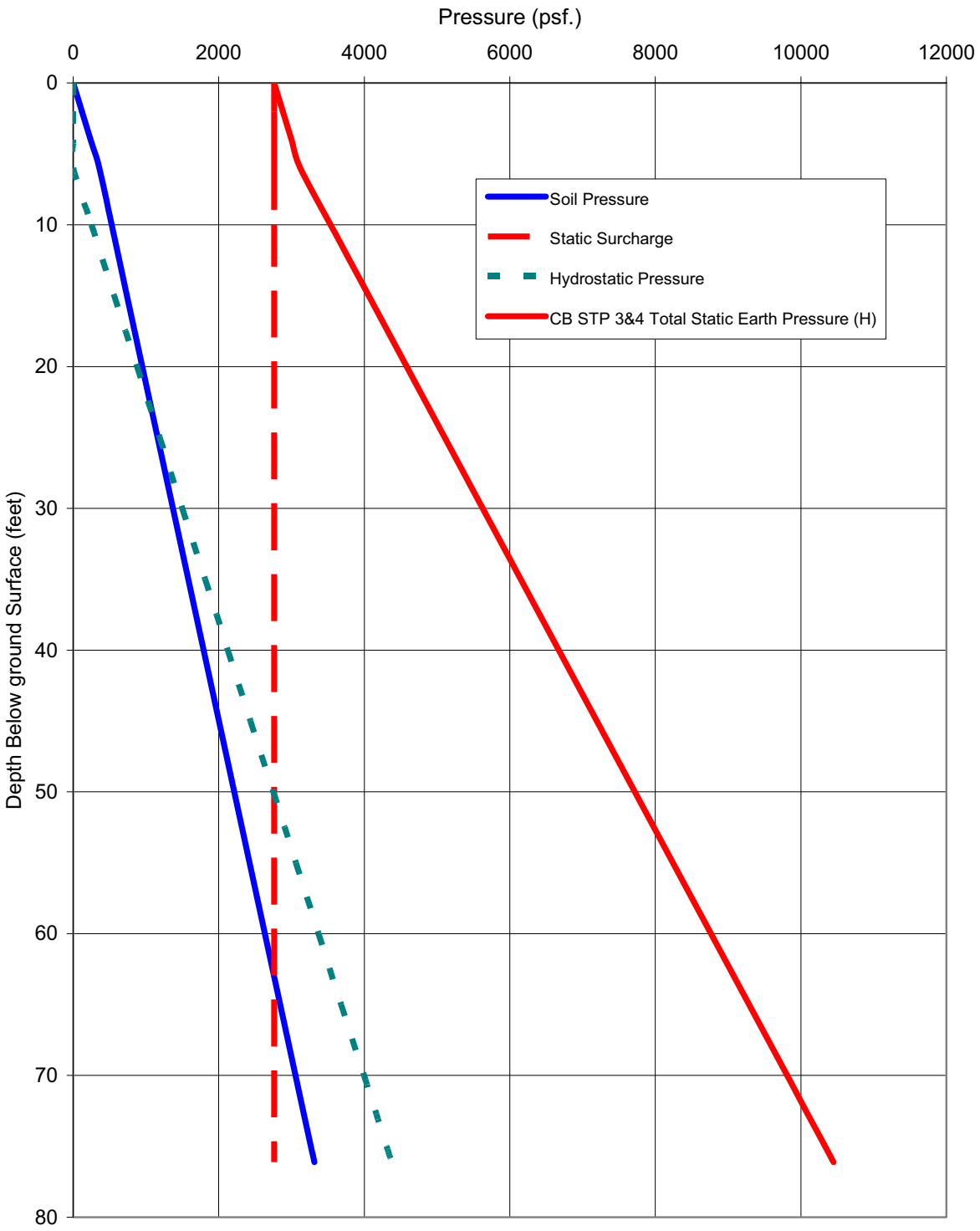


Figure 03.08.04-17i: STP 3 & 4 total at-rest lateral soil pressure excluding SSE increment, H, for CB walls

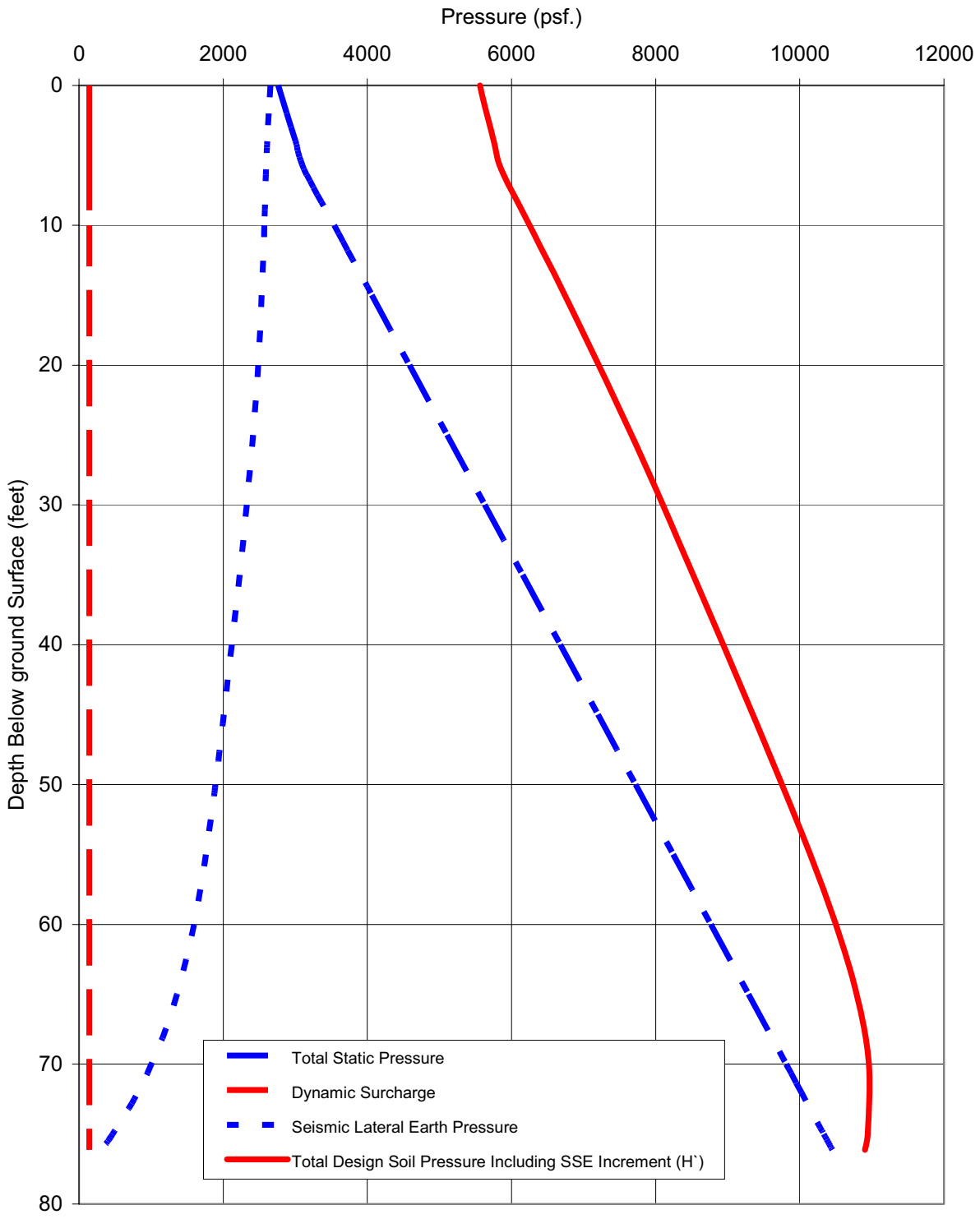


Figure 03.08.04-17j: STP 3 & 4 total at-rest lateral soil pressure including SSE increment, H', for CB walls

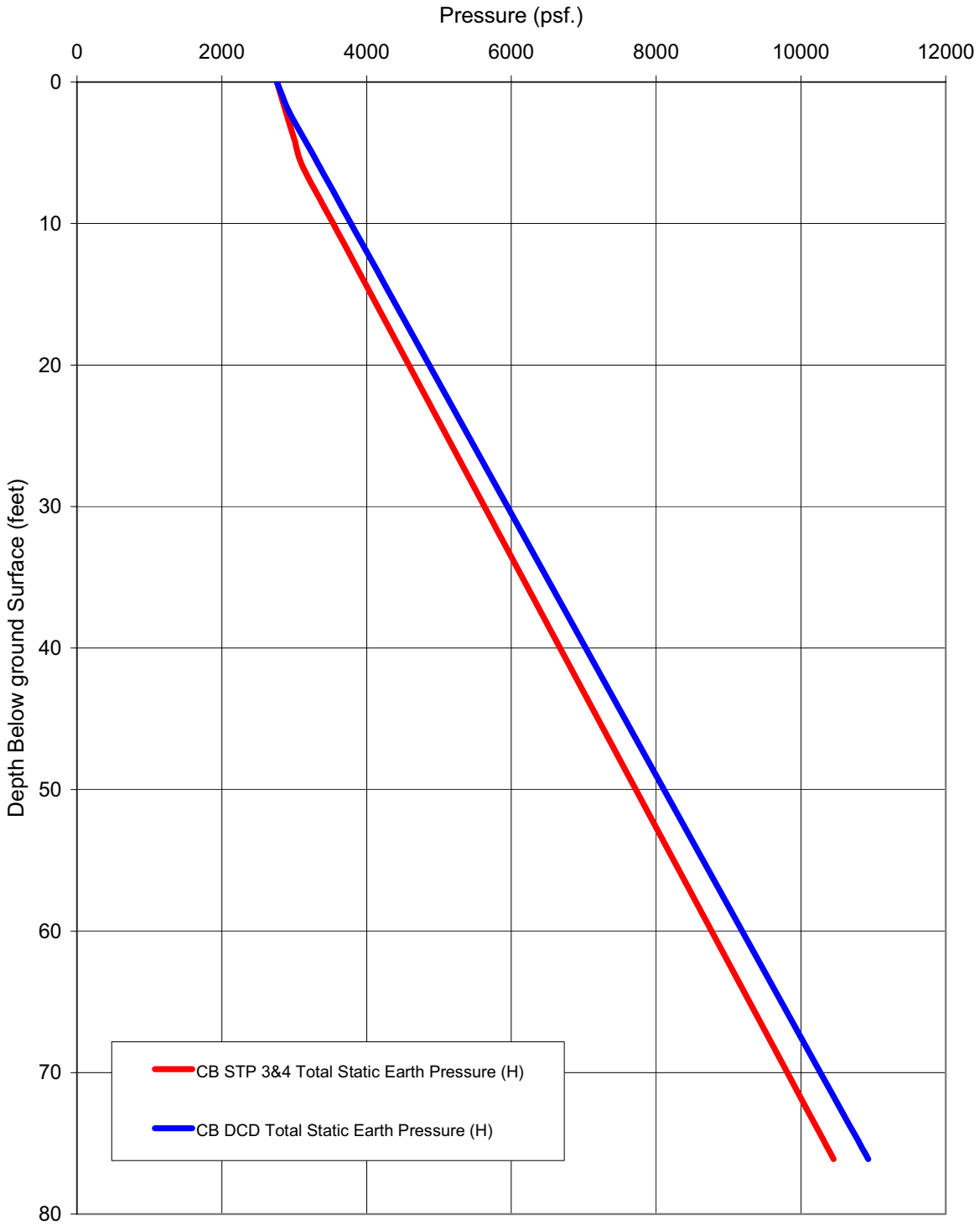


Figure 03.08.04-17k: Comparison of DCD and STP 3 & 4 lateral soil pressure H for design of CB Walls

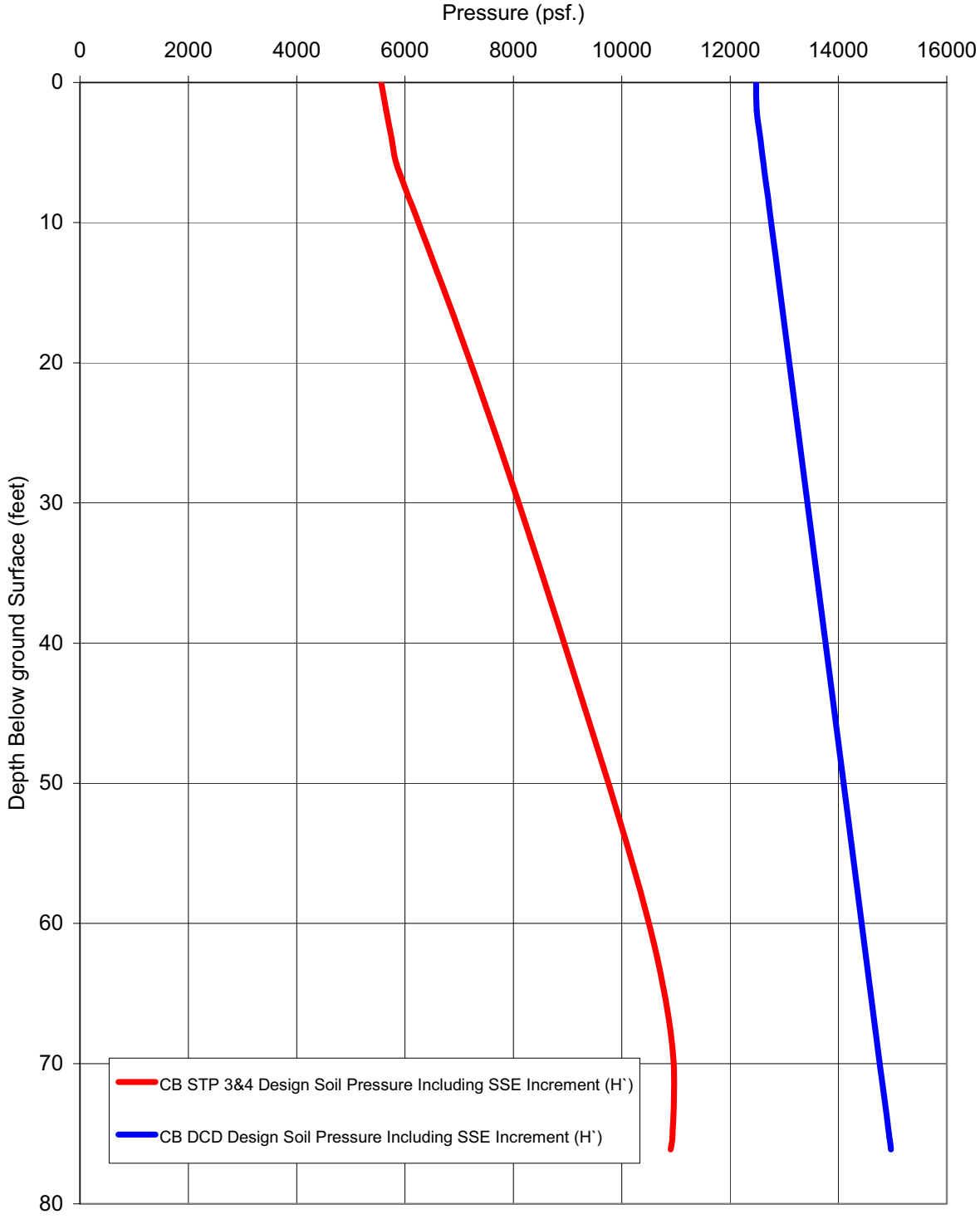


Figure 03.08.04-17l: Comparison of DCD and STP 3 & 4 lateral soil pressure H' for design of CB Walls

RAI 03.08.04-18

QUESTION:

Follow-up to Question 03.08.04-2 (RAI 2964)

The applicant's response to Question 03.08.04-2 states that the Radwaste Building (RWB) will be designed in accordance with the requirements of RG 1.143, Revision 2. The applicant also discussed the design criteria for this building for seismic category II/I evaluation. In order for the staff to conclude that the Radwaste Building design meets the requirements of RG 1.143, and also meets the requirement in ABWR DCD Section 3.7.2.8, item (3), the FSAR needs to include sufficient design information for the building to demonstrate that the design meets the pertinent design criteria. Guidance provided in SRP Section 3.8.4 may be used for providing such information. Therefore, the applicant is requested to provide design information for the RWB in the FSAR that includes more detailed description of the structure; applicable codes, standards and specifications; loads and load combinations including live loads, seismic loads, thermal loads, flood loads, tornado loads, lateral soil pressure, etc.; design and analysis procedures; structural acceptance criteria; materials and quality control; design of critical sections, stability evaluation, etc.

RESPONSE:

The Radwaste Building (RWB) for each STP unit houses the liquid and solid radwaste treatment and storage facilities, and radwaste processing and handling areas. The RWB is a reinforced concrete structure consisting of walls and slabs supported by a mat foundation. Liquid radwaste storage tanks are housed inside concrete cubicles located below grade at basement level. These cubicles are lined with steel liner plates to eliminate migration of any liquid outside the concrete cubicles.

The RWB is classified as RW-IIb (Hazardous) for STP 3 & 4 site per Section 5 of Regulatory Guide (RG) 1.143 Revision 2 and designed to meet or exceed applicable requirements of RG 1.143 Revision 2. Although, the RWB is classified as RW-IIb, it is designed conservatively for earthquake, tornado and wind loadings based on the requirements for RW-IIa classification. Design for other loads is based on the requirements for RW-IIb classification.

Due to its close proximity to Seismic Category I structures, the RWB structure is also designed to meet Seismic II/I requirements to ensure that the building does not collapse on the nearby Seismic Category I structures.

The codes and standards that are used for determining loads, load combinations, load factors and acceptance criteria meet or exceed those noted in Tables 1 through 4 of RG 1.143 Revision 2. The RWB is not subjected to any accident pressure or temperature loading. The minimum floor live load is 200 psf and the minimum roof live load is 50 psf. The seismic analysis of the RWB is performed using a fixed base stick model. The input motion of the seismic analysis is as follows:

For design basis:

- One-half of the DCD Safe Shutdown Earthquake (SSE) defined in Tier 1 Table 5.0.

For II/I design:

- The SSE input at the foundation level is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Reactor Building (RB). In this SSI analysis, five interaction nodes at the depth corresponding to the bottom elevation of the RWB foundation are added to the three dimensional SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level.

Tornado parameters are as follows:

For design basis:

- Tornado parameters are equal to three-fifths of the Region 1 tornado parameters defined in Table 1 of RG 1.76, Rev. 1. The Region 1 maximum tornado wind speed and pressure drop per Table 1 of RG 1.76, Rev. 1 are 230 mph and 1.2 psi, respectively. Three-fifths of 230 mph equals 138 mph and three-fifths of 1.2 psi equals 0.72 psi.
- Tornado missiles are in accordance with Table 2 of RG 1.143 revision 2 for RW-IIa classification.

For II/I design:

- Tornado parameters and missiles are the same as those defined in DCD Tier 1 Table 5.0.

The Seismic II/I stability evaluations of the RWB structure for sliding, overturning and floatation are in accordance with the criteria provided in response to RAI 03.07.02-13 being provided concurrently with this response. The required safety factors for floatation, sliding and overturning are the same as those specified in Standard Review Plan (SRP), Section 3.8.5.The analysis and design of the RWB is performed using a SAP2000 3D finite element model with shell and frame elements. Per Table 1 of RG 1.143 revision 2, all concrete and steel designs are in accordance with the ACI 349-97 and ANSI/AISC N690, 1984 code requirements, respectively. Also, for II/I design, the structure is conservatively designed to remain elastic.

The results of analysis and design of the RWB will be available by May 31, 2010.

More detailed and specific description of the loads, load combination etc. is provided in the COLA mark-up shown on subsequent pages.

The COLA Section 3H.3 will be revised as a result of this response as shown below.

3H.3 ~~Not Used~~ Radwaste BuildingSTD DEP T1 2.15-1

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

3H.3.1 Objective and Scope

The scope of this subsection is to document the structural design and analysis of the Radwaste Building (RWB) for STP Units 3 & 4. The RWB is a not a Seismic Category I structure. The RWB is classified as RW-IIb (Hazardous) for STP 3 & 4 site per Section 5 of Regulatory Guide (RG) 1.143 Revision 2 and designed to meet or exceed applicable requirements of RG 1.143 Revision 2. Although, the RWB is classified as RW-IIb, it is designed conservatively for earthquake, tornado and wind loadings based on the requirements for RW-IIa classification. Design for other loads is based on the requirements for RW-IIb classification

Due to its close proximity to safety-related seismic category I structures, the RWB structure is also designed to meet Seismic II/I requirements to ensure that the building does not collapse on the nearby safety-related buildings.

3H.3.2 Summary

Summary of the analysis and design results will be provided later.

3H.3.3 Structural Description

The Radwaste Building (RWB) for each STP unit houses the liquid and solid radwaste treatment and storage facilities, and radwaste processing and handling areas. The RWB is a reinforced concrete structure consisting of walls and slabs supported by a mat foundation. Liquid radwaste storage tanks are housed inside concrete cubicles located below grade at basement level. These cubicles are lined with steel liner plates to eliminate migration of any liquid outside the concrete cubicles. Metal decking supported by steel framing is used as form work to support the slabs during construction.

3H.3.4 Structural Design Criteria

3H.3.4.1 Design Codes and Standards

The RWB is designed to meet the design requirements of RG 1.143 Revision 2 and also satisfy the Seismic II/I requirements that it does not collapse on the adjacent safety related structures in the proximity of the RWB under seismic and tornado loadings. The following codes, standards, and regulatory documents are applicable for the design of the RWB.

- ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary"
- ACI 349-97, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary"
- ANSI/AISC N690, 1984 "Specifications for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities"
- AWS D1.1 "Steel Structural Welding Code", 2000
- ASCE 7-95, "Minimum Design Loads for Buildings and Other Structures"
- NRC RG 1.143, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," Rev. 2, November 2001
- NUREG-0800 SRP 3.3.2, "Tornado Loadings," Rev. 2, July 1981
- NRC RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments)," Rev 2, November 2001
- NRC RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," Rev 1, March 2007.

3H.3.4.2 Site Design Parameters

3H.3.4.2.1 Soil Parameters

- Poisson's ratio (above groundwater)……0.42
- Poisson's ratio (below groundwater)……..….0.47
- Unit Weight (moist)……..…120 pcf
- Unit Weight (saturated)……..….140 pcf
- Liquefaction potential ……...……None
- Static Soil Bearing Capacity: ……..……Calculated Factor of Safety (LATER)
- Dynamic Soil Bearing Capacity: ……Calculated Factor of Safety (LATER)

3H.3.4.2.2 Design Ground Water Level

Design groundwater level is at elevation 32 feet MSL, as shown in DCD, Tier 1, Table 5.0. This value bounds the groundwater elevations discussed in Section 2.4S.12.

3H.3.4.2.3 Design Flood Level

Design flood level is 33 feet MSL, as shown in DCD, Tier 1, Table 5.0. This flood level is above the level derived from ASCE 7-95 for the STP 3 & 4 site.

3H.3.4.2.4 Maximum Snow Load

Roof snow load is 50 psf as shown in DCD Tier 1 Table 5.0. This snow load is above the value derived from ASCE 7-95 for the STP 3 & 4 site. This load is not combined with normal roof live load.

3H.3.4.2.5 Maximum Rainfall

Design rainfall is 19.4 in/hr (50.3 cm/hr) as shown in DCD Tier 1 Table 5.0. This load is not combined with normal roof live load.

3H.3.4.3 Design Load and Load Combinations

The RWB is not subjected to any accident temperature or pressure loading.

3H.3.4.3.1 Normal Loads

Normal loads are those that are encountered during normal plant startup, operation, and shutdown.

3H.3.4.3.1.1 Dead Loads (D)

Dead loads include the weight of the structure, permanent equipment, and other permanent static loads. An additional 50 psf (2.39 kPa) uniform load is considered to account for dead loads due to piping, raceways, grating, and HVAC duct work.

3H.3.4.3.1.2 Live Loads (L)

Live loads include floor and roof area live loads, movable loads, and laydown loads. A minimum normal floor live load of 200 psf (9.6 kPa) is considered for all floors of the RWB. A normal live load of 50 psf (2.4 kPa) is considered for the roof. The floor area live load shall be omitted from areas occupied by equipment whose weight is included in the dead load.

For the computation of global seismic loads, the live load is limited to the expected live load present during normal plant operation which is defined as 25% of the normal floor and roof live loads. However, design of local elements such as beams and slabs is based on consideration of full normal live load.

3H.3.4.3.1.3 Snow Loads

The normal roof snow load is 50 psf. This load is not combined with normal roof live load.

3H.3.4.3.1.4 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

3H.3.4.3.2 Severe Environmental Load

Severe environmental loads consist of loads generated by wind and earthquake.

3H.3.4.3.2.1 Wind Load (W)

The following parameters are used in the computation of the wind loads.

- Basic wind speed (50 year recurrence interval, 3-second gust)……126 mph (203 km/h), as shown in COLA Part 2, Tier 2, Table 2.0-2 as the ABWR Standard Plant Site Parameter. This value envelops the value derived from ASCE 7-95 for STP 3 & 4 site.
- Exposure:.....D
- Importance factor:.....1.15
- Velocity pressure exposure coefficient per ASCE 7 Table 6-3, but ≥ 0.87
- Topographic factor.....1.0
- Wind directionality factor.....1.0

Wind loads are calculated in accordance with the provisions of Chapter 6 of ASCE 7-95.

3H.3.4.3.2.2 Earthquake (E_o)

The earthquake loads are those due to one-half of the Safe Shutdown Earthquake (SSE) defined in DCD Tier 1, Table 5.0. This corresponds to the Regulatory Guide 1.60 response spectra anchored to 0.15g. The earthquake loads are applied in all three orthogonal directions. The total structural response is predicted by combining the applicable maximum co-directional responses by the square root of the sum of the squares (SRSS) method.

3H.3.4.3.2.3 Flood Load (FL)

The flood level is at 33 feet MSL, as stated in Section 3H.3.4.2.3 above.

3H.3.4.3.3 Extreme Environmental Load

Extreme environmental loads consist of loads generated by tornado.

3H.3.4.3.3.1 Tornado Loads

The tornado load effects consist of wind pressure, differential pressure, and tornado generated missile loads. The tornado parameters are as follows:

- Tornado parameters are equal to three-fifths of the Region 1 tornado parameters defined in Table 1 of RG 1.76, Rev. 1. The Region 1 maximum tornado wind speed and pressure drop per Table 1 of RG 1.76, Rev. 1 are 230 mph and 1.2 psi, respectively. Three-fifths of 230 mph equals 138 mph and three-fifths of 1.2 psi equals 0.72 psi.

- Tornado missile parameters are in accordance with Table 2 of RG 1.143 Revision 2 for RW-IIa classification

3H.3.4.3.4 Load Combinations

3H.3.4.3.4.1 Notations

S = Normal allowable stress for allowable stress design method
U = Required strength for strength design method
D = Dead load
F = Load due to weight and pressure of fluid with well-defined density and controllable maximum height
FL = Hydrostatic and hydrodynamic load due to flood
L = Live load
R_o = Piping and equipment reaction under normal operating condition (excluding dead load, thermal expansion and seismic)
T_o = Normal operating thermal expansion loads from piping and equipment
T_b = Upset thermal expansion loads from piping and equipment
H = Lateral soil pressure and groundwater effects
H' = Lateral soil pressure and groundwater effects, including dynamic effects
W = Wind load
W_t = Total tornado load, including missile effects
E_o = Earthquake load

3H.3.4.3.4.2 Structural Steel Load Combinations

$S = D + L + F + H + R_o + T_o$

$1.33S = D + L + F + H + R_o + T_b$

$1.33S = D + L + F + H + R_o + T_o + W$

$1.33S = D + L + F + H' + R_o + T_o + E_o$

$1.33S = D + L + F + H + R_o + T_o + FL$

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

For the computation of global seismic loads, the live load is limited to the expected live load present during normal plant operation which is defined as 25% of the normal floor and roof live loads. However, design of local elements such as beams and slabs is based on consideration of full normal live load.

3H.3.4.3.5.3 Reinforced Concrete Load Combinations

$U = 1.4D + 1.7L + 1.4F + 1.7H + 1.7R_o + 1.7T_o$

$U = 1.4D + 1.7L + 1.4F + 1.7H + 1.7R_o + 1.7T_b$

$U = 1.4D + 1.7L + 1.4F + 1.7H + 1.7R_o + 1.7T_o + 1.7W$

$U = 1.4D + 1.7L + 1.4F + 1.9H' + 1.7R_o + 1.7T_o + 1.9E_o$

$U = D + L + F + H + R_o + T_o + FL$

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

For the computation of global seismic loads, the live load is limited to the expected live load present during normal plant operation which is defined as 25% of the normal floor and roof live loads. However, design of local elements such as beams and slabs is based on consideration of full normal live load

3H.3.4.4 Materials

Structural materials used in the design of RWB are as follows:

3H.3.4.4.1 Reinforced Concrete

Concrete conforms to the requirements of ACI 349. Its design properties are:

- Compressive strength ……..…4.0 ksi (27.6 MPa)
- Modulus of elasticity ……..……..…..3,597 ksi (24.8 GPa)
- Shear modulus ……1,537 ksi (10.6 GPa)
- Poisson's ratio……... 0.17

3H.3.4.4.2 Reinforcement

Deformed billet steel reinforcing bars are considered in the design. Reinforcement conforms to the requirements of ASTM A615. Its design properties are:

- Yield strength …….……60 ksi (414 MPa)
- Tensile strength ……...……...……..90 ksi (621 MPa)

3H.3.4.4.3 Structural Steel

High strength, low-alloy structural steel conforming to ASTM A572, Grade 50 is considered in the design for wide-flange sections. The steel design properties are:

- Yield strength ……..50 ksi (345 MPa)
- Tensile strength ……65 ksi (448 MPa)

3H.3.4.4.4 Steel Grating

Bearing bars conforming to ASTM A1011 are considered in the design. The design property is:

Yield strength ……..……..30 to 50 ksi (207 to 345 MPa)

3H.3.4.4.5 Anchor Bolts

Material for anchor bolts conforms to the requirements of ASTM F1554, Grade 36. Its design properties are:

- Yield strength …….36 ksi (248 MPa)
- Tensile strength …….58 ksi (400 MPa)

3H.3.5 Structural Design and Analysis Summary

3H.3.5.1 Seismic Analysis

The seismic analysis of the RWB is performed using a fixed base stick model. The structure is represented by a lumped-mass model consisting of structural masses lumped at selected nodes which are connected by massless elements representing the stiffness properties of the shear walls between the nodes. The building masses are lumped at elevations where the building weights are concentrated such as the floors and roof.

For modeling reinforced concrete shear wall elements, the shear walls in each particular vibration direction are identified. The stiffness of a shear wall along its length consists of a combination of its shear stiffness and its flexural stiffness, both of which are calculated individually and combined to obtain the stiffness of the wall.

The input motion of the seismic analysis is the Regulatory Guide 1.60 response spectra for 0.15g.

3H.3.5.2 Analysis and Design

The analysis and design of the RWB is performed using a SAP2000 3D finite element model with shell and frame elements. Per Table 1 of RG 1.143 revision 2, all concrete and steel designs are in accordance with the ACI 349-97 and ANSI/AISC N690, 1984 code requirements, respectively. Also, for II/I design, the structure is conservatively designed to remain elastic.

3H.3.5.3 Seismic II/I Evaluation

The seismic II/I evaluation for the RWB is performed to ensure that the RWB will not collapse on the nearby Category I structures. The structure is conservatively designed to remain elastic for this evaluation. The earthquake input used at the foundation level is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Reactor Building (RB). In this SSI analysis,

five interaction nodes at the depth corresponding to the bottom elevation of the RWB foundation are added to the three dimensional SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level.

For tornado parameters, including the missiles, the same parameters as those defined in DCD Tier 1 Table 5.0 are used. For flood, the extreme flood level of 40 ft (12.2 m) MSL with maximum hydrodynamic force of 44 psf is used, which is caused by the Main Coolant Reservoir dike breach.

The II/I stability evaluations for sliding and overturning are performed using the site-specific SSE and other site-specific parameters such as soil properties.

RAI 03.08.04-19

QUESTION:

Follow-up to Question 03.08.04-5 (RAI 2965)

The applicant's response to Question 03.08.04-5 regarding placing a chemical agent on the exposed concrete surface of the mudmat provides descriptive explanations of the waterproofing. Per the SRP 3.8.5 guidance, the applicant needs to show that the foundation can transfer the forces from the structure to soil with the proper factor of safety. Also, because a new material is being used, the applicant needs to provide additional data on testing and other relevant information to meet guidance of SRP 3.8.5. Therefore, the applicant is requested to provide the following additional information, and update FSAR as appropriate:

(1) the specific material that will be used for the waterproof membrane; sufficient data showing that the selected waterproofing will adequately protect the concrete foundations against degradation from soil/groundwater conditions at the STP Units 3 and 4 site;

(2) the final thickness of the membrane based on the physical properties of the selected material;

(3) the application procedures for all aspects of the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repairs;

(4) tests demonstrating that the waterproofing requirements and the coefficient of friction required to transfer seismic loads for STP Units 3 and 4 have been met;

(5) methods for testing that simulate field conditions to demonstrate that the minimum required coefficient of friction is achieved by the structural concrete fill-waterproof membrane structural interface; and documentation summarizing the basis for determining that the material will meet the friction factor and waterproofing requirements;

(6) site-specific sliding evaluation for the Reactor Building and the Control Building to demonstrate that the minimum coefficient of friction needed for maintaining the minimum factor of safety against sliding is available at all sliding interfaces between the structures and foundation soil; and,

(7) specification and properties of the structural concrete fill below the RB and CB foundations.

RESPONSE:

(1) The material used for the waterproof membrane will be a two-coat color-coded Methyl Methacrylate (MMA) resin, which is an elastomeric "spray-on" membrane. The physical properties have been specifically designed to cope with the rigorous requirements of below grade conditions.

(2) The final thickness of all coats of the waterproofing membrane will be a nominal 120 mils.

(3) The vendor for the waterproofing membrane materials has not been selected. The application procedures for the coating application including batch qualification, surface preparation, application techniques, film thickness, cure time, and repairs will be determined based on manufacturer recommendations and the results of the qualification testing.

(4) As discussed in the response to RAI 03.08.04-5, the coefficient of friction will be determined with a qualification program prior to procurement of the membrane material. The qualification program will be developed to demonstrate that the selected material will meet the waterproofing and friction requirements. The qualification program will include testing to demonstrate that the waterproofing requirements and the coefficient of friction required to transfer seismic loads for STP 3 & 4 have been met. Testing methods will simulate field conditions to demonstrate that the minimum required coefficient of friction is achieved by the structural concrete fill - waterproof membrane structural interface. A technical report will document the basis for determining that the material will meet the required friction factor and waterproofing requirements. An ITAAC will be added to the COLA to document that testing results comply with the required friction factor. The proposed ITAAC is provided at the end of this response.

(5) The test program will be based on the test methods contained in ASTM D1894 and ASTM D5321. The tests will be performed with the expected range of normal compressive stresses. The coefficient of friction, as defined in ASTM D5321-08, is the slope of the line relating limiting value of the shear stress that resists slippage between two materials and the normal stress across the contact surface of the two bodies. The test fixture assembly will be designed to obtain a series of shear / lateral forces and the corresponding applied normal compressive loads. Since resistance to sliding is a global building consideration and therefore based on the average coefficient of friction value of the entire foundation, the test data will be generally represented by a best fit straight line whose slope is the coefficient of friction. An ITAAC will be added to the COLA to require a test report to document the basis for determining the material will meet the required friction factor.

(6) The site-specific sliding evaluation for the Reactor Building and the Control Building will demonstrate that the minimum coefficient of friction needed for maintaining the minimum factor of safety against sliding is available at all sliding interfaces between the structures and foundation soil. This confirmatory evaluation will be completed by April 30, 2010.

(7) The structural concrete fill below the RB and CB foundations will be comprised of unreinforced normal weight concrete with a minimum compressive strength (f'_c) of 3000 psi.

COLA will be revised as shown below as a result of this response:

1. COLA Part 2, Tier2, Section 3.8.6.1 will be revised as follows:

3.8.6.1 Foundation Waterproofing

The following standard supplement addresses COL License Information Item 3.23.

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

The coefficient of friction of the waterproofing material will be determined with a qualification program prior to procurement of the membrane material. The qualification program will be developed to demonstrate that the selected material will meet the waterproofing and friction requirements. The qualification program will include testing to demonstrate that the waterproofing requirements and the coefficient of friction required to transfer seismic loads for STP 3 & 4 have been met. Testing methods will simulate field conditions to demonstrate that the minimum required coefficient of friction is achieved by the structural concrete fill - waterproof membrane structural interface. The material will meet the required friction factor.

The test program will be based on the test methods contained in ASTM D1894 and ASTM D5321. The tests will be performed with the expected range of normal compressive stresses. The coefficient of friction, as defined in ASTM D5321-08, is the slope of the line relating limiting value of the shear stress that resists slippage between two materials and the normal stress across the contact surface of the two bodies. The test fixture assembly will be designed to obtain a series of shear / lateral forces and the corresponding applied normal compressive loads. Since resistance to sliding is a global building consideration and therefore based on the average coefficient of friction value of the entire foundation, the test data will be generally represented by a best fit straight line whose slope is the coefficient of friction.

2. COLA Part 9 will be revised to add the following site-specific ITAAC.

3.0 Site-Specific ITAAC

The reference ABWR DCD Tier 1, Chapter 4.0, “Interface Requirements,” identifies significant design provisions for interface between systems within the scope of the ABWR standard design and other systems that are wholly or partially outside the scope of the ABWR standard design. The interface requirements define the attributes and performance characteristics that the out-of-scope (site-specific) portion of the plant must have in order to support the certified ABWR design.

The STP 3 & 4 site-specific systems that require ITAAC because they have a safety-related, safety-significant, or risk significant function are listed below:

- Ultimate Heat Sink (UHS)
- Offsite Power System
- Makeup Water Preparation (MWP) System
- Reactor Service Water (RSW) System
- Communication System (See Section 4.0 - Emergency Planning ITAAC)
- Site Security (See Section 5.0 - Physical Security ITAAC)
- Circulating Water (CW) System
- Backfill under Category 1 Structures
- Breathing Air (BA) System
- Waterproofing Membrane

Table 3.0-13
Waterproofing Membrane

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
The friction coefficient to resist sliding meets the required friction coefficient to prevent sliding	Type testing will be performed to determine the minimum coefficient of friction of the type of material used in the mudmat-waterproofing-mudmat interface beneath the basemats of the Category I structures	A report exists and documents that the waterproof system (mudmat-waterproofing-mudmat interface) has a coefficient of friction to support the analysis against sliding.

RAI 03.08.04-22

QUESTION:

Follow-up to Question 03.08.04-12 (RAI 2965)

The applicant's response to Question 03.08.04-12 refers to the response submitted for RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112, dated August 20, 2009). However, a review of the FSAR subsections identified in that response reveals that the response provided only a definition of these loads, and the thermal, hydrostatic and lateral soil pressure load values are not provided. Therefore, the applicant is requested to include in the FSAR the values of the thermal, hydrostatic and lateral soil pressure loads that are used in the analysis.

RESPONSE:

Thermal Loads:

The RSW piping tunnels are not subjected to any thermal loads. The thermal loads applied to the UHS/RSW Pump House finite element model are calculated as follows:

Notation:

T_c = reference concrete placement temperature
T_i = inside temperature
T_o = outside temperature
t = thickness of section (wall/slab)

Thermal gradient load = $(T_i - T_o) / t$

Thermal axial load = $[(T_i + T_o) / 2] - T_c$

Thermal gradient loads and thermal axial loads are applied to the finite element model for six (6) separate thermal conditions.

The following temperature values are applicable to all six (6) thermal conditions:
60 ^oF reference concrete placement temperature
70 ^oF soil temperature
90 ^oF pump house inside air temperature

The basin water temperatures and the outside air temperatures for the six thermal conditions are as follows:

(1) Winter – Accident Basin Water Temperature:

95 ^{o}F basin water temperature
24 ^{o}F outside air temperature

This thermal condition maximizes the winter thermal gradient across the basin walls.

(2) Winter - Minimum Basin Water Temperature:

50 ^{o}F basin water temperature
24 ^{o}F outside air temperature

This thermal condition maximizes the thermal axial contraction of the basin walls.

(3) Winter – Typical Operating Temperature:

55 ^{o}F basin water temperature
45 ^{o}F outside air temperature

This thermal condition is applicable only for basin basemat and basin walls below 71 ft maximum water level with ACI 350-01 durability factors. Per Section 9.2.7 of ACI 350-01, estimation of contraction, expansion, and temperature change should be based on realistic assessment of such effects occurring in service. Section R.9.2.7 of ACI 350-01 specifically states that the term "realistic assessment" is used to indicate the most probable values rather than the upper bound values.

(4) Summer - Accident Basin Water Temperature:

95 ^{o}F basin water temperature
90 ^{o}F outside air temperature

This thermal condition maximizes the thermal axial expansion of the basin walls.

(5) Summer – Minimum Basin Water Temperature:

60 ^{o}F basin water temperature
90 ^{o}F outside air temperature

This thermal condition maximizes the summer thermal gradient across the basin walls.

(6) Summer – Typical Operating Temperature:

95 ^{o}F basin water temperature
90 ^{o}F outside air temperature

This thermal condition is applicable only for basin basemat and basin walls below 71 ft maximum water level with ACI 350-01 durability factors. Conservatively, the summer accident temperatures are considered as the typical summer operating temperatures.

Design Basis Flood Load:

The design basis flood level is conservatively established as 40.0 ft MSL, in accordance with Subsections 2.4S.2.2 and 3H.6.4.2.3. The flood water unit weight is conservatively considered as 80 pcf to account for minor debris in the flood water. The maximum hydrodynamic force due to the design basis flood is 44 psf (see Main Cooling Reservoir (MCR) embankment breach analysis results provided in Attachment 1 of letter U7-C-STP-NRC-090012, dated February 23, 2009). The maximum pressure on the UHS/RSW Pump House due to the design basis flood is 0.524 ksf (0.48+0.044 = 0.524) at grade level (34.0 ft MSL).

Hydrostatic Loads:

This load is only applicable to the UHS/RSW Pump House. For all load combinations in the finite element model analysis of UHS/RSW Pump House, the hydrostatic load due to water inside the basin is conservatively calculated considering the maximum water height of 71 ft above the top of the UHS basin basemat. The maximum hydrostatic pressure is 4.43 ksf at the top of UHS basin basemat elevation.

Lateral Soil Pressure:

Lateral soil pressures used for design of UHS/RSW Pump House and RSW Piping Tunnels (Figures 3H.6-41 through 3H.6-44) and stability evaluations of the UHS/RSW Pump House (Figures 3H.6-45 through 3H.6-50) have been provided as part of Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230 dated 12/30/2009).

As a result of this response, COLA Part 2, Tier 2, Sections 3H.6.4.3.1.4 through 3H.6.4.3.1.6, 3H.6.4.3.3.3, 3H.6.4.3.3.4, and 3H.6.4.3.4.3 will be revised and Section 3H.6.4.3.4.4 will be added as shown below:

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.6.4.3.1.5 Thermal Loads (T_O)

~~Internal moments and forces caused by temperature distribution.~~

The RSW piping tunnels are not subjected to any thermal loads. Thermal gradient loads and thermal axial loads are applied to the UHS/RSW Pump House finite element model for six (6) separate thermal conditions.

The following temperature values are applicable to all six (6) thermal conditions:

- Reference concrete placement temperature.....60°F
- Soil temperature…….....70°F
- Pump house inside air temperature.....90°F

The basin water temperature and the outside air temperature for the six (6) thermal conditions are as follows:

(1) Winter – Accident Basin Water Temperature

- Basin water temperature.....95°F
- Outside air temperature.....24°F

(2) Winter – Minimum Basin Water Temperature

- Basin water temperature.....50°F
- Outside air temperature.....24°F

(3) Winter – Typical Operating Temperatures

- Basin water temperature.....55°F
- Outside air temperature.....45°F

This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors. Per Section 9.2.7 of ACI 350-01, estimation of contraction, expansion, and temperature change should be based on realistic assessment of such effects occurring in service. Section R.9.2.7 of ACI 350-01 specifically states that the term “realistic assessment” is used to indicate the most probable values rather than the upper bound values.

(4) Summer – Accident Basin Water Temperature

- Basin water temperature.....95°F
- Outside air temperature.....90°F

(5) Summer – Minimum Basin Water Temperature

- Basin water temperature.....60°F
- Outside air temperature.....90°F

(6) Summer – Typical Operating Temperatures

- Basin water temperature.....95°F
- Outside air temperature.....90°F

This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors. Conservatively, the summer accident temperatures are considered as the typical summer operating temperatures.

3H.6.4.3.1.6 Hydrostatic Loads (F)

~~The hydrostatic load due to the water inside the UHS basin.~~

This load is only applicable to UHS/RSW Pump House. The hydrostatic load due to water inside the UHS basin is conservatively calculated considering the maximum water height of 71 ft above the top of the UHS basin basemat. The maximum hydrostatic pressure is 4.43 ksf at the top of UHS basin basemat elevation.

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

~~This is the total lateral soil pressure, including the dynamic effect of SSE.~~

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.6.4.3.3.4 Extreme Environmental Flood (FL)

~~See Subsection 3H.6.4.2.3.~~

The design basis flood level is 40.0 ft MSL, in accordance with Subsections 2.4S.2.2 and 3H.6.4.2.3. The flood water unit weight is conservatively considered as 80 pcf to account for minor debris in the flood water. The maximum hydrodynamic force due to design basis flood is 44 psf. The maximum pressure on the UHS/RSW Pump House due to the design basis flood is 0.524 ksf at grade level (34.0 ft MSL).

3H.6.4.3.4.3 Reinforced Concrete Load Combinations

$U = 1.4D + \cancel{1.7}1.4F + 1.7L + 1.7H + 1.7R_o$

$U = 1.4D + \cancel{1.7}1.4F + 1.7L + 1.7H + 1.7W + 1.7\ R_o$

$U = D + F + L + H + Ta + E'$

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

$U = D + F + L_o + H' + T_o + R_o + E'$

$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.2T_o + 1.3R_o$

$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3W + 1.2T_o + 1.3R_o$

$$U = D + F + L + H + T_o + R_o + FL$$

$$U = D + F + L + H + T_o + R_o + S_E$$

~~For the UHS basin, the required strength defined by the above load combinations are multiplied by the following Environmental Durability Factors defined in ACI 350:~~

~~Flexural strength... 1.3~~

~~Axial tension (including hoop tension) 1.65~~

~~Excess shear strength carried by shear reinforcement... 1.3~~

3H.6.4.3.4.4 ACI 350 Reinforced Concrete Load Combinations for UHS Basin Design

ACI 350 requirements are applicable to portions of environmental engineering concrete structures where durability, liquid-tightness, or similar serviceability are considerations. Therefore, the ACI 350 requirements and load combinations listed in this section are applicable only to the UHS basemat and basin walls below the maximum water level elevation.

Per ACI 350, although fluid densities and heights are usually well known, the load factor for fluid loads should be taken as 1.7 as part of the concept of environmental durability and long-term serviceability. ACI 350 states that the required strength from ACI 350 load combinations shall be multiplied by the following environment durability factors:

Flexural strength.....1.3

Axial tension (including hoop tension)……..1.65

Excess shear strength carried by shear reinforcement.....1.3

In addition to the reinforced concrete load combinations listed in Section 3H.6.4.3.4.3, the UHS basemat and basin walls below the maximum water level elevation are also designed for the load combinations listed below with ACI 350 durability factors applied. Except durability factors need not be applied for the hydrostatic leak-tightness testing condition, which is a temporary loading where environmental durability and long term serviceability are not required. The hydrostatic leak-tightness testing load combination uses a load factor of 1.4 on the fluid load because it is not a long-term serviceability condition that requires a load factor of 1.7. Per ACI 350, durability factors need not be applied to load combinations that include earthquake loads. As stated in Section 3H.6.4.3.1.5, the design thermal loads used in ACI 350 load combinations should be based on most probable temperature values, rather than the upper bound temperature values.

$U = 1.4D + 1.7F + 1.7L + 1.7H$

$U = 1.4D + 1.7F + 1.7L + 1.7H + 1.7W$

$U = 1.4D + 1.4F + 1.7W$ (Hydrostatic leak-tightness testing)

$U = 1.4D + 1.7F + 1.4\ T_o + 1.3H$

RAI 03.08.04-23

QUESTION:

Follow-up to Question 03.08.04-13 (RAI 2965)

In its response to Question 03.08.04-13, the applicant referred to FSAR mark-up provided in response to question 03.07.01-13 for structural analysis and design information for site-specific seismic category I structures (Letter U7-C-STP-NRC-090112 dated August 20, 2009). The staff noted that the above referenced response did not include all tables and figures referenced in the FSAR mark-up, and these are stated to be provided later. In addition, the level of detail included in FSAR Section 3H.6.6.3 regarding structural design of the various elements of site-specific structures is not sufficiently descriptive, and is not similar to that included in the ABWR DCD. Therefore, the applicant is requested to include in FSAR Section 3H.6.6.3 description of the various steel and concrete elements of the site specific structures including how these elements are designed including design results.

RESPONSE:

The Supplement 2 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated 12/30/09) contains the tables and figures that provide the design summary for the structural design of Ultimate Heat Sink (UHS) basin, UHS cooling tower enclosures, Reactor Service Water (RSW) pump house, and the RSW piping tunnels. The Supplement 2 provided the following:

- Table 3H.6-5: Factors of safety against sliding, overturning and flotation for UHS/RSW Pump House
- Table 3H.6-6: Results of RSW Piping Tunnel Design
- Table 3H.6-7: Results of UHS/RSW Pump House Concrete Wall Design
- Table 3H.6-8: Results of UHS/RSW Pump House Concrete Slab Design
- Table 3H.6-9: Results of UHS/RSW Pump House Beams and Column Design
- Table 3H.6-10: Tornado Missile Impact Evaluation for UHS/RSW Pump House

- Figures 3H.6-41 through 3H.6-43: At-rest lateral soil pressure diagrams for design of UHS/RSW Pump House
- Figure 3H.6- 44: At-rest lateral soil pressure diagram for design of RSW Piping Tunnels
- Figures 3H.6-45 through 3H.6-50: Lateral soil pressure diagrams used for stability evaluation of UHS/RSW Pump House
- Figures 3H.6-51 through 3H.6-136: Definition of reinforcement zones for UHS/RSW Pump House walls and Slabs

A three dimensional Finite Element Analysis (FEA) as shown in Figure 3H.6-40, provided with Supplement 1 Response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090208 dated 11/19/09), is used for structural analysis and design of the UHS/RSW Pump House.

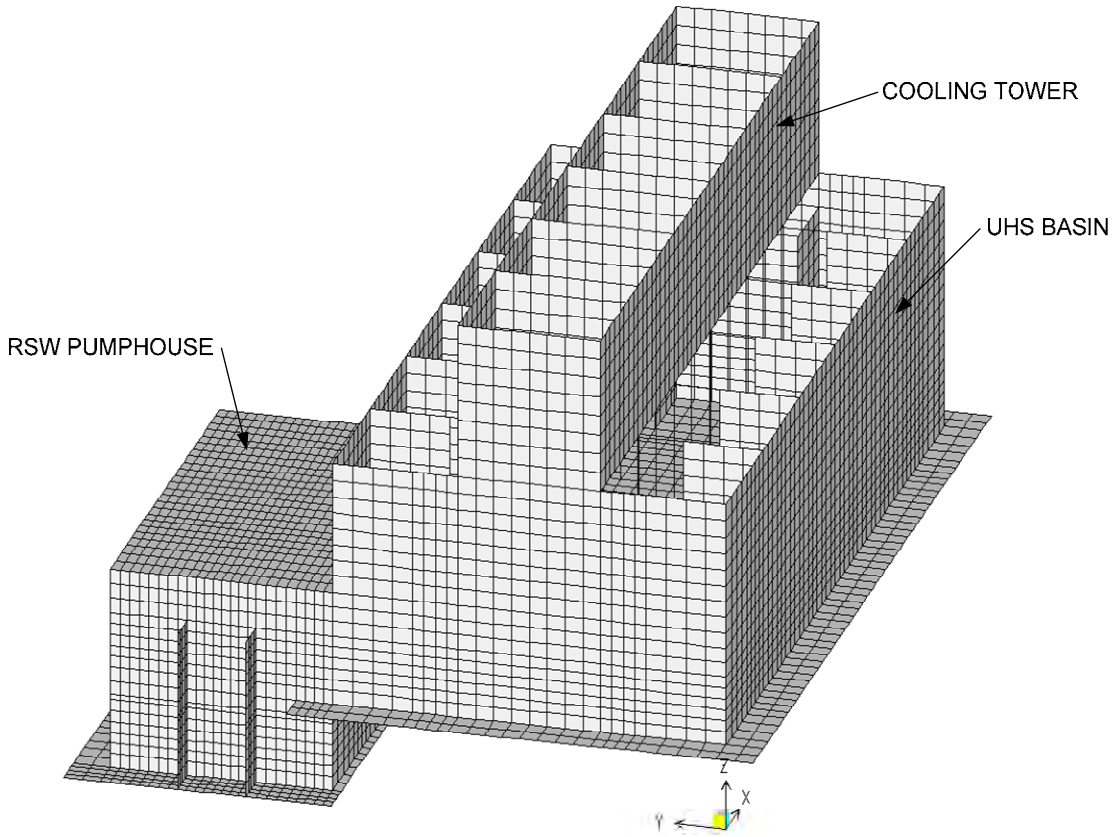


Figure 3H.6-40: SAP Finite Element Model for UHS and RSW Pump House Design

The forces in the structure caused by differential settlements due to the flexibility of the basin and pump house supporting soil are accounted for through the use of foundation soil springs in the FEA model. The methodology for computing the soil springs is presented next, followed by the values obtained.

<u>Soil Springs – Static Loading</u>

The calculated settlements due to the loading of the individual structures (S_{ss} in COLA Part 2, Tier 2, Table 2.5S.4-42) are the relevant quantity for calculating the soil spring under static loading. The unit static spring in units of force / length^3 is determined using the following equation from Section 10.5 of Bowles 1996, COLA Part 2, Tier 2, Reference 2.5S4-55 (Reference 1):

$k = q_c / S_{ss}$ force / length^3

q_c = applied foundation stress
S_{ss} = settlement of structure only due to q_c
k = modulus of subgrade reaction

As described by Bowles 1996, the modulus of subgrade reaction is an average of several local values within the foundation area. Table 2.5S.4-42 provides settlement (s_{ss}) values at nine locations in various building foundations. The local modulus of subgrade reaction (k) value at each of these 9 locations was computed and the average k for the foundation was computed as the average of these 9 local values.

Soil Springs – Seismic Loading

Reference 2 (Gazetas, 1991) provides algebraic formulas for computing the spring constants of foundations supported on/in a homogeneous half-space. These foundations have a rigid basemat of any realistic solid geometric shape. The embedded foundations are prismatic, having a sidewall-soil contact surface of height *d,* which may be a fraction of the total embedment depth *D.*

The algebraic equations of Reference 2 (Gazetas, 1991) were used to compute the soil springs for seismic loading. The algebraic equations to calculate the spring constant of a foundation require a single value of soil modulus (and Poisson's ratio) as input. The soil at the STP 3 & 4 consists of multiple layers, each with a shear modulus specific to the layer. Therefore, use of the equations for a homogeneous half-space requires finding a way to determine an appropriate value of shear modulus that accounts for the presence of the multiple soil layers. Reference 3 (Christiano, et. al., 1974) presents a method for obtaining the equivalent stiffness coefficients for a foundation resting on a layered system such as at the STP 3 & 4. The equations in Reference 3 (Christiano, et. al., 1974) can be used to calculate the appropriate single value for the soil modulus that represents the contribution of the soil layers within the influence zone of the foundation. The method involves weighting the contribution of each layer in proportion to its elastic modulus and its depth below the foundation. This weighting is done using the concept of strain energy occurring in each layer. The foundation area is represented as an equivalent circular shape and the cumulative strain energy is plotted against a dimensionless depth ratio: depth/radius. The strain energies are plotted in Reference 3 (Christiano, et. al. 1974) for vertical, horizontal, rocking and twisting displacements of the foundation. Only the vertical and horizontal modes are considered herein. The cumulative strain energy plot for the vertical mode shows an influence zone depth of 10 times the radius of the equivalent circular area, or 5 times the diameter (width). The plot for horizontal mode shows an influence zone depth of 5 times the radius of the circular area, or 2.5 times the diameter (width). Thus layers even at a considerable depth contribute to the foundation stiffness.

The SSE strain-compatible shear wave velocity is used to determine the low range, best estimate and upper range of soil shear modulus, *G,* of the individual layers via the following equation:

$$G = \rho \cdot V_s$$

Where:

G = shear modulus of individual layers
V_s = shear wave velocity (S-Wave Vel.);
ρ = mass density = γ/g;
γ = unit weight; and
g = gravitational acceleration constant (32.2 ft/s^2).

From the above information, Young's modulus of elasticity, *E*, may be calculated by:

$$E = 2G \cdot (1 + \nu)$$

Where:

E = Young's Modulus of Elasticity
ν = Poisson's Ratio

Equivalent Shear Modulus of Soil-Vertical Mode

Reference 3 (Christiano, et. al, 1974) is used to compute the equivalent modulus of the layered soil under the foundation. In this procedure, an appropriate average of the shear modulus is developed whereby each layer is weighted in accordance with the strain energy in that layer. Christiano, et. al. calculate the vertical spring using Equation 8 in the Reference 3 and their chart reproduced herein as Figure 03.08.04-23a:

$$k_v = \left[\sum \frac{(1 - \nu_i)^2}{8 \cdot a \cdot \mu_i} \cdot \Delta U_i \right]^{-1} \qquad \text{(Christiano, et. al., Equation 8)}$$

Where:

k_v = the vertical stiffness of the rigid foundation;
a = the radius of the equivalent circular area of the foundation;
ν_i = Poisson's ratio of the ith layer;
μ_i = the shear modulus of the ith layer (same as G);
ΔU_i = the strain energy coefficient change over the thickness of the ith layer (difference in U values between the top and bottom of the layer as determined from Figure 03.08.04-23a).

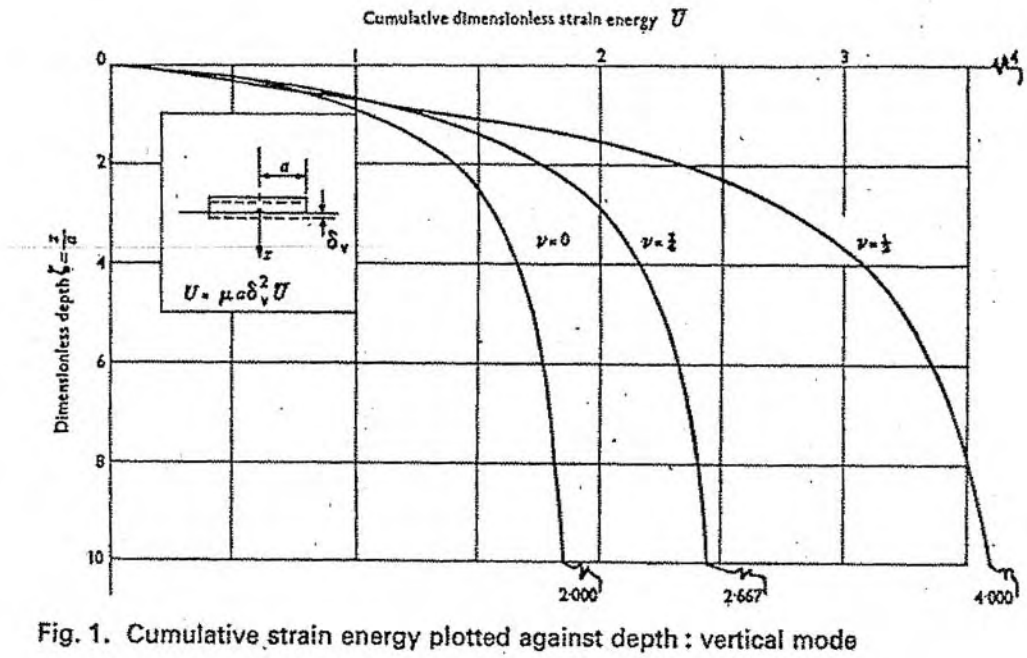


Figure 03.08.04-23a – Cumulative Strain Energy versus Depth, Vertical Mode (Reference 3 (Christiano, et al., 1974))

For Poisson's ratios of layers intermediate between those in Figure 03.08.04-23a, linear interpolation is used.

The Poisson's ratio values (ν) for individual layers were computed from the strain-adjusted wave velocities using the following equation:

$$\nu = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)}$$

The average Poisson's ratio, ν_{avg}, is computed as a layer-weighted value according to the following equation:

$$\nu_{avg} = \frac{\sum(\nu_i)\cdot \Delta U_i}{\sum \Delta U_i}$$

The average shear modulus (μ_{avg}) for vertical loading is back-calculated from a rearrangement of the equation in Reference 3 (Christiano, et. al. 1974) for the half space.

$$\mu_{avg} = k_v \cdot \frac{(1-\nu_{avg})}{4\cdot a}$$

Equivalent Shear Modulus of Soil-Horizontal Mode

Christiano, et al. calculate the horizontal spring using their Equation 9 in Reference 3 and their chart reproduced herein as Figure 03.08.04-23b:

$$k_h = \left[\sum \frac{(2-\nu_i)^2}{32 \cdot a \cdot \mu_i} \cdot \Delta U_i \right]^{-1} \quad \text{(Christiano, et al., Equation 9)}$$

Where:

k_h = the horizontal stiffness of the rigid foundation;
a = the radius of the equivalent circular area of the foundation;
ν_i = Poisson's ratio of the ith layer;
μ_i= the shear modulus of the ith layer (same as G);
ΔU_i = the strain energy coefficient change over the thickness of the ith layer (difference in U values between the top and bottom of the layer as determined from Figure 03.08.04-23b).

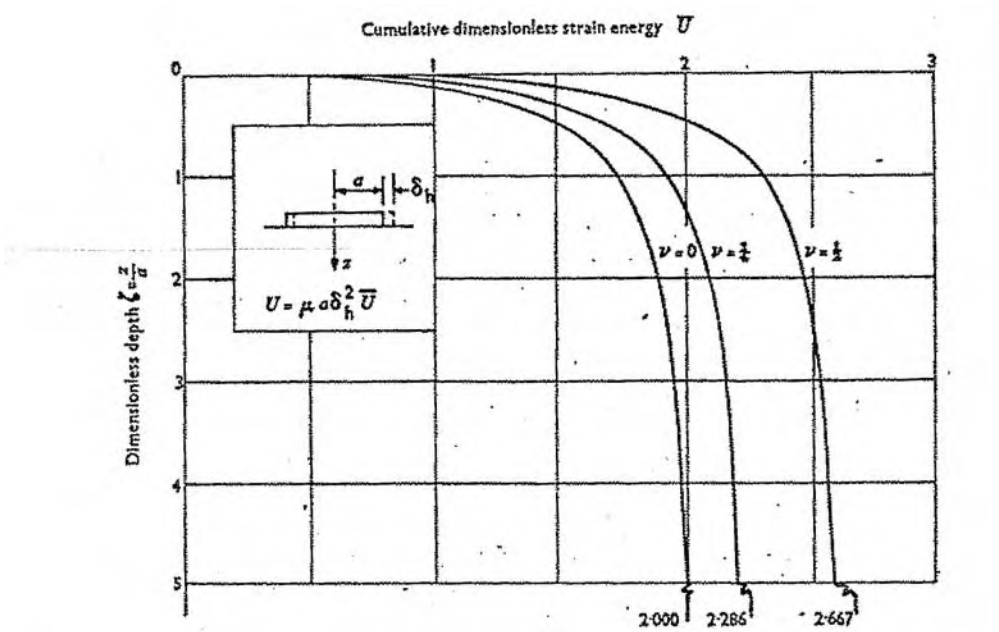


Fig. 2. Cumulative strain energy plotted against depth : horizontal mode

Figure 03.08.04-23b – Cumulative Strain Energy versus Depth, Horizontal Mode (Reference 3 (Christiano, et al., 1974))

The average shear modulus (μ_{avg}) for horizontal loading is back-calculated from a rearrangement of the equation in Reference 3 (Christiano, et al., 1974) for the half space.

$$\mu_{avg} = k_h \cdot \frac{(2 - v_{avg})}{8 \cdot a}$$

Gazetas Equations for Soil Seismic Springs

In the equations that follow, the shear modulus, G, is μ_{avg} and the Poisson's ratio, ν, is v_{avg} from the preceding Christiano, et. al. equations for the vertical and horizontal modes.

Other terms in the equations are as follows:

B = ½ foundation width (parallel to y axis);
L = ½ foundation length (parallel to x axis);
h = depth to center of constant effective sidewall height;
d = constant effective sidewall height;
A_w = sidewall soil contact area, e.g. d· perimeter = d·2(2B+2L)
A_b = base soil contact area, e.g. (2B)·(2L)

$$\chi = \frac{A_b}{4L^2}$$

Vertical (z) on surface:

$$K_{z\_surf} = \frac{2GL}{1-\nu}\left(0.73 + 1.54\chi^{0.75}\right)$$

Vertical (z) embedded below surface:

$$K_{z\_emb} = K_{z\_surf}\left[1 + \frac{1}{21}\frac{D}{B}(1 + 1.3\chi)\right]\left[1 + 0.2\left(\frac{A_w}{A_b}\right)^{2/3}\right]$$

Horizontal (y) on surface

$$K_{y\_surf} = \frac{2GL}{2-\nu}\left(2 + 2.5\chi^{0.85}\right)$$

Horizontal (y) embedded below surface:

$$K_{y\_emb} = K_{y\_surf}\left(1 + 0.15\sqrt{\frac{D}{B}}\right)\left[1 + 0.52\left(\frac{h}{B}\frac{A_w}{L^2}\right)^{0.4}\right]$$

Horizontal (x) on surface

$$K_{x\_\text{surf}} = K_{y\_\text{surf}} - \frac{0.2}{0.75-\nu} GL\left(1-\frac{B}{L}\right)$$

Horizontal (x) embedded below surface

$$K_{x\_\text{emb}} = K_{x\_\text{surf}} \frac{K_{y\_\text{emb}}}{K_{y\_\text{surf}}}$$

Unit Seismic Springs

The preceding spring values have units of force/length. The springs are divided by the base soil contact area to produce unit area spring values having units of force/length^3. These springs are a composite of soil layer influences to significant depths and thus are representative of conditions anywhere on the base area of the foundation.

The UHS basin basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads).....30 kips/ft/ft^2

Vertical springs (with seismic loads)……. Lower Bound 80 kips/ft/ft^2 (Mean 121 kips/ft/ft^2, Upper Bound 182 kips/ft/ft^2)

North-south springs (with static and seismic loads) ……. Lower Bound 33 kips/ft/ft^2 (Mean 50 kips/ft/ft^2, Upper Bound 77 kips/ft/ft^2)

East-west springs (with static and seismic loads) ……..…Lower Bound 30 kips/ft/ft^2 (Mean 46 kips/ft/ft^2,Upper Bound 70 kips/ft/ft^2)

The RSW pump house basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)... .....…...……60 kips/ft/ft^2

Vertical springs (with seismic loads)……...…… Lower Bound 170 kips/ft/ft^2 (Mean 251 kips/ft/ft^2, Upper Bound 375 kips/ft/ft^2)

North-south springs (with static and seismic loads) .....Lower Bound 112 kips/ft/ft^2 (Mean 173 kips/ft/ft^2 , Upper Bound 267 kips/ft/ft^2)

East-west springs (with static and seismic loads) ……...Lower Bound 104 kips/ft/ft^2 (Mean 161 kips/ft/ft^2 , Upper Bound 248 kips/ft/ft^2)

Tables 3H.6-7 through 3H.6-9, submitted with Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated 12/30/09), include the calculated design forces and the provided reinforcement for the walls, slabs, beams, and columns of the UHS basin/UHS cooling tower/RSW pump house structures. Figures 3H.6-51 through 3H.6-136, submitted with Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated 12/30/09) show the various wall and slab reinforcement zones used to define the provided reinforcement based on the finite element analysis results. The actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement, and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The UHS/RSW pump house design used an iterative approach of checking the design axial force and moment couples for every load combination from the finite element model versus ACI 349-97 axial force and moment (P&M) interaction diagrams that were calculated based on actual reinforcement bar diameters, spacings, and layers. If the design axial force and moment couple for any load combination was outside of the allowable ACI 349-97 P&M interaction curve for a given reinforcement pattern, the design axial force and moment couples for every load combination were rechecked versus the allowable ACI 349-97 P&M interaction curve for a reinforcement pattern with a higher capacity (higher area of steel). When all of the axial force and moment couples from every load combination were within the allowable ACI 349-97 P&M interaction curve for a given reinforcement pattern, the area of steel corresponding to this reinforcement pattern plus any additional required reinforcement for in-plane shear was reported in Tables 3H.6-7 and 3H.6-8 as the "provided longitudinal reinforcing".

Please see the response to RAI 03.07.02-15, items 1 through 6 for information regarding the RSW Piping Tunnels.

References: The following references are used in this RAI response:

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

2. Gazetas, G., 1991. "Formulas and Charts for Impedances of Surface and Embedded Foundations", Journal of Geotechnical Engineering, Vol. 117, No. 9, pages 1363-1381.

3. Christiano, P. P., Rizzo, P. C., and Jarecki, S. J., 1974. "Compliances of Layered Elastic Systems", Proceedings of the Institute of Civil Engineers, Part 2, Vol. 57, December, pages 673-683.

The following COLA changes will be made to add additional details on the design of UHS/RSW Pump House.

3H.6.6.3 Structural Design

The strength design criteria defined in ACI 349 as supplemented by RG 1.142 as well as ACI 350 (note: ACI 350 is applicable only to the exterior walls below the 71 ft maximum water level and basemat of UHS basin), was used to design the reinforced concrete elements making up the UHS basin and cooling tower enclosures as well as the RSW pump house and piping tunnels. Concrete with a compressive strength of 4.0 ksi (27.6 MPa) and reinforcing steel with a yield strength of 60 ksi (414 MPa) are considered in the design.

3H.6.6.3.1 UHS Basin/UHS Cooling Tower/RSW Pump House Concrete Wall and Slab Design

The design forces and provided reinforcement for UHS basin, UHS cooling tower, and RSW pump house walls and slabs are shown in Tables 3H.6-7 and 3H.6-8. Each face and each direction of each wall and slab has a corresponding longitudinal reinforcement zone figure. Each wall and slab also has a corresponding transverse shear reinforcement zone figure when transverse shear reinforcement is required. The reinforcement zone figures (Figures 3H.6-51 through 3H.6-136) show the various zones used to define the provided reinforcement based on the finite element analysis results. Actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The shell forces from every element for every load combination in the finite element analysis were evaluated to determine the provided reinforcement in each reinforcement zone. For each reinforcement zone, the following out-of-plane moment and axial force couples with the corresponding load combination are reported in Tables 3H.6-7 and 3H.6-8:

- The maximum tension axial force with the corresponding moment acting simultaneously from the same load combination.
- The maximum compression axial force with the corresponding moment acting simultaneously from the same load combination.
- The maximum moment that has a corresponding axial tension acting simultaneously in the same load combination.
- The maximum moment that has a corresponding axial compression in the same load combination.

For each reinforcement zone, the following in-plane and transverse shears with the corresponding load combination are reported in Tables 3H.6-7 and 3H.6-8:

- The in-plane shear is the maximum average in-plane shear along a plane that crosses the longitudinal reinforcement zone.
- The transverse shear is the maximum average transverse shear along a plane in that transverse reinforcement zone.

The provided longitudinal reinforcing for each face and each direction is determined based on the out-of-plane moments, axial forces, and in-plane shears occurring simultaneously for every load combination.

The provided transverse shear reinforcing (as required) is determined based on the transverse shears and axial forces perpendicular to the shear plane occurring simultaneously for every load combination. The UHS basin and RSW pump house basemats were also evaluated for punching shear at critical locations under buttresses and columns.

The forces in the structure caused by differential settlements due to the flexibility of the basin and pump house basemats and supporting soil were accounted for through the use of foundation soil springs in the finite element model. The soil spring stiffness values used in the finite element model were based on the calculated soil subgrade modulus, which is a function of the foundation settlement.

The UHS basin basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads).....30 kips/ft/ft^2

Vertical springs (with seismic loads)……..……...80 kips/ft/ft^2

North-south springs (with static and seismic loads) ……..……33 kips/ft/ft^2

East-west springs (with static and seismic loads) ……..30 kips/ft/ft^2

The RSW pump house basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)... ..... ……60 kips/ft/ft^2

Vertical springs (with seismic loads)..... ….170 kips/ft/ft^2

North-south springs (with static and seismic loads) ..... ….112 kips/ft/ft^2

East-west springs (with static and seismic loads) ..... ….104 kips/ft/ft^2

The RSW pump house operating floor and roof were designed with composite steel beams and concrete slabs for vertical loading. The composite beams span in the east-west direction with the concrete slab designed as spanning one-way between the composite beams. The operating floor and roof slabs also act as diaphragms to transfer lateral loads. The provided reinforcing for the operating floor and roof slabs is reported in Table 3H.6-8.

3H.6.6.3.2 UHS Basin Beam and Column Design

The beams and columns in the UHS basin were represented with frame elements in the finite element model. The frame forces for every load combination in the finite element model were evaluated to determine the provided reinforcement for each beam and column in Table 3H.6-9. For each beam and column, the following forces and the corresponding load combination are reported in Table 3H.6-9:

- The maximum axial compression force with the corresponding biaxial bending moments (M2 and M3) acting simultaneously from the same load combination.
- The maximum axial tension force with the corresponding biaxial bending moments (M2 and M3) acting simultaneously from the same load combination. Note that the columns do not have an axial tension case.
- The maximum M2 bending moment with the corresponding M3 bending moment and axial force acting simultaneously from the same load combination.
- The maximum M3 bending moment with the corresponding M2 bending moment and axial force acting simultaneously from the same load combination.
- The maximum shear V2.
- The maximum shear V3.
- The maximum torsion.

The provided longitudinal reinforcing in Table 3H.6.9 is determined based on the axial force, biaxial moments (M2 and M3), and torsion. The provided stirrup reinforcing is determined based on the axial force, shears (V2 and V3), and torsion.

RAI 03.08.04-25

QUESTION:

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

The applicant's response to Question 03.08.04-15 provides a conceptual design for the interface connection between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings. The applicant states that the interface design will be finalized during detailed design. The response does not include any information regarding size, dimension, and material for the interface, or calculated data to support the displacement capacity requirement of the joint. Therefore, the applicant is requested to provide detailed information to demonstrate that the design joint has enough deformation capacity to accommodate the deformation demand that is obtained from analysis to confirm that the tunnel interface will maintain integrity, and confirm that loads due to interaction of the tunnel and the building are appropriately included in the design. The applicant is also requested to include in the FSAR critical design information pertaining to the design of the interface, e.g., separation gap, calculated differential displacement, material and stiffness properties of the interface material, etc. Please also address potential degradation of the interface material due to groundwater, in-service inspection of the interface material, and measures against potential in-leakage of groundwater.

RESPONSE:

The joint is designed to accommodate the expected relative building movements without transmitting significant forces. The separation gap between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump Houses and the Control Buildings will be at least 50% larger than the absolute sum of the calculated displacement due to seismic movements and long term settlement. The material used as flexible filler will be able to be compressed to approximately 1/3 of its thickness (based on 50% margin or a commensurate value if a margin larger than 50% is provided) without subjecting the building to more than a negligible force relative to the resistance capacity of the building.

The joint material will be a polyurethane foam impregnated with a waterproof sealing compound, or a similar material. Typical vendor data indicates that the material tensile strength is about 21 psi. Vendor testing for this material in a 5 inch joint compressed to 50% movement has a 7 psi compressive stress in the compressed condition. Considering the negligible strength and limited area of the sealing material compared to strength (minimum compressive strength (f'c) of 4000 psi) and massive size of the tunnels and abutting structures the effect on interaction between structures, if any, is negligible.

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

The value for the separation gap will be the total displacement due to seismic movements plus long term settlement calculated in detailed design. Results confirming actual gap sizing

conforming to the criteria stated above and any associated COLA change will be provided by April 15, 2010.

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

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

RAI 03.08.04-27

QUESTION:

Follow-up to Question 03.08.04-6 (RAI 2965)

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

RESPONSE:

The details of the Structural Integrity Test (SIT) and the instrumentation required for the test will be provided in the ASME Construction Specification. In accordance with RG 1.206 Section CIII.4.3 situation 4, the ASME Construction Specification will be available for review by the staff a minimum of six months before performance of the SIT. Based on the current schedule this is estimated to be approximately June 15, 2015.

The STP Units 3 and 4 COLA, Part 2, Tier 2, Section 3.8.6.3 will be revised to add the following paragraph at the end of the section.

> The details of the Structural Integrity Test (SIT) and the instrumentation required for the test will be provided in the ASME Construction Specification. The ASME Construction Specification will be provided to NRC for approval a minimum of six months before performance of the SIT.

TABLE 1 – SUPPLEMENTAL INFORMATION DATES

RAI Number	INFORMATION DESCRIPTION	STPNOC LETTER NUMBER	SUPPLEMENTAL DATE
03.07.01-18	Increase in Soil Pressure due to Structure to Structure interaction	U7-C-STP-NRC-100035	April 30, 2010
03.07.01-19	Details for Diesel Generator Fuel Oil Storage Vaults	U7-C-STP-NRC-100035	April 30, 2010
03.07.01-24	Effects of Crane Wall on the Reactor and Control Buildings	U7-C-STP-NRC-100036	April 15, 2010
03.07.02-13	Stability Evaluations for Seismic Category II Structures	U7-C-STP-NRC-100036	April 30, 2010
03.07.02-19	Seismic Input for II/I Evaluation for Radwaste Building	U7-C-STP-NRC-100035	April 30, 2010
03.07.03-3	Revise previous RAI response for Control Building Annex Input Motion based on DCD model – As previously discussed in the January 19-20 meeting	U7-C-STP-NRC-090225 U7-C-STP-NRC-100036	April 30, 2010
03.08.01-8	Effect of Increase in Pool Swell Height and Pressure	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036	April 15, 2010
03.08.04-18	Radwaste Building Analysis Results and Design Details	U7-C-STP-NRC-100036	May 31, 2010
03.08.04-19	Results of Sliding Evaluation	U7-C-STP-NRC-100036	April 30, 2010
03.08.04-25	Details of Interface Connections Between the RSW Piping Tunnel and Buildings	U7-C-STP-NRC-100036	April 15, 2010
03.08.05-2	Results of Time Rate of Settlement and Evaluation of Gaps for Site-Specific Structures	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036	April 15, 2010
03.08.05-3	Revise previous RAI response for Acceptance Criteria for Building Tilt due to Settlement – As previously discussed in the January 19-20 meeting	U7-C-STP-NRC-100018 U7-C-STP-NRC-100036	April 15, 2010