RAI 03.07.02-24, Supplement 1, Revision 1

QUESTION:

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

UHS Basin and RSW Pump House:

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

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

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

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

RSW Piping Tunnel:

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

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

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

REVISED SUPPLEMENTAL RESPONSE:

The original supplement 1 response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100253, dated November 29, 2010. This revision is being provided to clarify how the induced seismic forces at the Reactor Service Water (RSW) Piping Tunnels bends are combined with other seismic loads including seismic soil pressures based on discussions with the NRC on February 2nd and 3rd, 2011. The revisions are indicated by revision bars in the margin.

The original response to Part 1b of the UHS Basin/RSW Pump House of this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. The response to Part 1a of the UHS Basin/RSW Pump House was provided in supplement 2 of this RAI response submitted with STPNOC letter U7-C-STP-NRC-100268, dated December 14, 2010. This supplemental response provides the response to Parts 1 and 2 of the RSW Piping Tunnel.

RSW Piping Tunnel:

Part 1

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

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

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

SSI Analysis of the Typical 2D Section of RSW Tunnel

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

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

- The structural properties (i.e. mass and stiffness) for the 2D model correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 124 ft depth with halfspace below it (more than two times the horizontal dimension of RSW Piping Tunnel plus its embedment depth).
- Six cases of strain dependent soil properties representing in-situ lower bound, mean and upper bound; and backfill lower bound, mean and upper bound are considered.
- Analysis cases also include one case with cracked concrete (50% concrete modulus value) and one case with soil separation (20 ft depth).
- Concrete and mud mat damping are assigned 4% for all cases, except 7% damping is assumed for the cracked case.

- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies for both vertical and horizontal directions at least up to 32.9 Hz.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is the amplified site specific SSE motion considering the effect of nearby heavy RB and UHS/RSW Pump House structures. These amplified motions were obtained from three dimensional (3D) SSI analyses of the RB and UHS/RSW Pump House SSI analyses.
- The horizontal direction and vertical direction input motions were applied at the grade elevation.
- The responses from the horizontal and vertical direction excitations were combined using square root of sum of square (SRSS) method.
- The responses from all SSI analyses from the six soil cases, concrete cracked case and soil separation case were enveloped.
- The in-structure response spectra were peak widened by ± 15% at frequency scale.
- Envelope of the resulting response spectra for the base slab, intermediate floors and the roof slab are shown in revised COLA Part 2, Tier 2 Figures 3H.6-138 and 3H.6-139, which are used as the design in-structure response spectra for the RSW Piping Tunnel.

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

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

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

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

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

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

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

Part 2

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

In addition, a finite element analysis using a two dimensional (2D) SAP2000 model with soil springs representing the foundation was performed to confirm adequacy of the design using manual calculations described in response to RAI 03.07.02-15. Furthermore, design of the RSW Piping

Tunnel accounts for the axial tensile strain and induced forces at tunnel bends due to SSE wave propagation. The axial tensile strain is accounted for as described in COLA Part 2, Tier 2 Section 3H.6.6.2.2. The induced forces at the tunnel bends are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation. To determine the required reinforcement, the induced forces at the tunnel bends are considered to act simultaneously with all other applicable loads (including dynamic soil pressures) in the seismic load combinations.

The COLA will be revised as shown in Enclosure 1.

Enclosure 1

Revision to COLA Section 3H.6

3H.6.1 Objective and Scope

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

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

(2) RSW piping tunnel for each unit.

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

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

3H.6.2 Summary

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

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

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

- Stresses in concrete and reinforcement are less than the allowable stresses in accordance with the applicable codes listed in Subsection 3H.6.4.1.

- The factors of safety against flotation, sliding, and overturning of the structures under various loading combinations are higher than the required minimum values identified in Subsection 3H.6.4.5.

- The calculated static and dynamic soil bearing pressures/displacements are less than the allowable values.

- The thickness of the roof slabs and exterior walls are more than the minimum required to preclude penetration, perforation, or spalling resulting from impact of design basis tornado missiles. In addition, the passage of tornado missiles through openings in the roof slabs and exterior walls is prevented by the use of missile- proof covers and doors, or the trajectory of missiles through ventilation openings is limited by labyrinth walls configured to prevent safety-related substructures and components from being impacted.

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

3H.6.4.3.1.4 Lateral Soil Pressures (H)

Lateral soil pressures are calculated using the following soil properties.

- Unit weight (moist):..120 pcf (1.92 t/m3)

- Unit weight (saturated): ....140 pcf (2.24 t/m3)

- Internal friction angle:...30°

- Poisson's ratio (above groundwater)..0.42

- Poisson's ratio (below groundwater)..0.47

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

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43.

- ~~Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44.~~

- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

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

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

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

3H.6.5.3 Seismic Analysis of RSW Piping Tunnels

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

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

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

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

<u>SSI Analysis of the Typical 2D Section of RSW Tunnel</u>

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

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

- The structural properties (i.e. mass and stiffness) for the 2D model correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 124 ft depth with half space below it (more than two times the horizontal dimension of RSW Piping Tunnel plus its embedment depth).
- Six cases of strain dependent soil properties representing in-situ lower bound, mean and upper bound; and backfill lower bound, mean and upper bound are considered.
- Analysis cases also include one case with cracked concrete (50% concrete modulus value) and one case with soil separation (20 ft depth).
- Concrete and mud mat damping are assigned 4% for all cases, except 7% damping is assumed for the cracked case.
- Groundwater is considered at 8 ft depth. The ground water effect is included by using minimum P-wave velocity of 5000 ft/sec except for cases where use of this minimum P-wave velocity results in Poisson's ratio in excess of 0.495.
- Model is capable of passing frequencies for both vertical and horizontal directions at least up to 32.9 Hz.
- Cut-off frequency for transfer function calculation is 33 Hz.
- Input motion is the amplified site specific SSE motion considering the effect of nearby heavy RB and UHS/RSW Pump House structures. These amplified motions were obtained from three dimensional (3D) SSI analyses of the RB and UHS/RSW Pump House SSI analyses.
- The horizontal direction and vertical direction input motions were applied at the grade elevation.
- The responses from the horizontal and vertical direction excitations were combined using square root of sum of square (SRSS) method.
- The responses from all SSI analyses from the six soil cases, concrete cracked case and soil separation case were enveloped.
- The in-structure response spectra were peak widened by ± 15% at frequency scale.
- Envelope of the resulting response spectra for the base slab, intermediate floors and the roof slab shown in Figures 3H.6-138 and 3H.6-139 are used as the design in-structure response spectra for the RSW Piping Tunnel.

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

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

- The structural properties (mass and stiffness) for the 2D model of the individual structures correspond to per unit depth (1 ft dimension in the out-of-plane direction) of the respective structure.

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

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

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

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

3H.6.6.2.2 RSW Piping Tunnels

The individual components of the RSW Piping Tunnels (roof slab, intermediate slabs, base mat and walls) have out-of-plane frequency in excess of 33 Hz and their out-of-plane seismic loads are determined using a conservative acceleration of 0.21g which exceeds the maximum Zero Period Acceleration (ZPA) of response spectra

Figures 3H.6-138 and 3H.6-139. Manual calculations are used for the analysis and design of individual components of the RSW Piping Tunnels (roof slab, intermediate slab, base mat, walls) considering all applicable loads and load combinations including dead load, live load, earth pressure loads, wind and tornado loads, SSE seismic loads, internal flood loads and external flood loads.

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

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

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

- Rayleigh waves with apparent wave velocity of 3,000 ft/sec (as recommended in appendix C3.5.2.1 of ASCE 4-98)
- Conservative ground acceleration of 0.21g
- Maximum ground velocity of 10.08 in/sec (which is based on 48 in/sec per 1.0g ground acceleration)

The tensile axial strain and strain due to maximum curvature are conservatively added together to obtain the actual strain in the longitudinal direction of the RSW Tunnel. The actual strain is then compared to the cracking strain of concrete and maximum allowable strain of the reinforcing. The maximum computed tensile axial strain is 2.9×10^{-4} in/in which is about 14% of the rebar yield strain of 2.069×10^{-3} in/in. The design also accounts for the induced forces at tunnel bends due to SSE wave propagation. These forces are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation. To determine the required reinforcement, the induced forces at the tunnel bends are considered to act simultaneously with all other applicable loads (including dynamic soil pressures) in the seismic load combinations.

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

- Dead load of the tunnel walls and the soil above the tunnel.
- Live load of 200 psf (9.6 kPa) applied to the floor of the tunnels.
- At-rest lateral soil pressure on the tunnel walls.
- Hydrostatic pressures on the tunnel walls due to groundwater.
- Envelope of d~~D~~ynamic lateral soil pressures on the tunnel walls, due to an SSE, calculated from: (a) ~~calculated~~ using the methodology defined in Subsection 3.5.3.2.2 of ASCE 4-98, (b) soil-structure interaction (SSI) analysis, and (c) the structure-soil-structure interaction (SSSI) analysis. At rest ~~L~~lateral soil pressures for typical section of the ~~used for design of~~ RSW Piping Tunnels using ASCE 4-98 methodology are presented in Figure 3H.6-44. Figures 3H.6-212 through 3H.6-215 provide comparison of lateral seismic soil pressures from SSSI analysis described in Section 3H.6.5.3 to those from ASCE 4-98 methodology.
- Surcharge pressure of 500 psf (23.9 kPa) applied to the ground above the tunnels.
- SSE forces corresponding to the weight of the tunnels being acted on by the accelerations established by the SSI analysis.

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

3H.6.6.5 Stability Evaluations

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

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

Location	Item	Thickness (ft)	Governing Load Combination	Design Moment (kip-ft/ft)	Design Shear (kip/ft)	Area of Reinforcement (in^2/ft)			
						Moment Reinforcement [1]		Shear Reinforcement	
						Required	Provided (both faces)	Required	Provided
Main Tunnel	Exterior Wall	3'-0"	1.4D+1.7L+1.4F+1.7H	136.47	21.95	1.16 (vertical)	1.27 (vertical)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	55.90	11.29	0.7 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	95.22	13.16	1.13 (east-west)	1.27 (east-west)	None	None
	Basemat	3'-0"	D+Lo+F+H'+E' [2]	123.94	19.10	0.97 (east-west)	1.00 (east-west)	None	None
North End of Main Tunnel (near Control Building)	Exterior Wall	3'-0"	1.4D+1.7L+1.4F+1.7H	324.37	34.23	2.19 (east-west)	2.25 (east-west)	None	None
	Interior Wall	2'-0"	D+Lo+F+H'+E' [2]	152.15	19.96	1.69 (east-west)	2.25 (east-west)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	86.64	15.29	0.70 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	136.30	16.03	1.49 (east-west)	2.25 (east-west)	None	None
	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	70.42	28.27	0.36 (north-south)	0.79 (north-south)	None	None
			1.4D+1.7L+1.4F+1.7H	155.74	36.39	1.16 (east-west)	1.27 (east-west)	None	None
Main Tunnel (near Access Region 1)	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	46.60	20.54	0.70 (north-south)	0.79 (north-south)	None	None

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

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

Notes:

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

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

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

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

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

Notes:

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

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

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

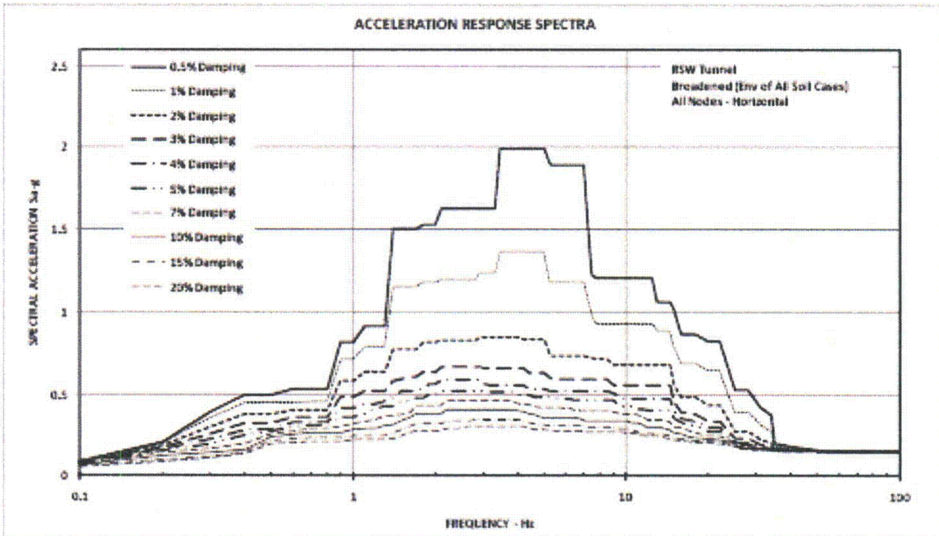


Figure 3H.6-138: RSW Piping Tunnel, Horizontal Response Spectra

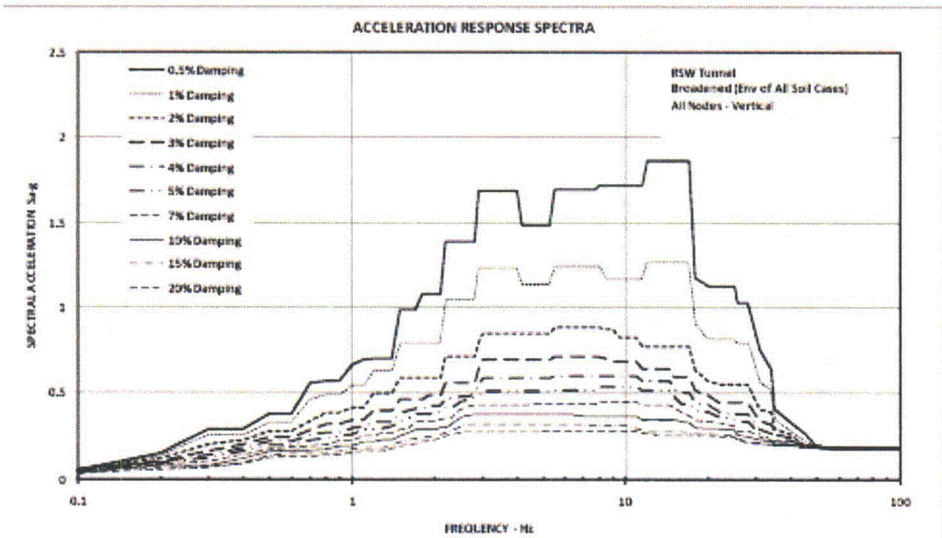


Figure 3H.6-139: RSW Piping Tunnel, Vertical Response Spectra

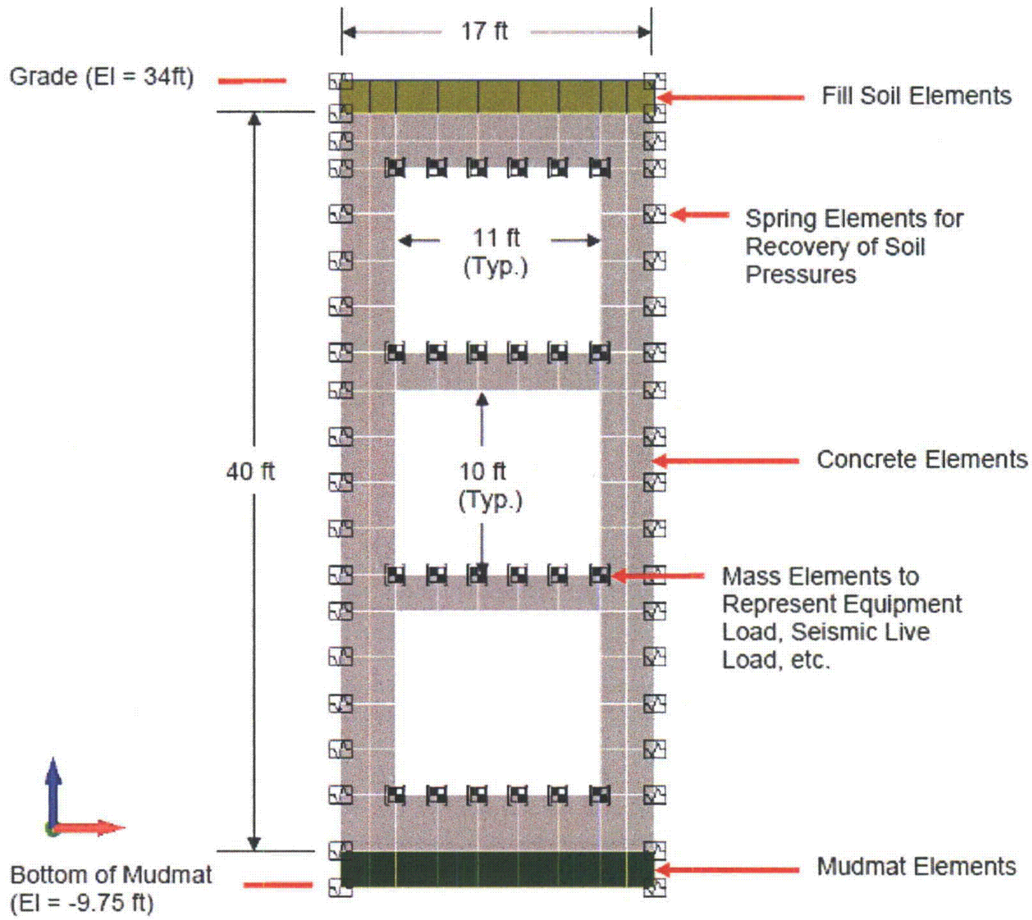


Figure 3H.6-209: SSI Model of RSW Piping Tunnel

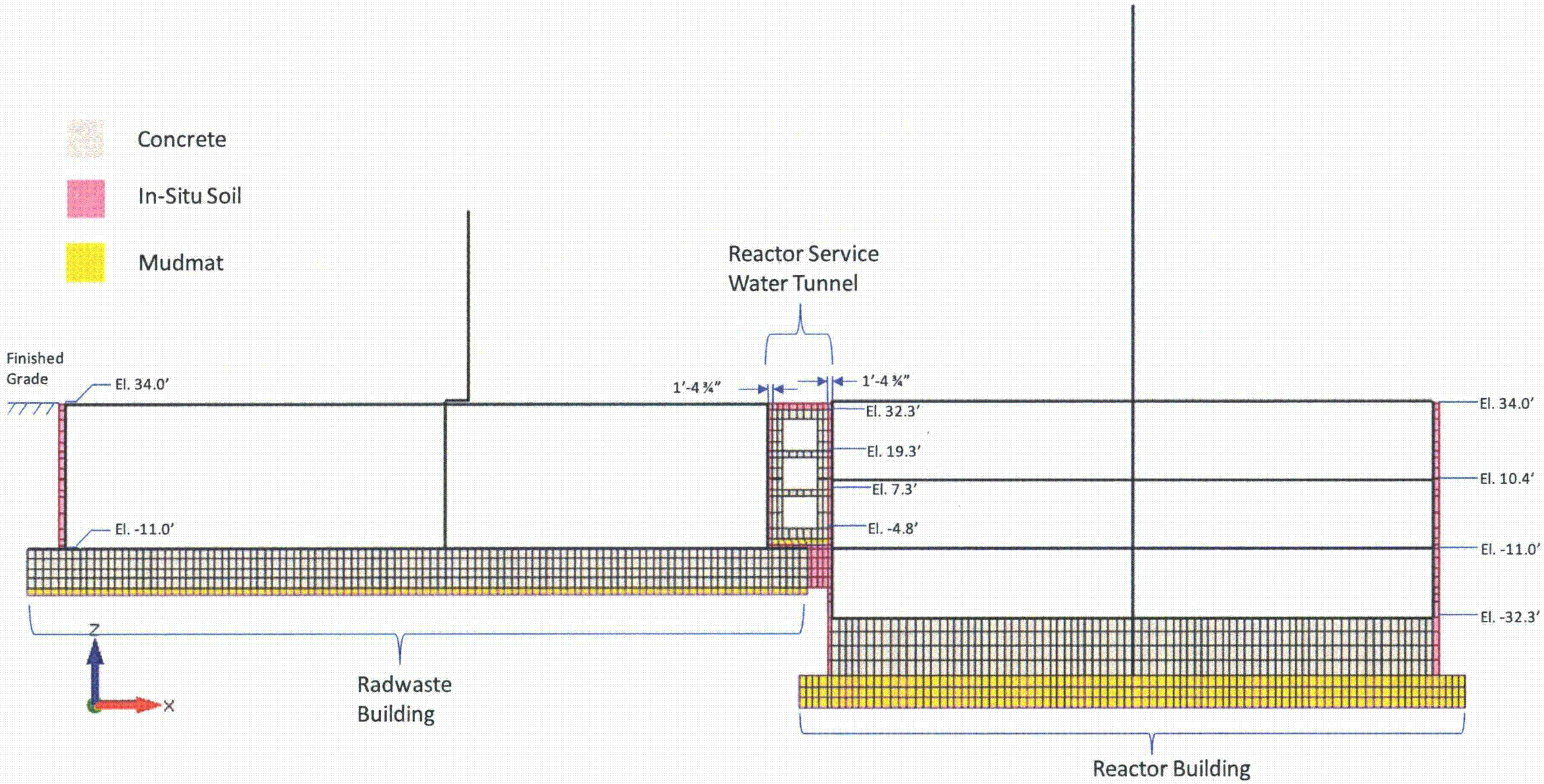


Figure 3H.6-210: SSSI 2D Model of RB + RSW Piping Tunnel + RWB

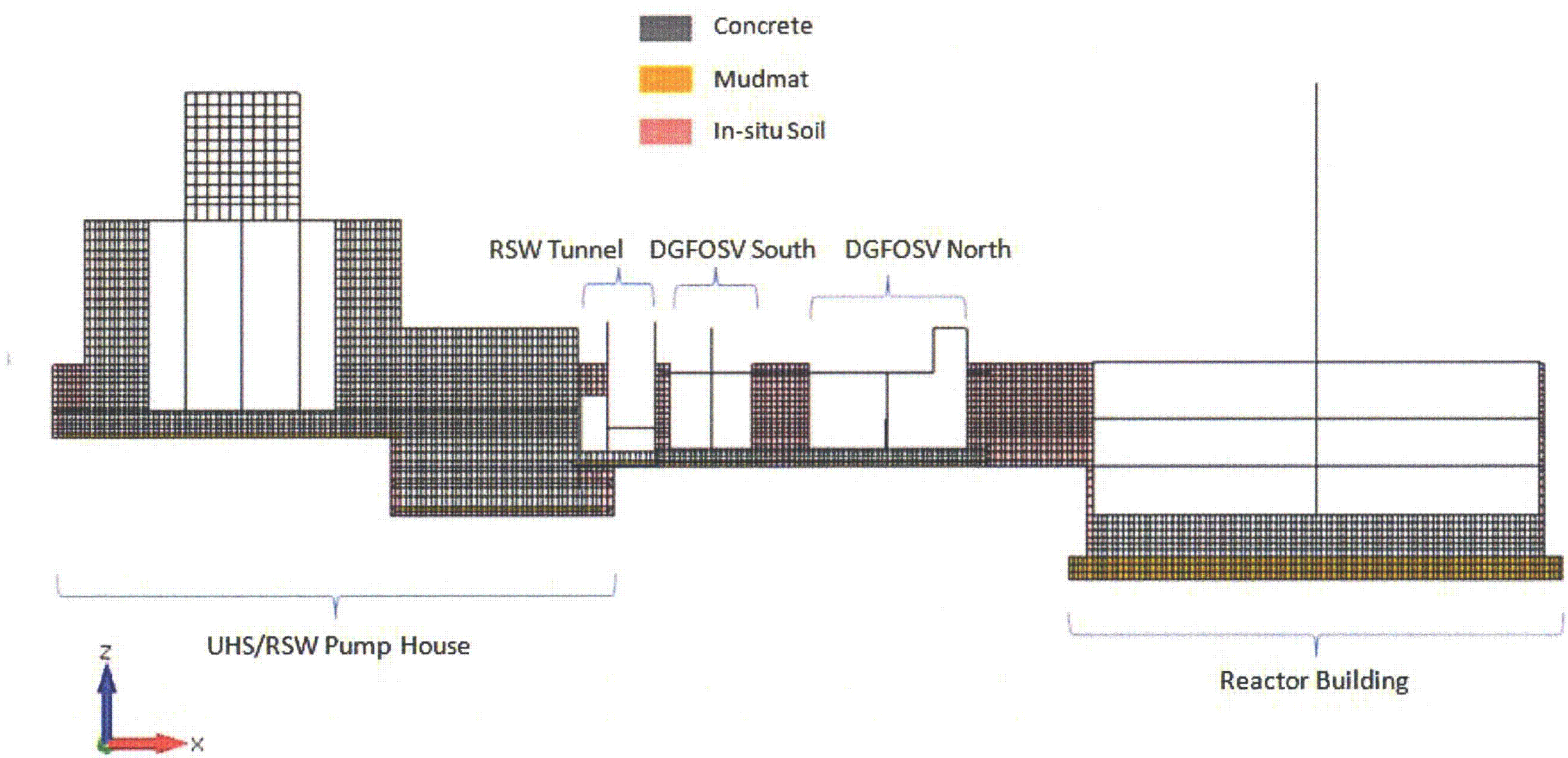


Figure 3H.6-211: 2D Model of UHS/RSW Pump House, RSW Piping Tunnel, DGFOSVs and RB

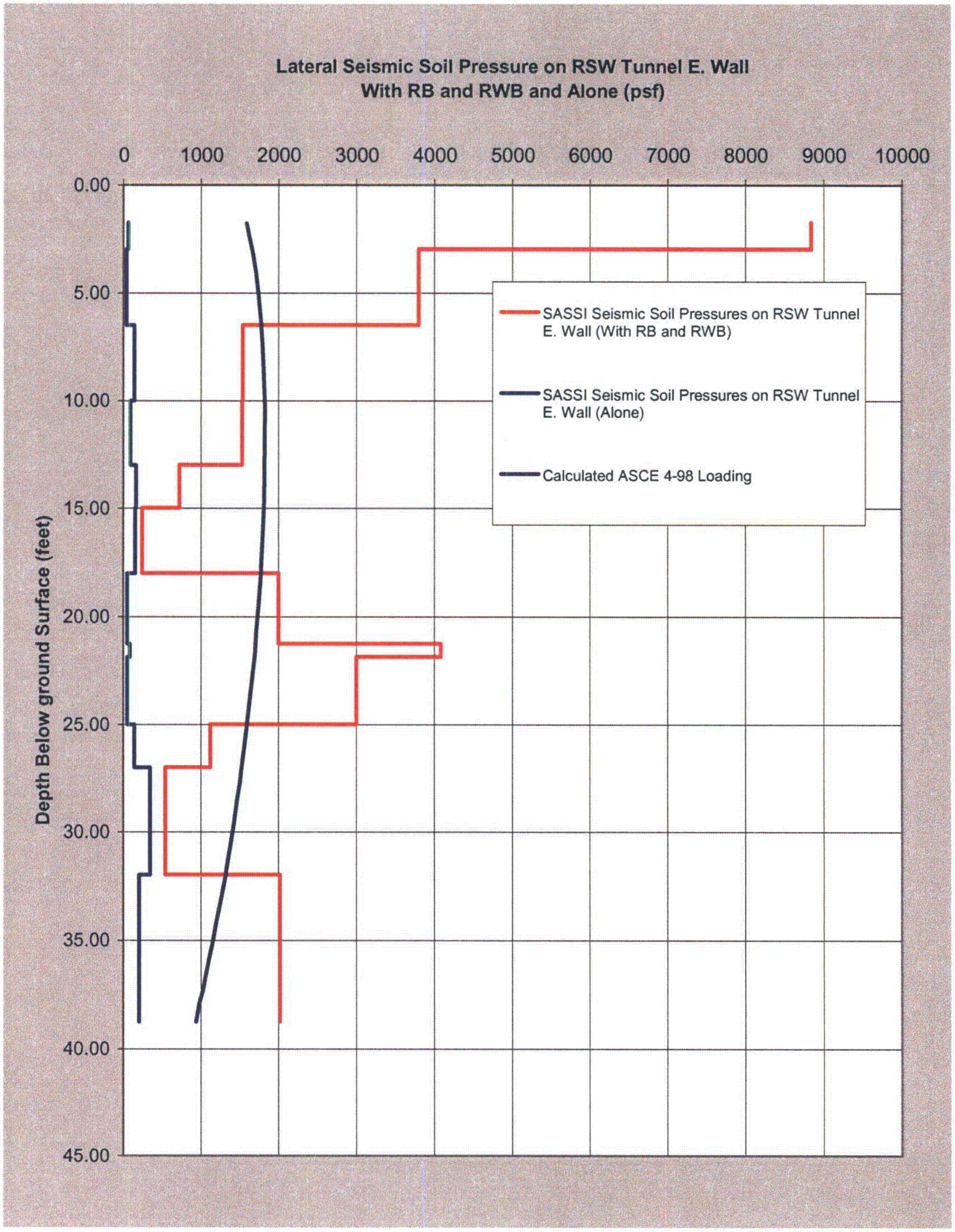


Figure 3H.6-212: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel East Wall (Main Cross Section of RSW Piping Tunnel)

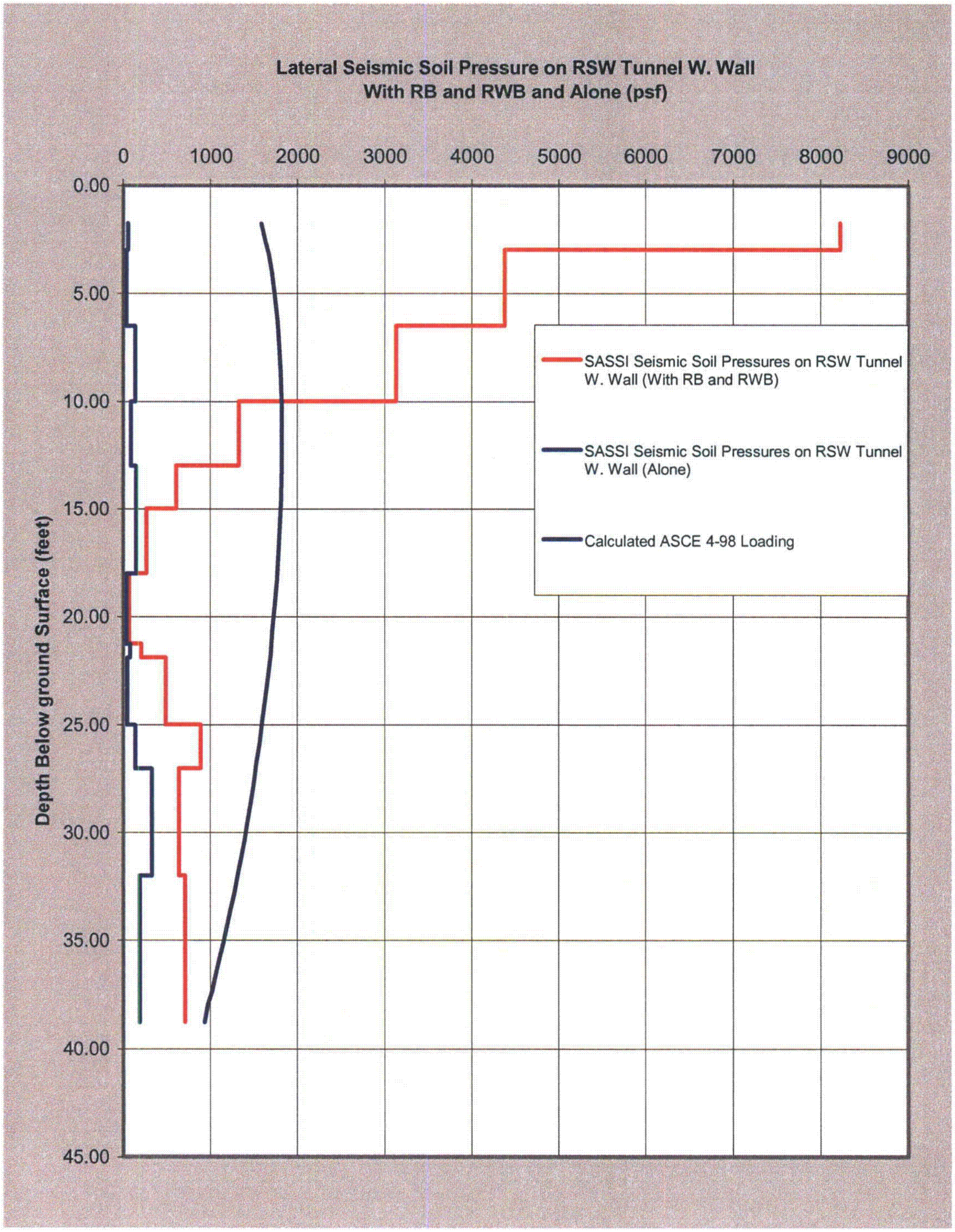


Figure 3H.6-213: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel West Wall (Main Cross Section of RSW Piping Tunnel)

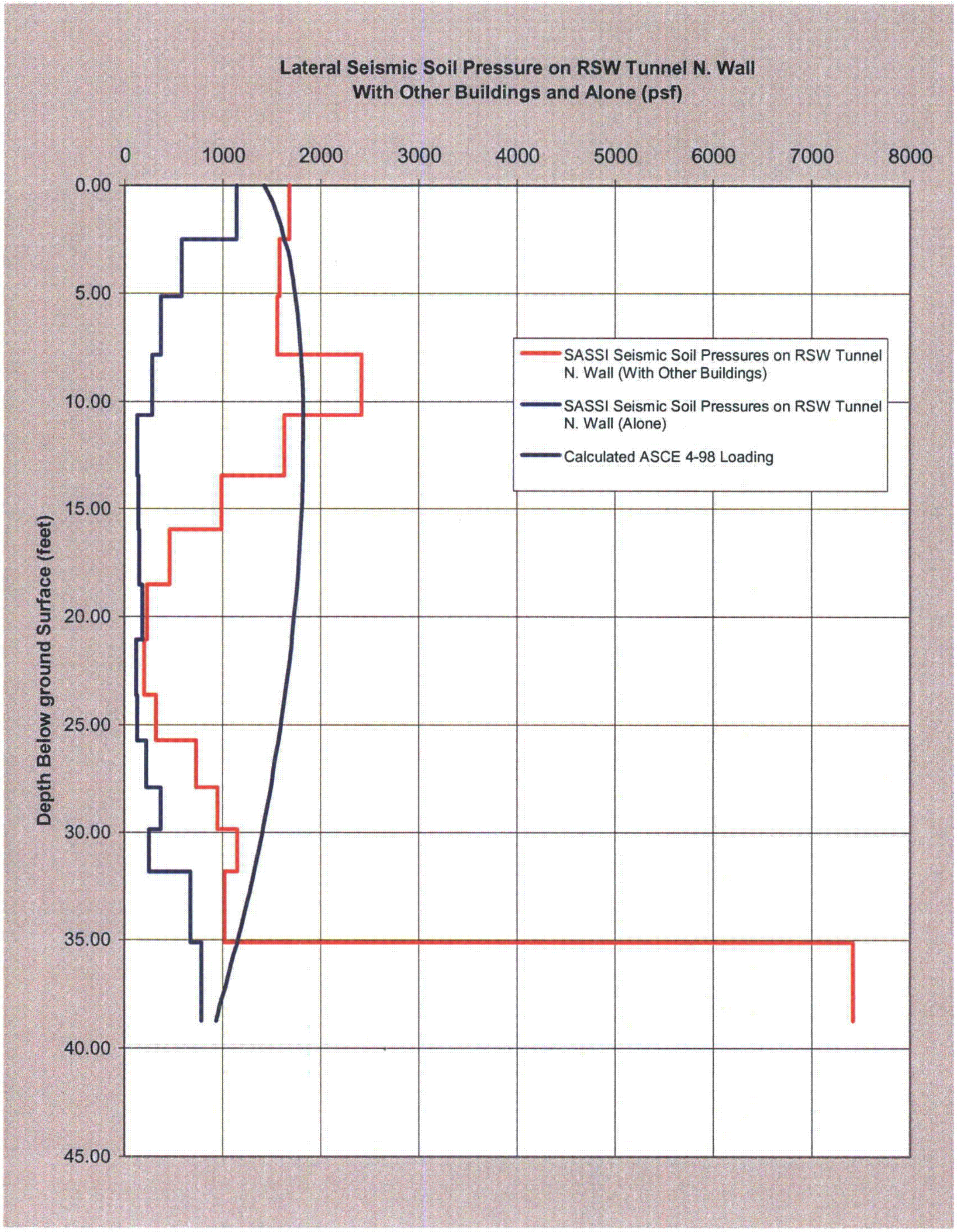


Figure 3H.6-214: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel North Wall (RSW Piping Tunnel near UHS/RSW Pump House)

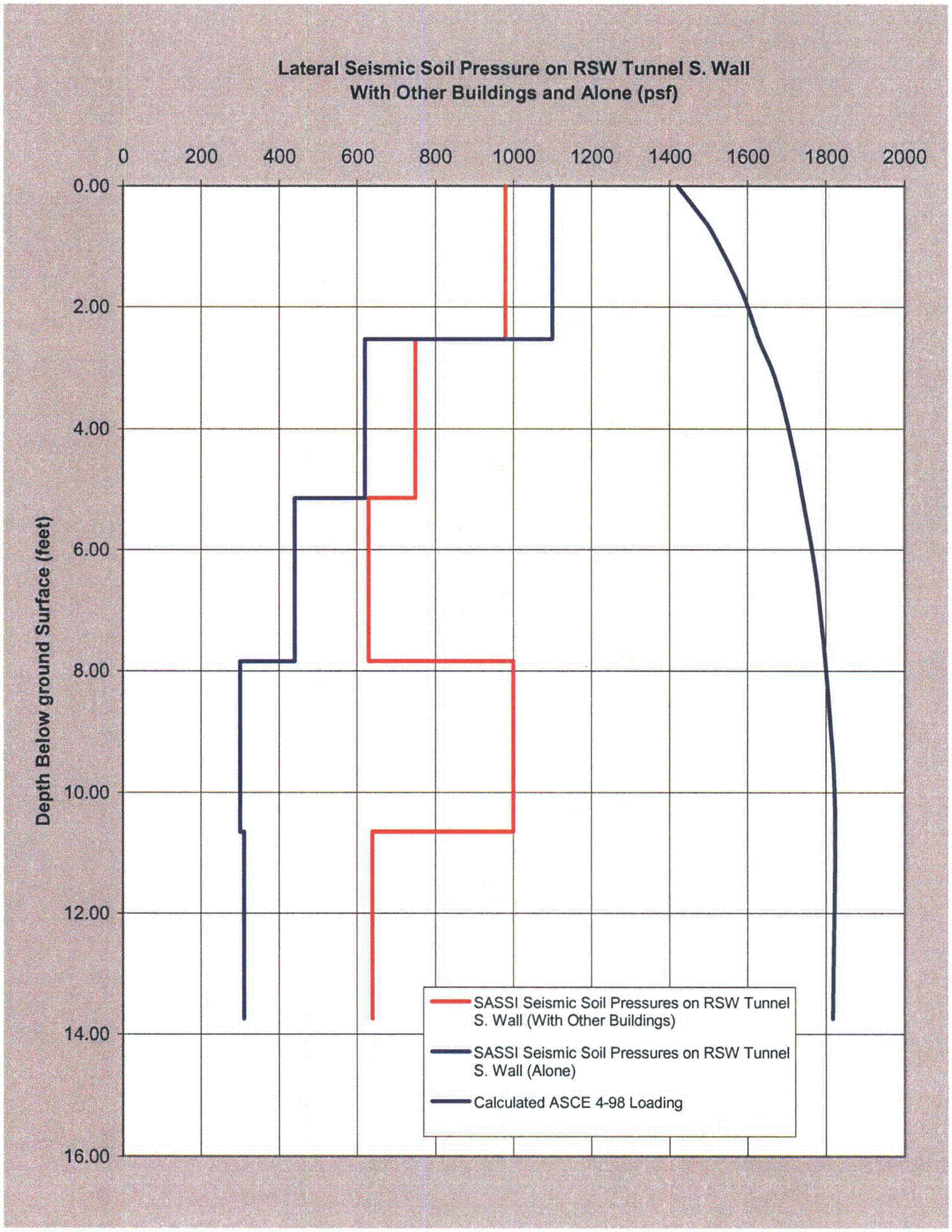


Figure 3H.6-215: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel South Wall (RSW Piping Tunnel near UHS/RSW Pump House)

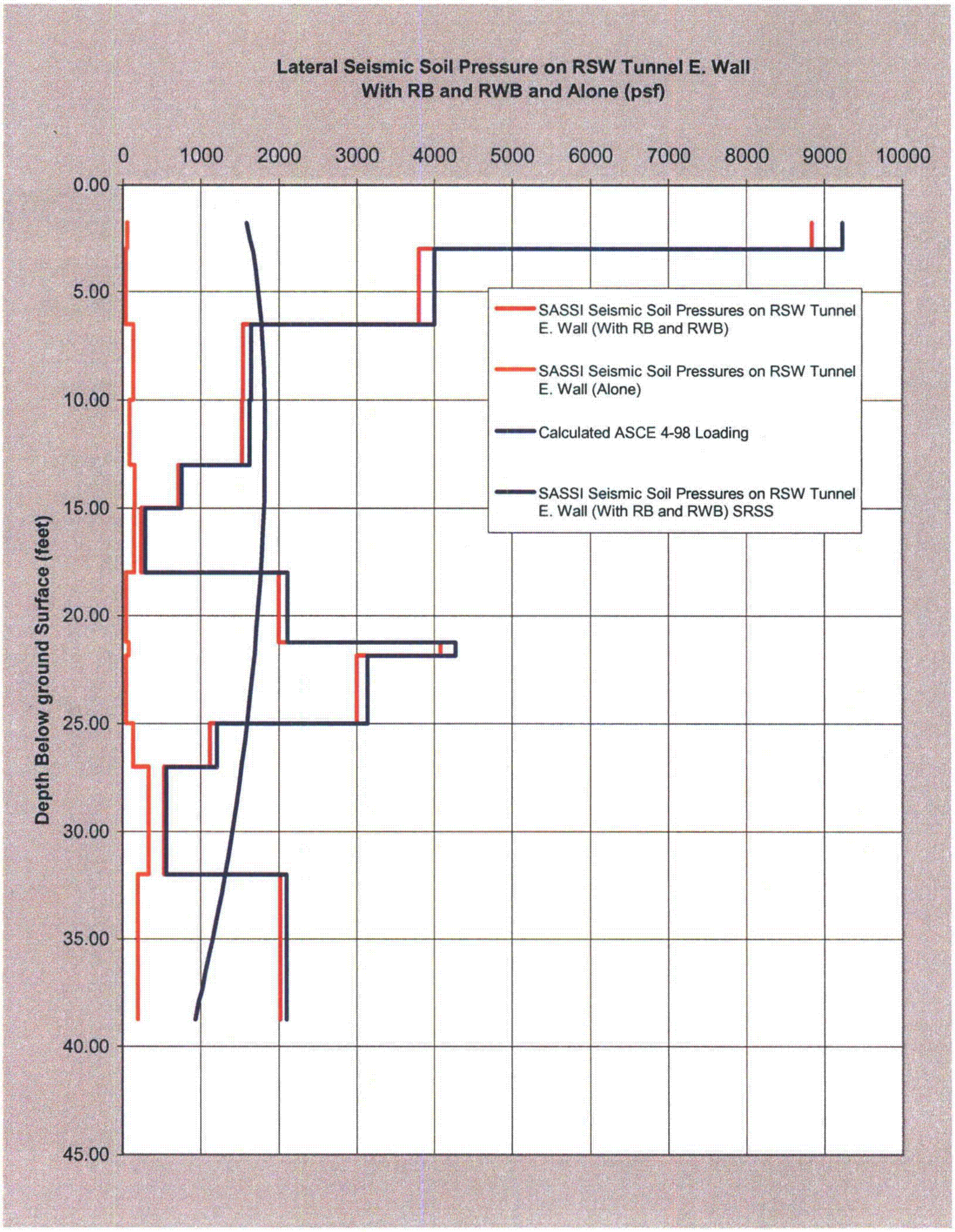


Figure 3H.6-216: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel East Wall (Main Cross Section of RSW Piping Tunnel, Including Effect of Vertical Excitation)

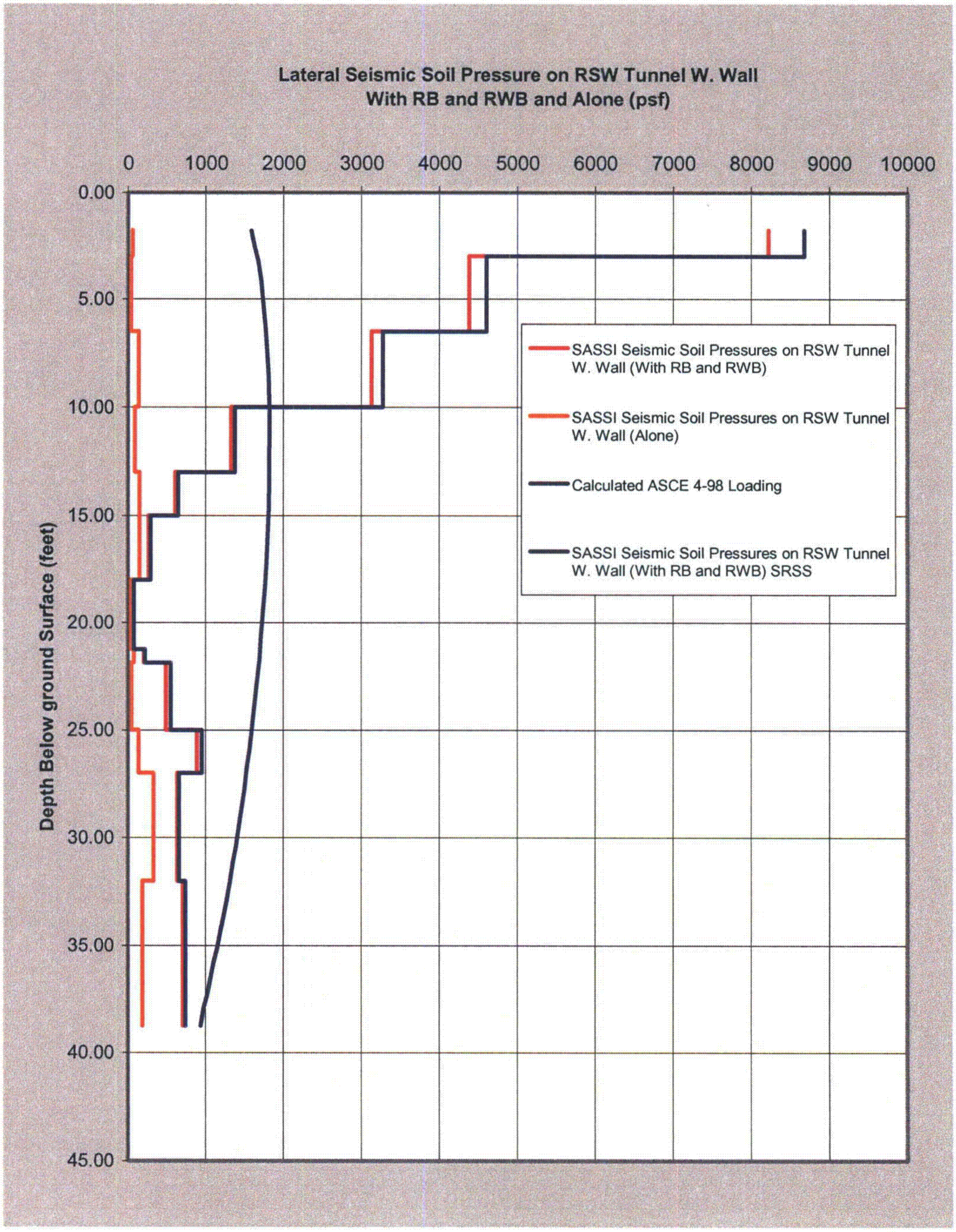


Figure 3H.6-217: Lateral Seismic Soil Pressures (psf) on RSW Piping Tunnel West Wall (Main Cross Section of RSW Piping Tunnel, Including Effect of Vertical Excitation)

RAI 03.08.01-9, Revision 2

QUESTION:

Follow-up to Question 03.08.01-6

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

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

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

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

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

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

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

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

    a. The allowable leakage of 1/10 gallon per linear foot of gasket per hour may potentially allow ingress of significant amount of water over time. Please provide justification why this leakage is considered to meet criterion for Type 2 closure, which is defined to form essentially dry barriers or seals, and the basis for the underlying assumption that such leakage will not compromise functionality of any safety related commodity or any other design basis.

    b. Since hydrostatic pressure on the door may help in providing a seal for the door, please explain why testing these doors against the maximum water pressure only is adequate, and will envelope performance of the seals during lower hydrostatic pressure.

    c. Since the applicant did not include in its response any information about the in-service surveillance programs for the watertight doors, and corresponding FSAR update, please explain how availability of the normally open watertight doors during a flooding event is ensured considering that these doors will need to be closed upon indication of an imminent flood.

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

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

REVISED RESPONSE:

The original response to RAI 03.08.01-09 was submitted with STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010. Revision 1 of the response to RAI 03.08.01-9 was submitted with STPNOC letter U7-C-STP-NRC-100253 dated November 29, 2010. Both previous revisions of this response are completely superseded by this revised response. The revisions are indicated by revision bars in the margin. This revision is based on the discussions with NRC in a meeting held on February 2nd and 3rd, 2011. The revised response includes the following requirements:

- The interior redundant water stops are to be Seismic Category I components.
- The testing program will demonstrate that the seal material can withstand ±25% movement in any resultant direction (due to settlement) and still be watertight.

- Testing will ensure that the seal material will function as a watertight barrier following Safe Shutdown Earthquake (SSE).
- The water stop on the interior side of the joint will be tested to withstand the SSE maximum displacements without degradation.
- Flood load requirements are clarified.

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

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

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

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

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

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

$1.6S = D + P + E'$

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

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

Where:

$S =$ Normal allowable stresses as defined in AISC N690
$D =$ Dead loads
$P_o =$ Normal Operating Differential Pressure
$E' =$ Loads generated by SSE, per Sections 3H.1 and 3H.2.
$FL =$ Design basis extreme flood loads, including the hydrostatic load due to flood elevation at 40 ft MSL, the associated drag effects of 44 psf, impact due to floating debris per Section 3.4.2, and hydrodynamic load due to wind-generated wave action per Figure 3.4-1(Figure 3.4-1 shall only be used to calculate hydrodynamic load due to wind-generated wave action). The weight of the water (above ground) due to the flood loads shall be 63.85 pcf in order to include the effects of suspended sediments in the water. (Figure 3.4-1 and revised Section 3.4.2 are included in the revised response to RAI 03.04.02-11, Revision 1, submitted with STPNOC letter U7-C-STP-NRC-100253, dated November 29, 2010).
$W =$ Normal wind loads, per DCD Sections 3H.1 and 3H.2
$W_t =$ Tornado loads per DCD Sections 3H.1 and 3H.2, including wind velocity pressure W_w, differential pressure W_p, and tornado-generated missiles (if not protected) W_m

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

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

4. The watertight doors will be designed by vendors in accordance with specific requirements given in the procurement specification. The procurement specification will

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

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

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

There are less than 1000 linear feet of gasket material for all the watertight doors used for protection against external flooding. A leakage rate of 1/10 gallon per linear foot of gasket per hour equates to 100 gallons/hour or 0.006 m^3/min. The allowable leakage of 1/10 gallon per linear foot of gasket per hour is far less than the 1.34 m^3/min accepted for internal flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for internal flooding in the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. The safety related equipment potentially subjected to external flooding is protected by curbs and raised equipment pads, similar to the safety related equipment potentially subjected to internal flooding.

5b. During the test, the hydrostatic head will be raised at a rate not more than 1 ft/min to a level of 25% higher than the flood level. Any leaks that occur during this time will be detected and if the leakage rate begins to diminish as the hydrostatic head increases, the assembly will be tested at a lower hydrostatic head. This requirement is added to the COLA markup provided in the revised response to RAI 03.04.02-6, Revision 3, being submitted concurrently with this response.

5c. The revised responses to RAI 03.04.02-6, Revision 3 (being submitted concurrently with this response) and RAI 19-30, Revision 2 (submitted with STPNOC letter U7-C-STP-NRC-100175 dated July 28, 2010) now state that all doors that protect against the design basis flood will be normally closed. For requirements pertaining to inspection and maintenance, see the response to RAI 03.04.01-6 submitted with STPNOC letter U7-C-STP-NRC-090045 dated May 13, 2009.

6. The joint seals between the Reactor Building and the Control Building below the design basis flood level will be made using a polyurethane foam impregnated with a waterproof sealing compound between the concrete surfaces and an interior redundant water stop.

The seal material and joint seal assembly shall be tested to be watertight when subjected to the maximum anticipated hydrostatic head. The testing program will demonstrate the following:

- The seal material can withstand movement ±25% of the gap size in any resultant direction and still be watertight
- The seal material can compress to 1/3 of its thickness without developing more than 25 psi pressure on the adjacent structures.
- The entire joint seal assembly, including the watertight joint seal and redundant water stop, prevents the total leakage during an SSE event from exceeding that which would cause internal flooding to exceed the height of the flooding protection curbs or raised equipment pads, which will ensure that the joint seal assembly limits leakage to a level that adequately protects the safety related equipment and components (For example, in the Clean Access Corridor, a leakage of 2 ft^3 of water per linear foot of seal would equate to a maximum water level of 8" (200 mm), which is below all water-sensitive safety related equipment per DCD/COLA Section 3.4.1.1.2). The total leakage of the joint seal assembly shall be determined for the entire duration of the SSE when subjected, simultaneously, to the maximum anticipated hydrostatic head pressure, the maximum differential displacements due to long term settlement or tilt, and the maximum differential displacements due to SSE.
- The seal material will function as a watertight barrier after being subjected to the maximum displacements due to a SSE and the redundant water stop on the interior side of the joint can withstand the SSE maximum displacements without degradation.

The foregoing requirements will demonstrate that the material is capable of being watertight after the effects of long term settlement or tilt, as well as during normal operating vibratory loading such as SRV actuation and not impact the adjacent structures.

The lowest required watertight joint seal is in the slab at nominal elevation 4.8m (the lowest elevation of the Clean Access Corridor between the Reactor Building and Control Building) and the hydrostatic head associated with this watertight joint seal is not anticipated to exceed 35 ft. The watertight joint seal and interior redundant water stops used to protect the safety-related buildings against external water entry are classified as Seismic Category I with respect to their ability to remain in-place to stop significant water leakage into the safety-related buildings during and after a seismic event. The gap size is determined based on the displacement under a SSE load plus long-term settlement, similar to the joints discussed in RAI 03.08.04-25, submitted with STPNOC letter U7-C-STP-NRC-100108 dated May 13, 2010. Movements of ±25% of the gap size will envelope any expected displacements anticipated under normal settlement loading. This will show that the watertight joint seal material is capable of being watertight after the effects of long-term settlement and tilt, as well as during normal operating vibratory loads, such as SRV actuation. Although this will provide margin to accommodate additional differential displacements from the majority of the movements from short duration extreme environmental loading, such as SSE and tornado, the watertight joint seals need not be designed to be watertight during the differential displacements from these extreme environmental loadings. For these events, the interior redundant water stop will act as a water-resistant barrier, which will only allow slight leakage during the event. Because of the interior water stop, leakage during local seal failure due to extreme environmental loading events will be less than the 1.34 m^3/min accepted for flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for flooding in the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. An in-service inspection program will ensure that the watertight joint seals and interior water stops do not significantly degrade during normal plant operation and after being subjected to an extreme environmental loading event. This will ensure that the watertight joint seals and interior water stops adequately protect safety-related equipment from significant leakage of water into the Reactor Building and Control Building. The requirements discussed above are added to the COLA markup provided in response to RAI 03.04.02-6, Revision 3.

The COLA markups resulting from this response are included in the revised COLA markup included in the revised response to RAI 03.04.02-6, Revision 3, being submitted concurrently with this response. No additional COLA revision is required as a result of this response.

RAI 03.08.04-17, Supplement 1

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.

SUPPLEMENTAL RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100036 dated February 10, 2010. This supplemental response provides clarifications and additional information on the lateral soil pressure acting on various structures, as discussed in meetings with NRC on February 2nd and 3rd, 2011.

Diesel Generator Fuel Oil Storage Vault (DGFOSV):

The structure-soil-structure interaction (SSSI) incremental seismic soil pressure curves for the Diesel Generator Fuel Oil Storage Vault (DGFOSV) have been provided in COLA Part 2, Tier 2, Figures 3H.6-226 through 3H.6-231, in the response to RAI 03.07.01-27, Supplement 1, Revision 1, which is being submitted concurrently with this response. The at-rest, dynamic at-rest, active, and passive soil pressure profiles are provided in Figures 3H.6-241 through 3H.6-244, included in Enclosure 1 of this response.

Reactor Service Water (RSW) Tunnel:

The SSSI incremental seismic soil pressure curves for the Reactor Service Water (RSW) Tunnel have been provided in COLA Part 2, Tier 2, Figures 3H.6-212 through 3H.6-217, in the response to RAI 03.07.02-24, Supplement 1, Revision 1, which is being submitted concurrently with this response. The at-rest, dynamic at-rest, active, and passive soil pressure profiles are provided in Figures 3H.6-44, and 3H.6-245 through 3H.6-247, included in Enclosure 1 of this response.

Ultimate Heat Sink (UHS) and RSW Pump House:

The SSSI incremental seismic soil pressure curves for the Ultimate Heat Sink (UHS) and RSW Pump House will be provided in response to RAI 03.07.02-22, currently scheduled to be submitted by March 15, 2011. The at-rest, dynamic at-rest, active, and passive soil pressure profiles are provided in Figures 3H.6-41 through 3H.6-43 and 3H.6-232 through 3H.6-240, included in Enclosure 1 of this response.

Diesel Generator Fuel Oil Tunnel (DGFOT):

The SSSI incremental seismic soil pressure curves for the standard plant Diesel Generator Fuel Oil Tunnel (DGFOT) have been provided in COLA Part 2, Tier 2, Figures 3H.7-5 through 3H.7-8, in the response to RAI 03.08.04-30, submitted with STPNOC letter U7-C-STP-NRC-110008, dated January 17, 2011. The at-rest, dynamic at-rest, active and passive soil pressure profiles are provided in Figures 3H.7-2 and 3H.7-33 through 3H.7-35, included in Enclosure 2 of this response.

Control Building:

The SSSI incremental seismic soil pressure profiles were provided in COLA Part 2, Tier 2, Figure 3A-302, in the response to RAI 03.07.01-26, Revision 1, which is being submitted concurrently with this response. Remaining soil pressure profiles were provided in the original response to this RAI.

Reactor Building:

The SSSI incremental seismic soil pressure profiles were provided in COLA Part 2, Tier 2, Figure 3A-301, in the response to RAI 03.07.01-26, Revision 1, which is being submitted concurrently with this response. Remaining soil pressure profiles were provided in the original response to this RAI.

Additional SSSI incremental seismic soil pressure profiles will be provided with RAI 03.08.04-30, Supplement 1, currently scheduled to be submitted by March 15, 2011.

RAI 03.08.04-17, Supplement 1
Enclosure 1

COLA Part 2, Tier 2, Section 3H.6

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/m^3)
- Unit weight (saturated): ....140 pcf (2.24 t/m^3)
- 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 and Figures 3H.6-232 through 3H.6-240.
- Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44, Figures 3H.6-245 through 3H.6-247.
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

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

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

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43 and Figures 3H.6-232 through 3H.6-240.
- Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44 and Figures 3H.6-245 through 3H.6-247.
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

3H.6.7 Diesel Generator Fuel Oil Storage Vaults (DGFOSV)

The Diesel Generator Fuel Oil Storage Vaults (DGFOSV) are reinforced concrete structures, located below grade with an access room above grade. The DGFOSV house fuel oil tanks and transfer pumps. The DGFOSV are buried in the structural back-fill. The embedment depth to the bottom of the 2 ft thick mudmat is approximately 45 ft, the maximum height from the bottom of the mudmat is approximately 61 ft, and the basemat dimensions are approximately 81.5 ft by 48 ft. Properties of the backfill are described in Section 3H.6.5.2.4. A 3-dimensional SAP2000 response spectrum analysis was used to obtain the SSE design forces due to structure inertia. The seismic induced dynamic soil pressures on DGFOSV walls and roof were computed using the method of ASCE 4-98, Subsection 3.5.3.2.

Two DGFOSV are located about 50 feet away from the south face of the Reactor Building (RB), which is a heavy multistory structure. The third DGFOSV is located approximately 38 feet away from the north face of the Reactor Service Water (RSW) Pump House. Considering the soil profile at the STP Units 3 & 4 site, the induced acceleration at the foundation level of the DGFOSV during a safe-shutdown earthquake (SSE) event may be amplified due to their close proximity to the RB (for the two) or the RSW Pump House (for the third). To establish the input motion for the soil-structure interaction (SSI) analysis of the DGFOSV, considering the impact of the nearby heavy RB (for the two) and RSW Pump House (for the third) structures, an analysis as described below was performed.

Five interaction nodes at the ground surface and five at the depth corresponding to the bottom elevation of the DGFOSV foundations are added to the three dimensional SSI SASSI2000 model of the RB for obtaining free field responses for the two DGFOSV close to the RB. These five nodes correspond to the four corners and the center of the DGFOSV. This RB SSI model is analyzed for the STP site-specific SSE. For each of these two DGFOSV, first an average of the spectra at five nodes at the surface and foundation each is calculated and then envelope of the two average spectra is calculated. A similar SSI analysis is performed for the third DGFOSV close to the RSW Pump House. Finally, the envelope of the envelope average spectra for the three DGFOSV and the 0.3g Regulatory Guide 1.60 response spectrum is used as the input response spectrum for the SSI analysis of the DGFOSV. The DGFOSV and the equipment and components inside the vault are designed using the results of the SSI analysis.

The comparison of response spectra (the minimum required 0.1g Regulatory Guide 1.60 spectra, the FIRS, and the deconvolved SHAKE outcrop spectra) at the foundation level of the DGFOSV is presented in Figures 3H.6-11d through 3H.6-11L. As can be seen from these figures, the deconvolved SHAKE outcrop spectra envelop the minimum required spectra and FIRS for the three sets of soil properties.

The applicable codes, standards, and specifications from Section 3H.6.4 are used for analysis and design of the DGFOSV.

The DGFOSV are designed to the applicable loads and load combinations specified in Section 3H.6.4.

The settlement information on the DGFOSV is included in Section 2.5S.4.10.

The forces and moments at critical locations in the DGFOSV along with the provided longitudinal and transverse reinforcement are included in Table 3H.6-11 in conjunction with Figures 3H.6-140 through 3H.6-208.

The calculated factors of safety against sliding, overturning, and flotation for the DGFOSV are included in Table 3H.6-12.

The tornado missile impact evaluation results for the DGFOSV are included in Table 3H.6-13.

Lateral soil pressures used in design are shown in Figures 3H.6-241 through 3H.6-244.

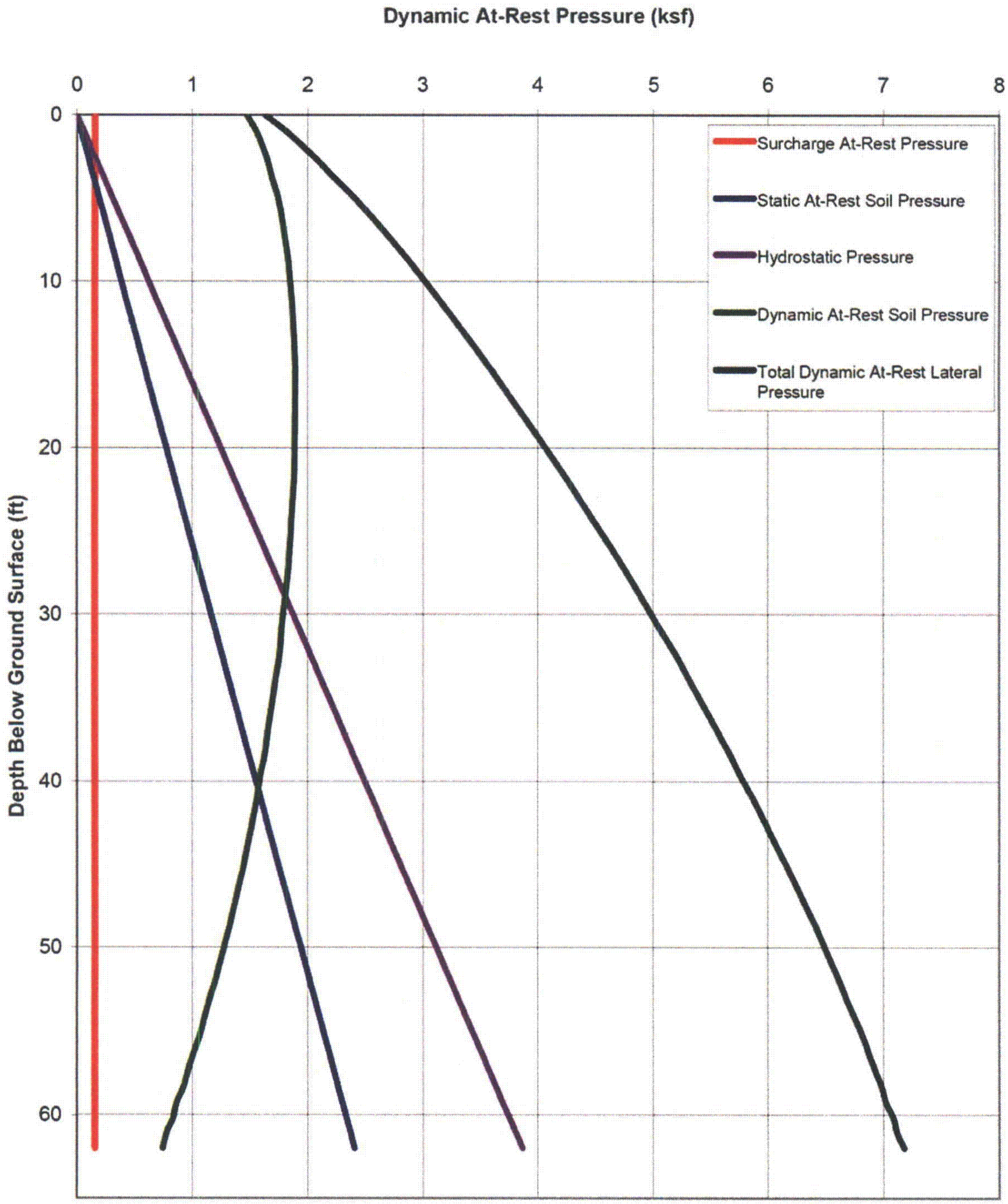


Figure 3H.6-41: Dynamic At-Rest Lateral Earth Pressure (Excluding SSI and SSSI Seismic Soil Pressures) on the East, West, and North Walls of Pump House

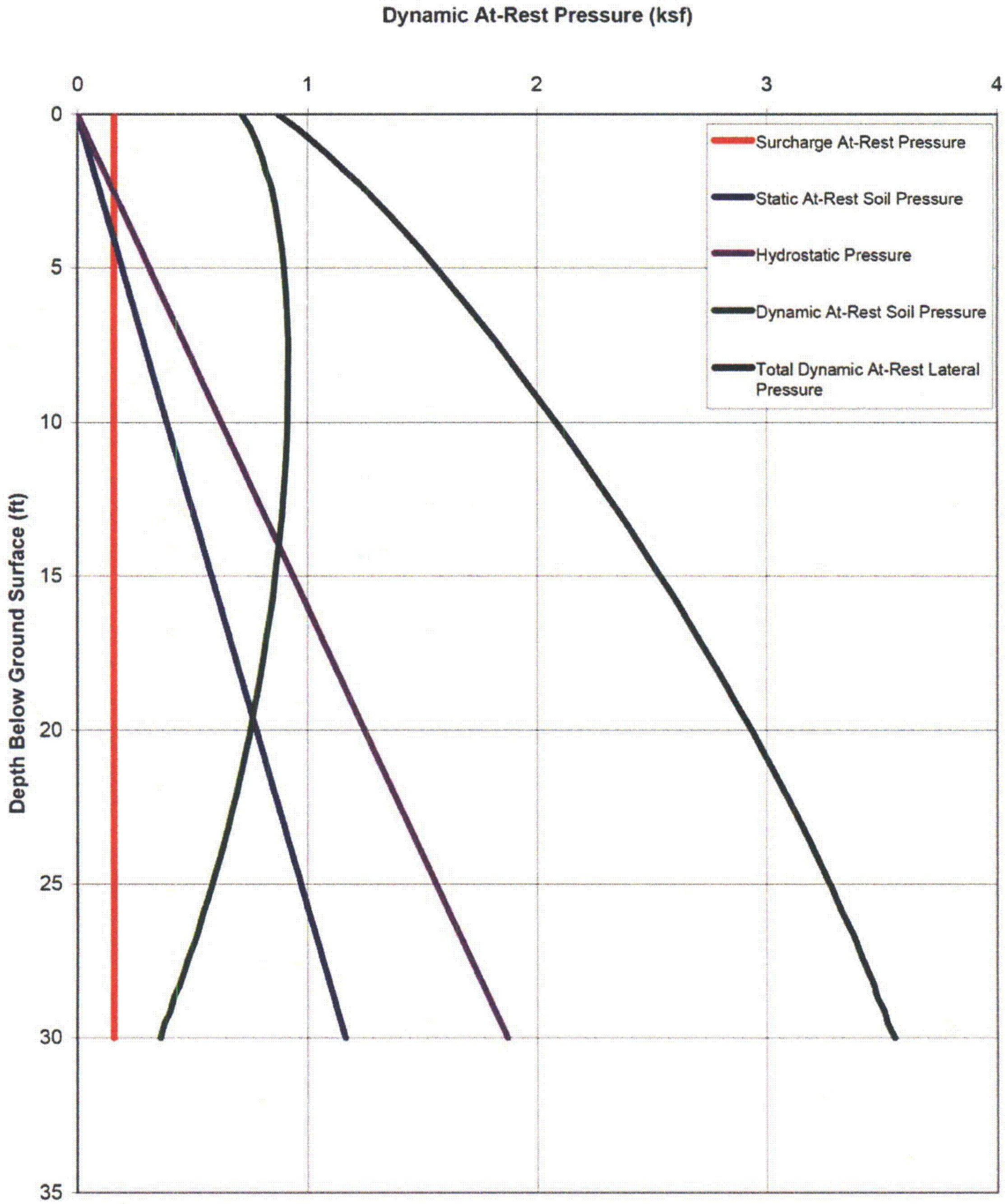


Figure 3H.6-42: Dynamic At-Rest Lateral Earth Pressure (Excluding SSI and SSSI Seismic Soil Pressures) on the UHS Basin Walls

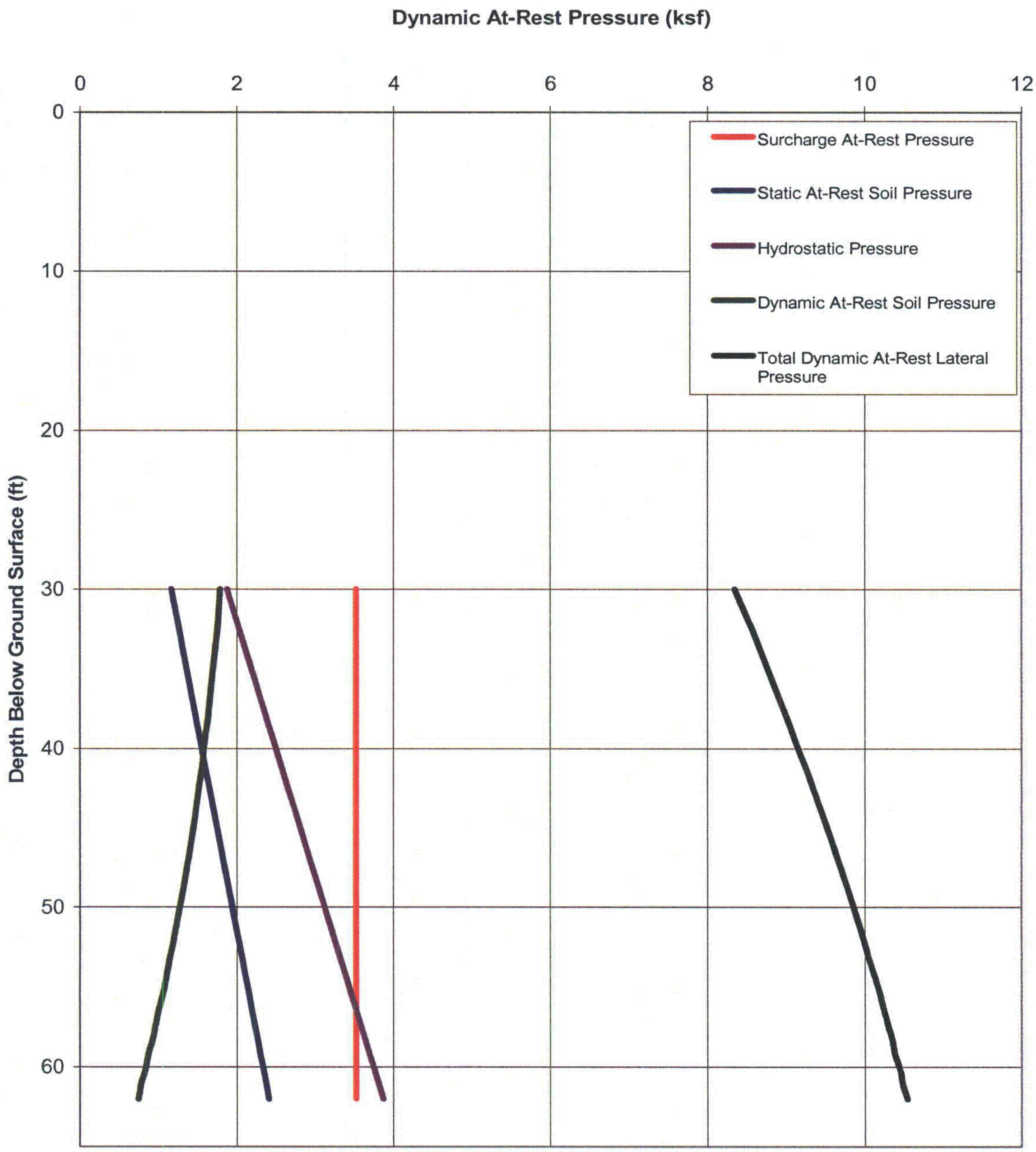


Figure 3H.6-43: Dynamic At-Rest Lateral Earth Pressure (Excluding SSI and SSSI Seismic Soil Pressures) on the South Wall of RSW Pump House

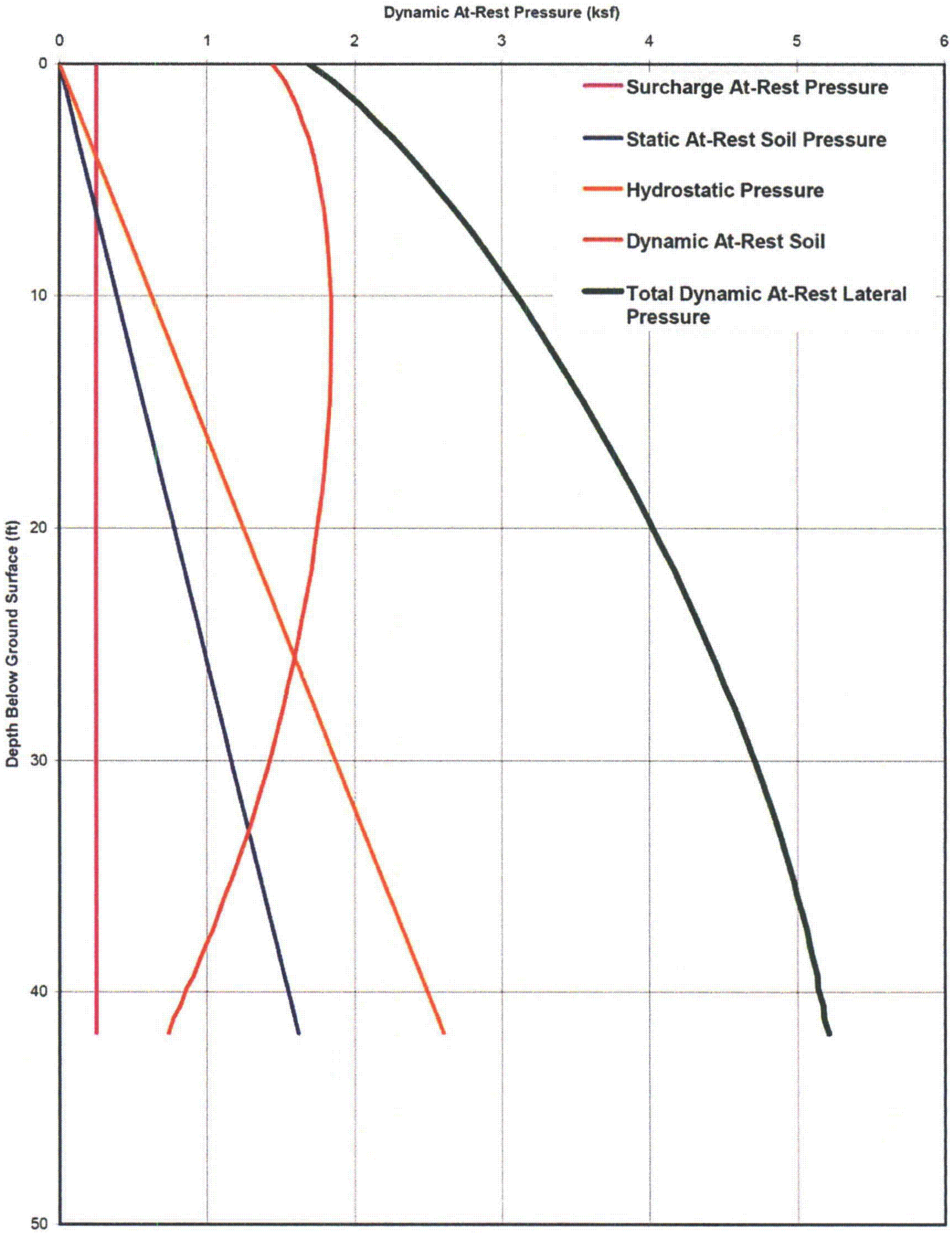


Figure 3H.6-44: Dynamic At-Rest Lateral Earth Pressure Diagrams (Excluding SSI and SSSI Seismic Soil Pressures) for Typical Section of RSW Tunnel

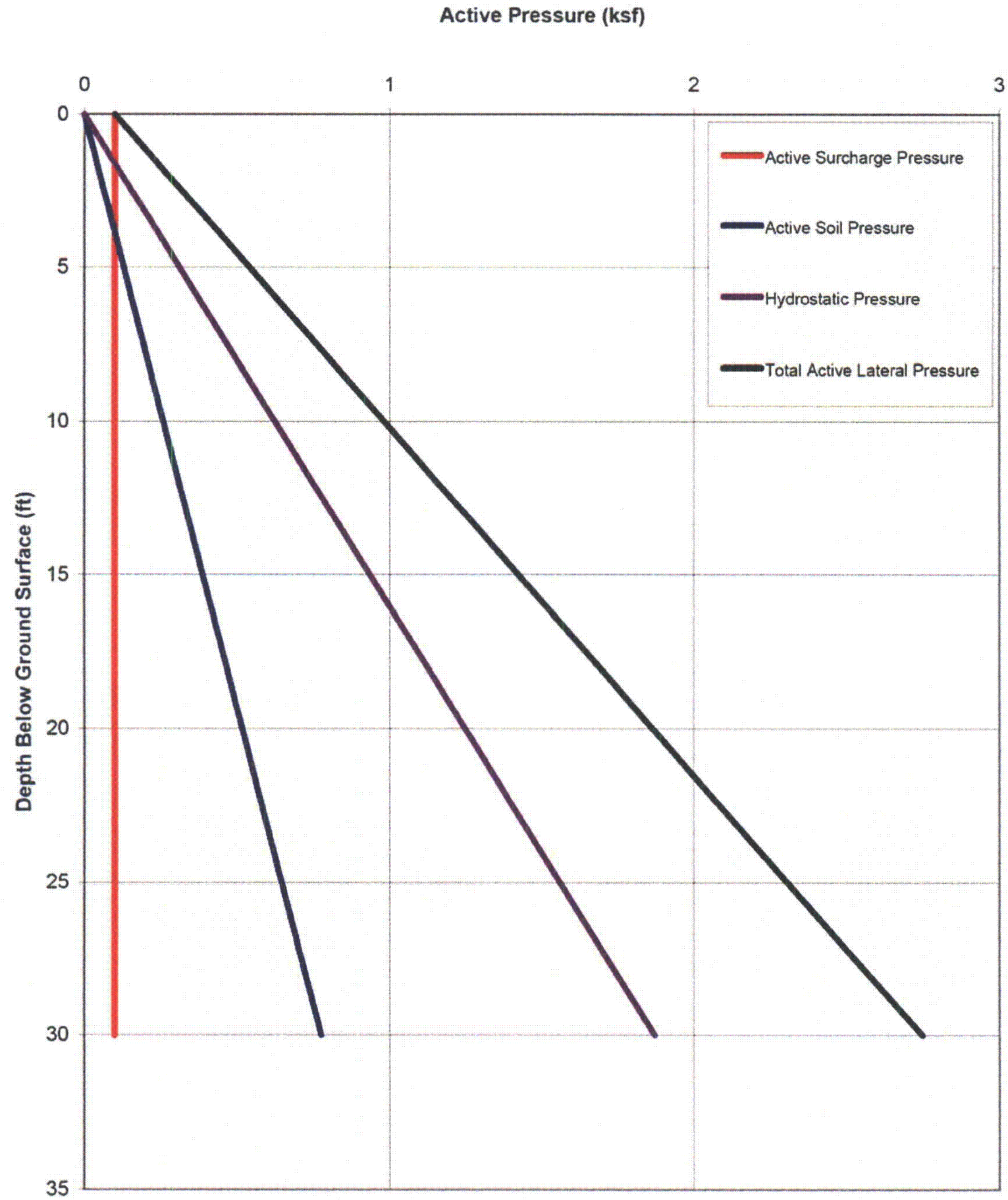


Figure 3H.6-232: Active Lateral Earth Pressure on the UHS Basin Walls

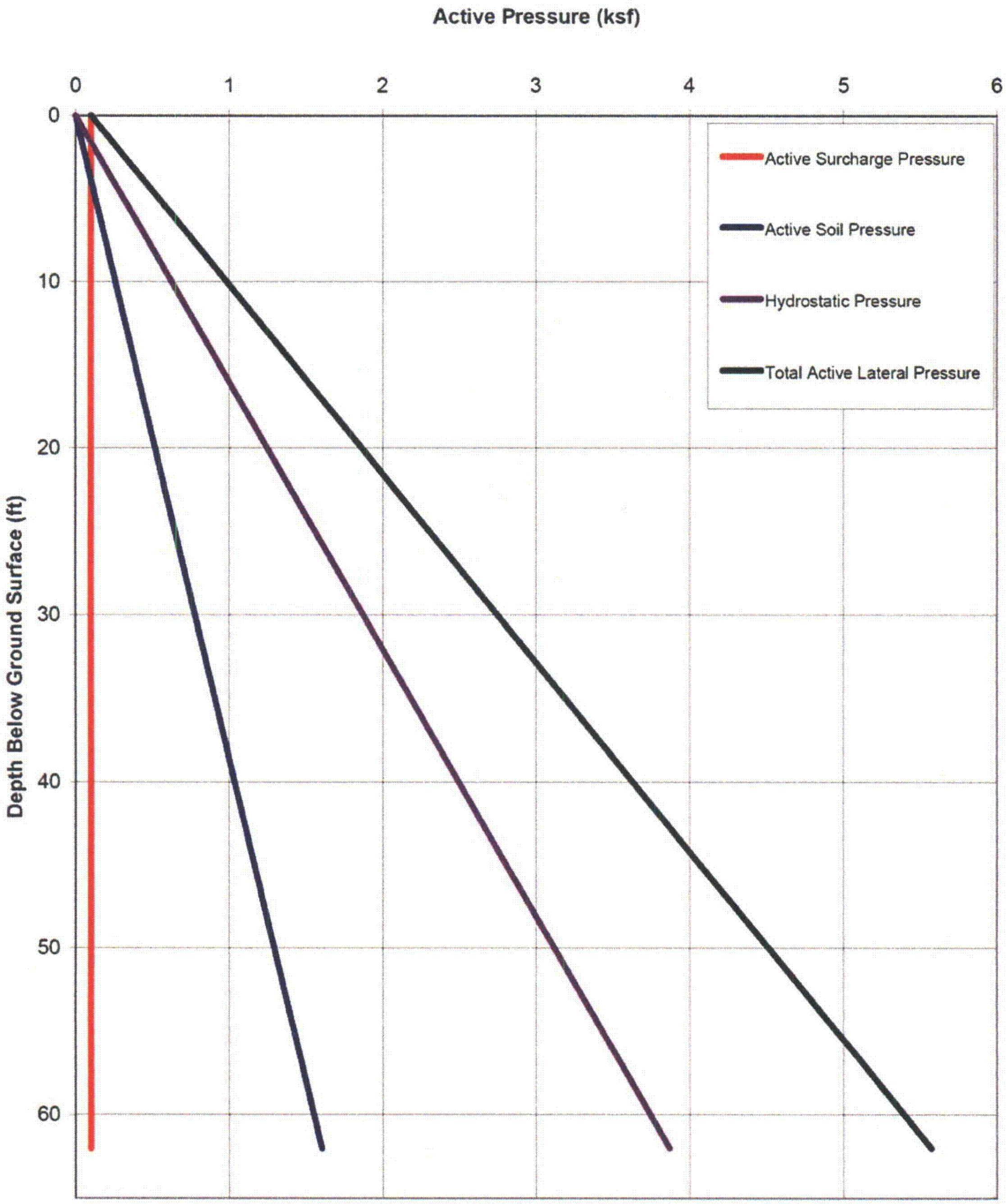


Figure 3H.6-233: Active Lateral Earth Pressure on the North, East and West Walls of the RSW Pump House

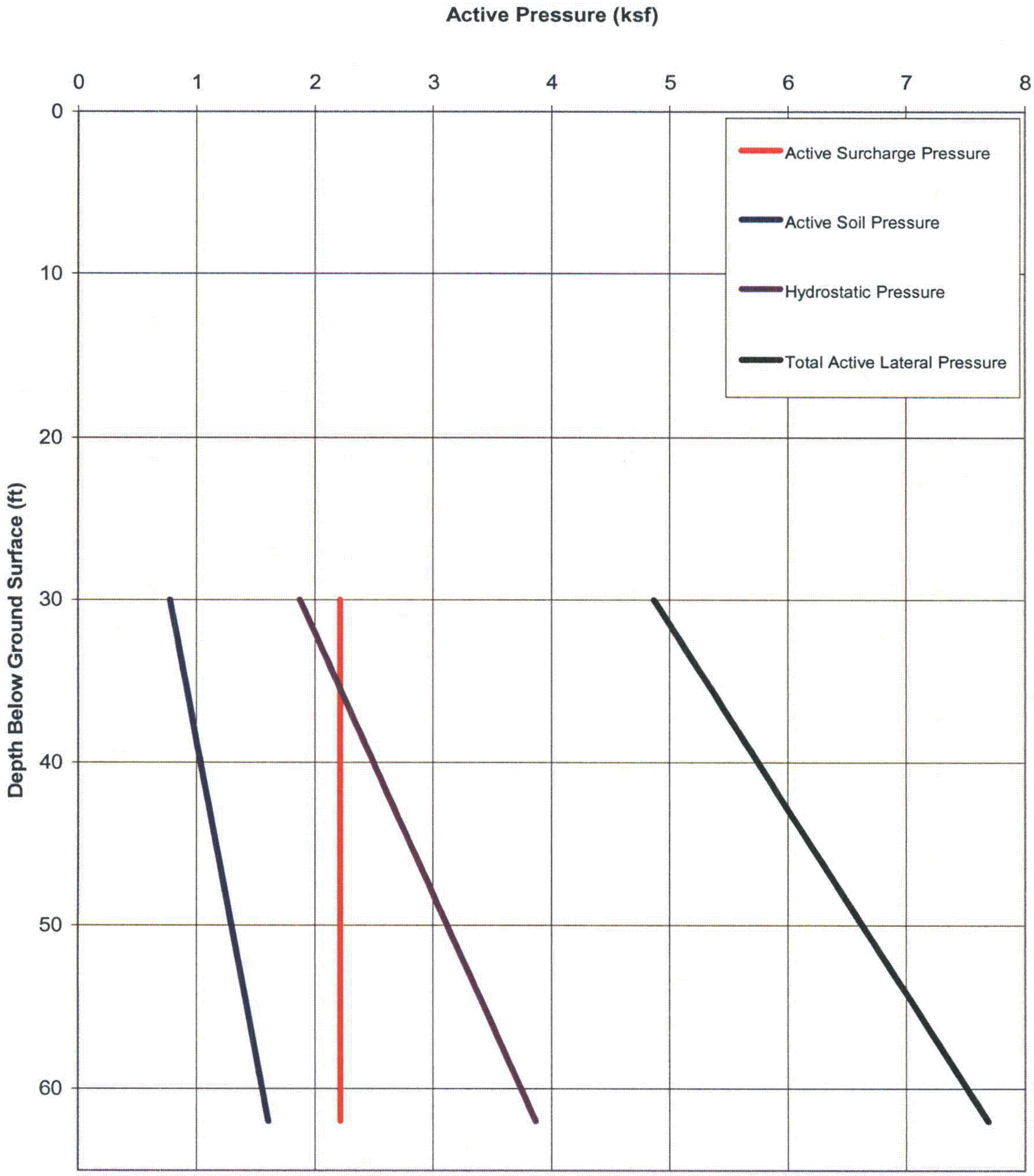


Figure 3H.6-234: Active Lateral Earth Pressure on the South Wall of the RSW Pump House

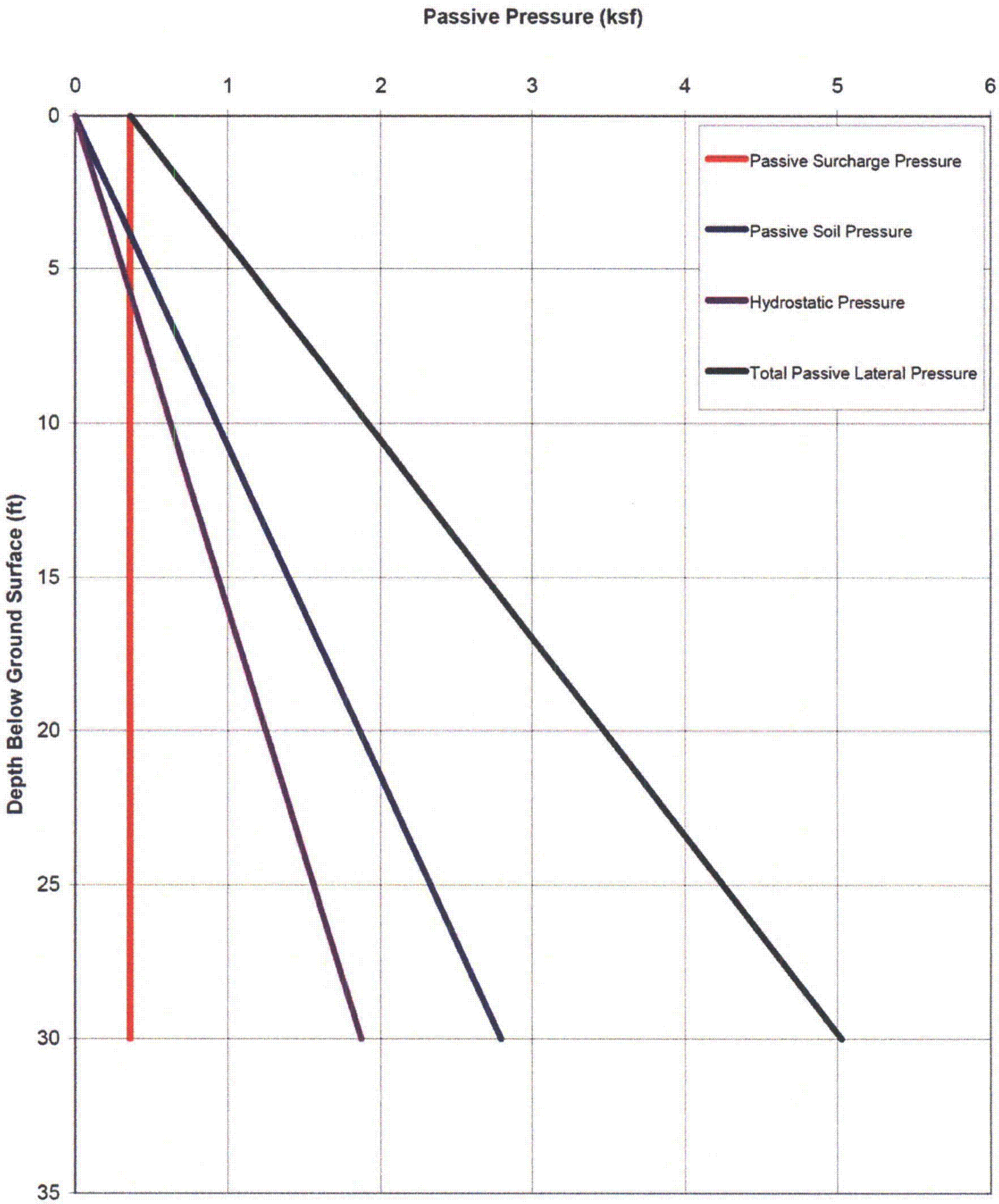


Figure 3H.6-235: Passive Lateral Earth Pressure on the UHS Basin Walls

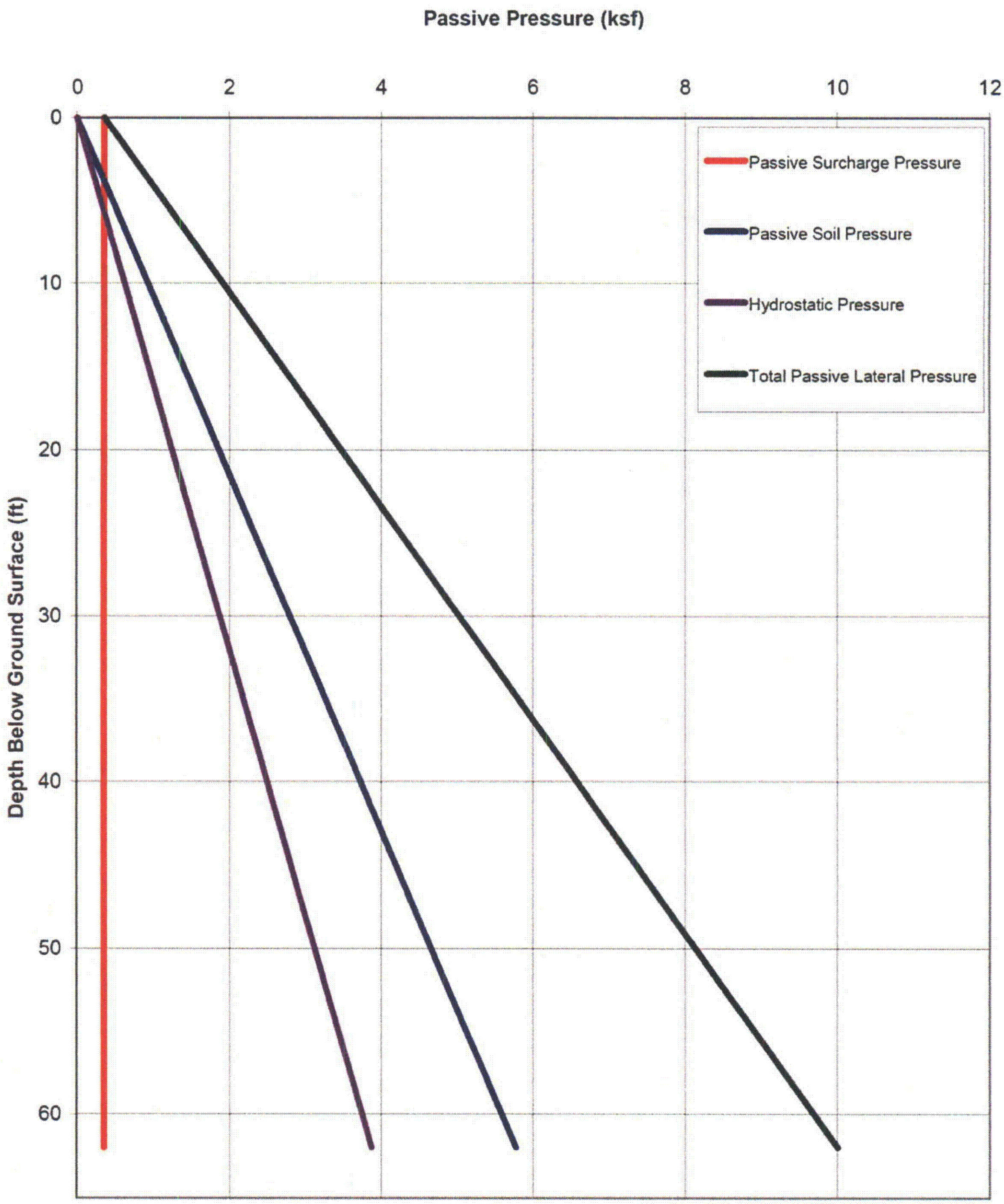


Figure 3H.6-236: Passive Lateral Earth Pressure on the North, East and West Walls of the RSW Pump House

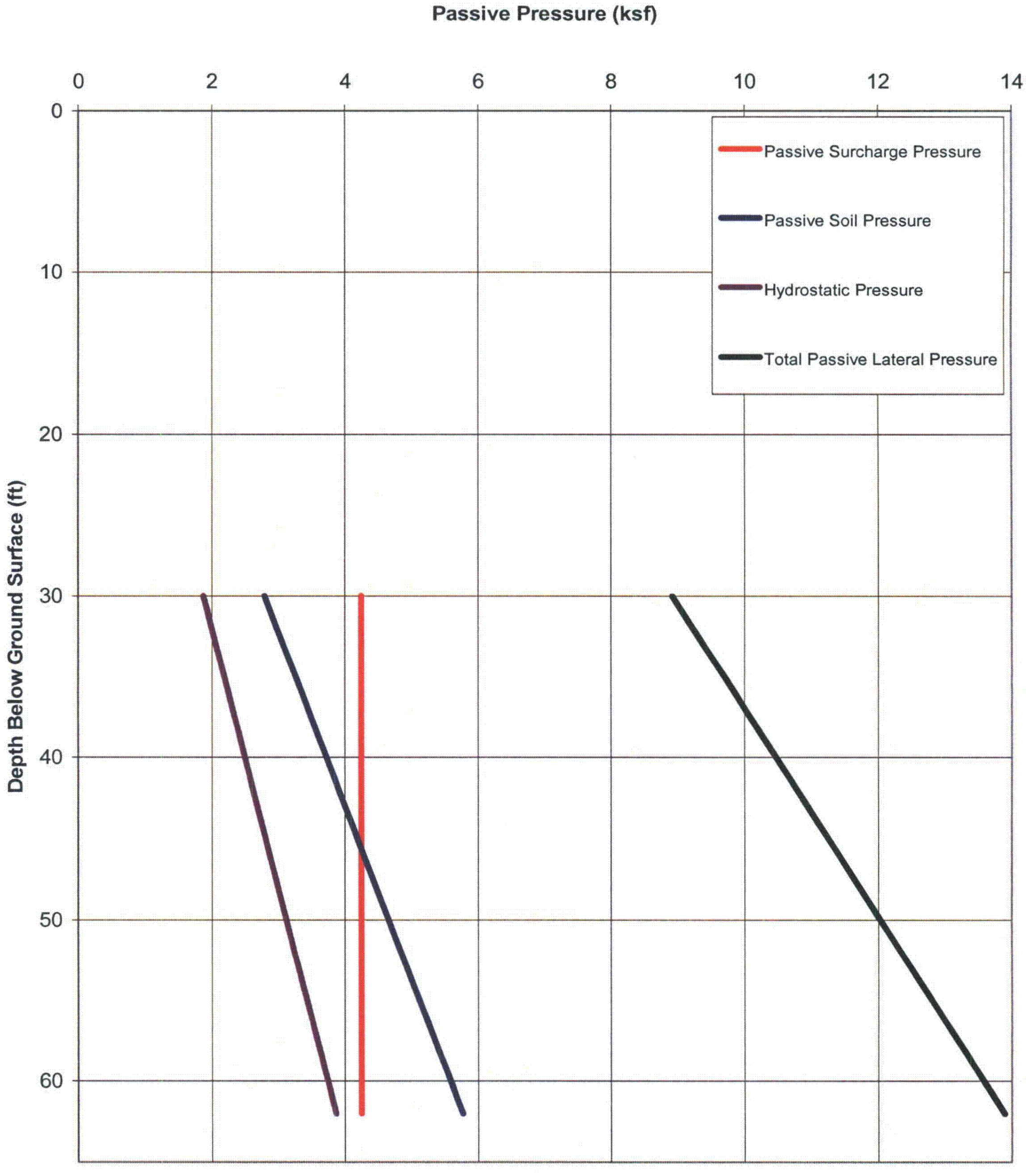


Figure 3H.6-237: Passive Lateral Earth Pressure on the South Wall of the RSW Pump House

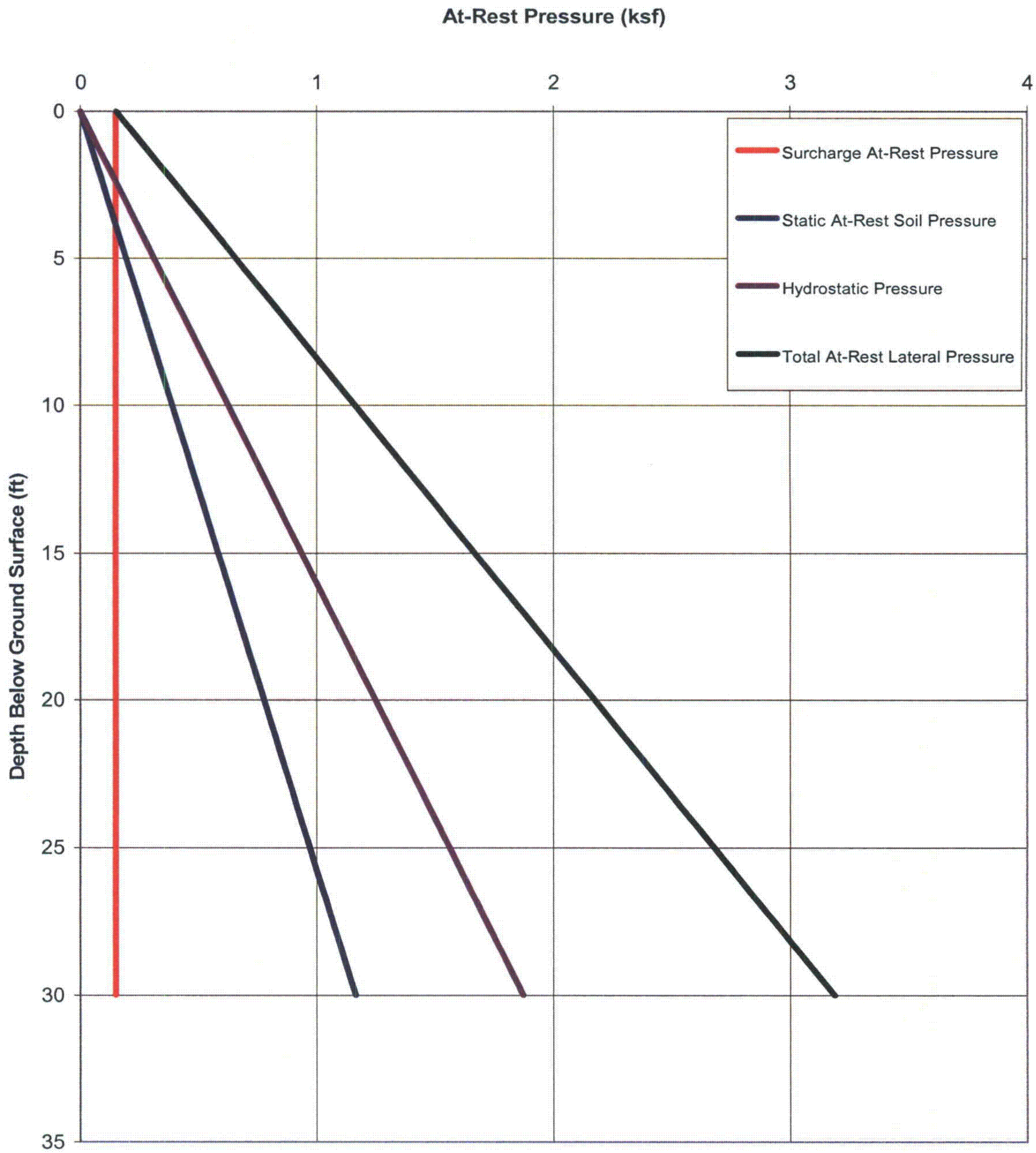


Figure 3H.6-238: At-Rest Lateral Earth Pressure on the UHS Basin Walls

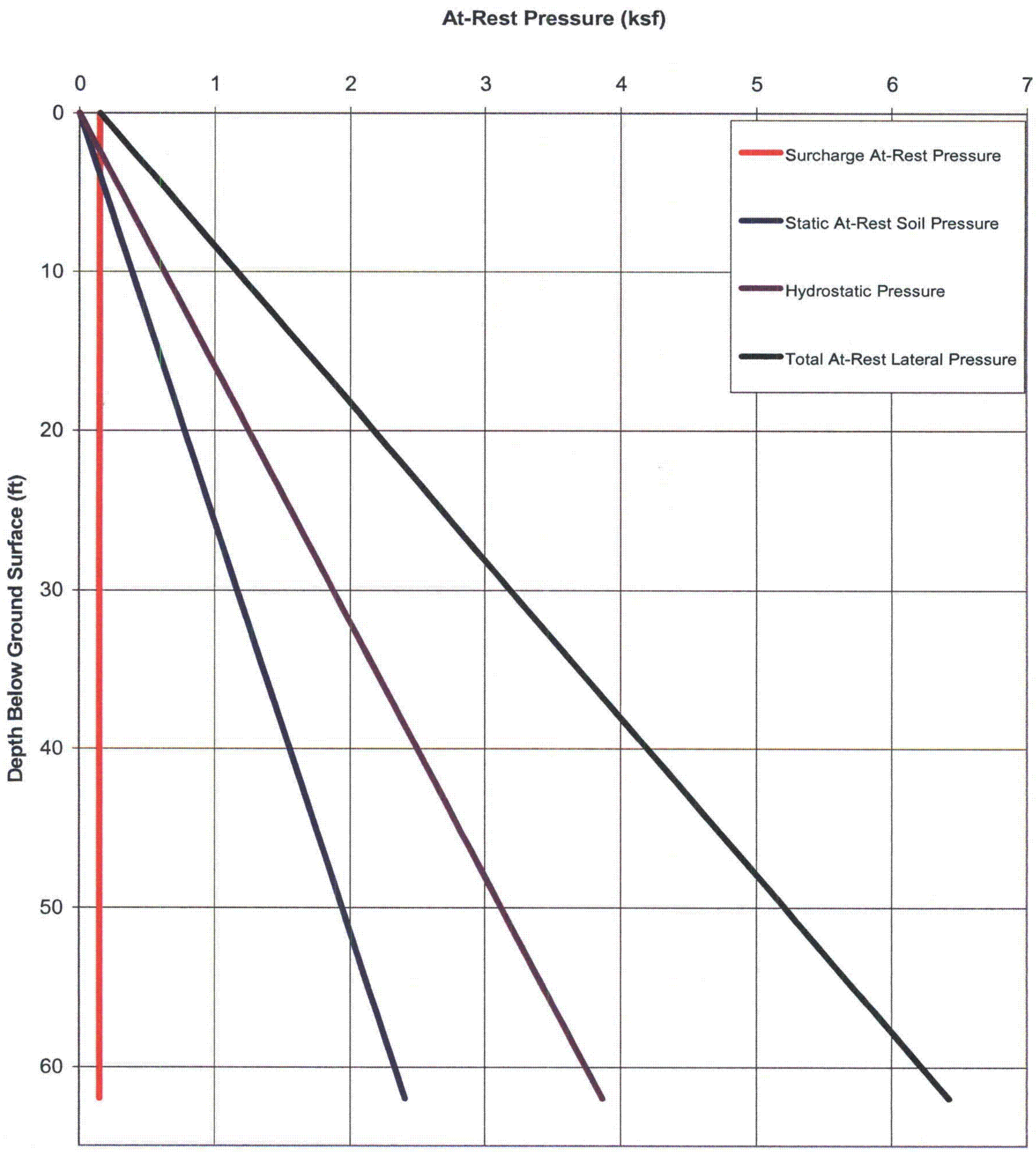


Figure 3H.6-239: At-Rest Lateral Earth Pressure on the North, East and West Walls of the RSW Pump House

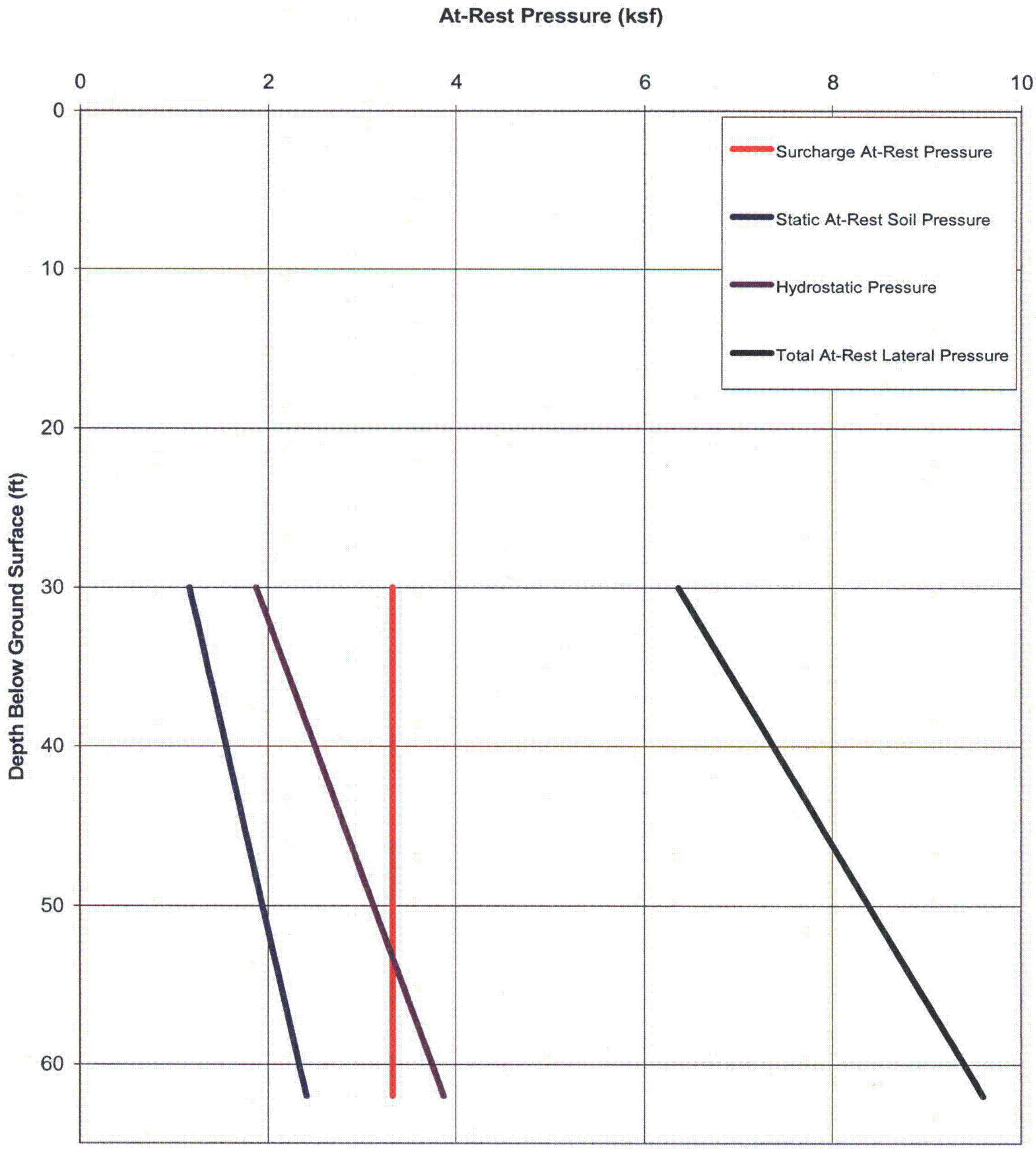


Figure 3H.6-240: At-Rest Lateral Earth Pressure on the South Wall of the RSW Pump House

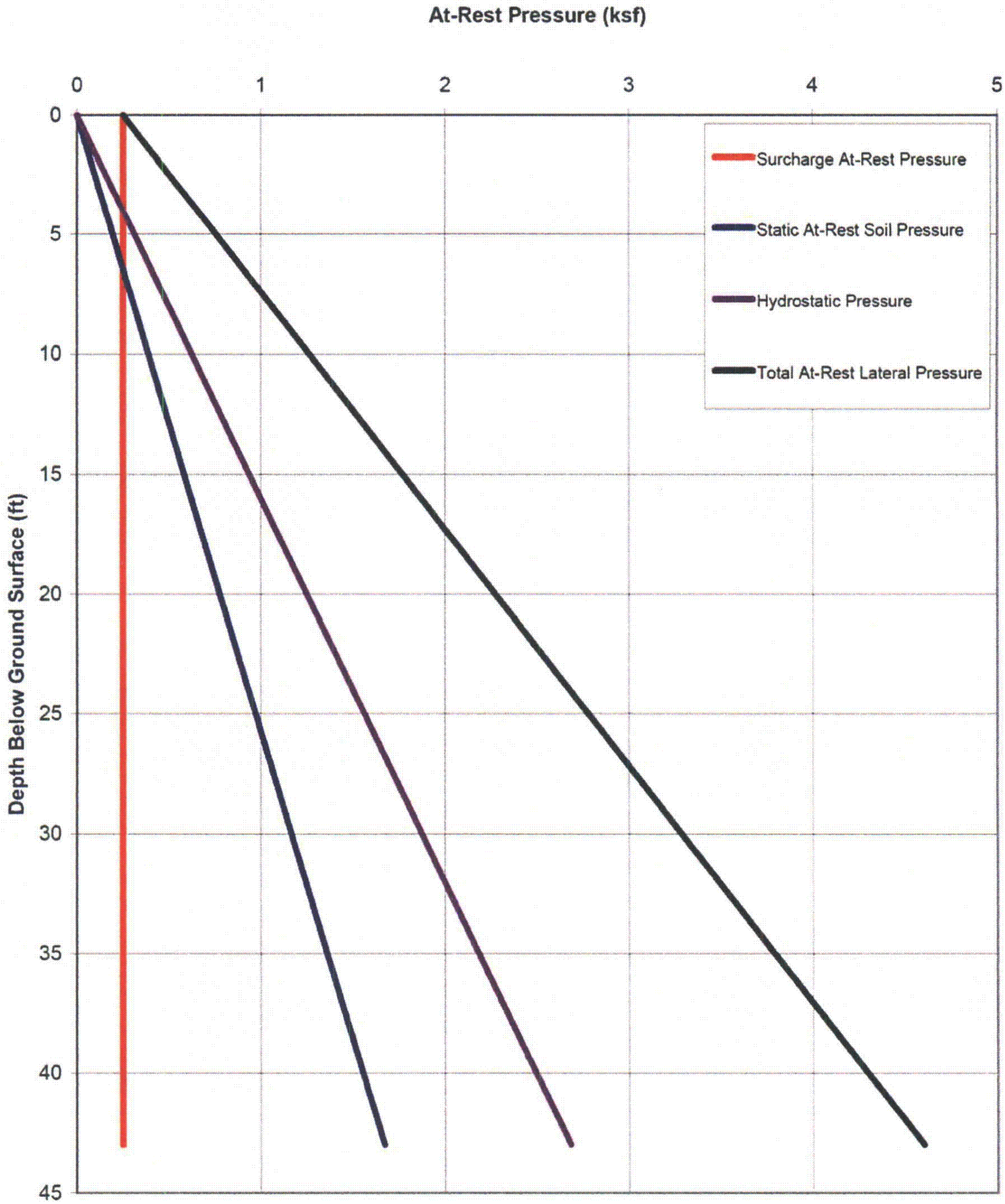


Figure 3H.6-241: At-Rest Lateral Earth Pressure on the Diesel Generator Fuel Oil Storage Vault Walls

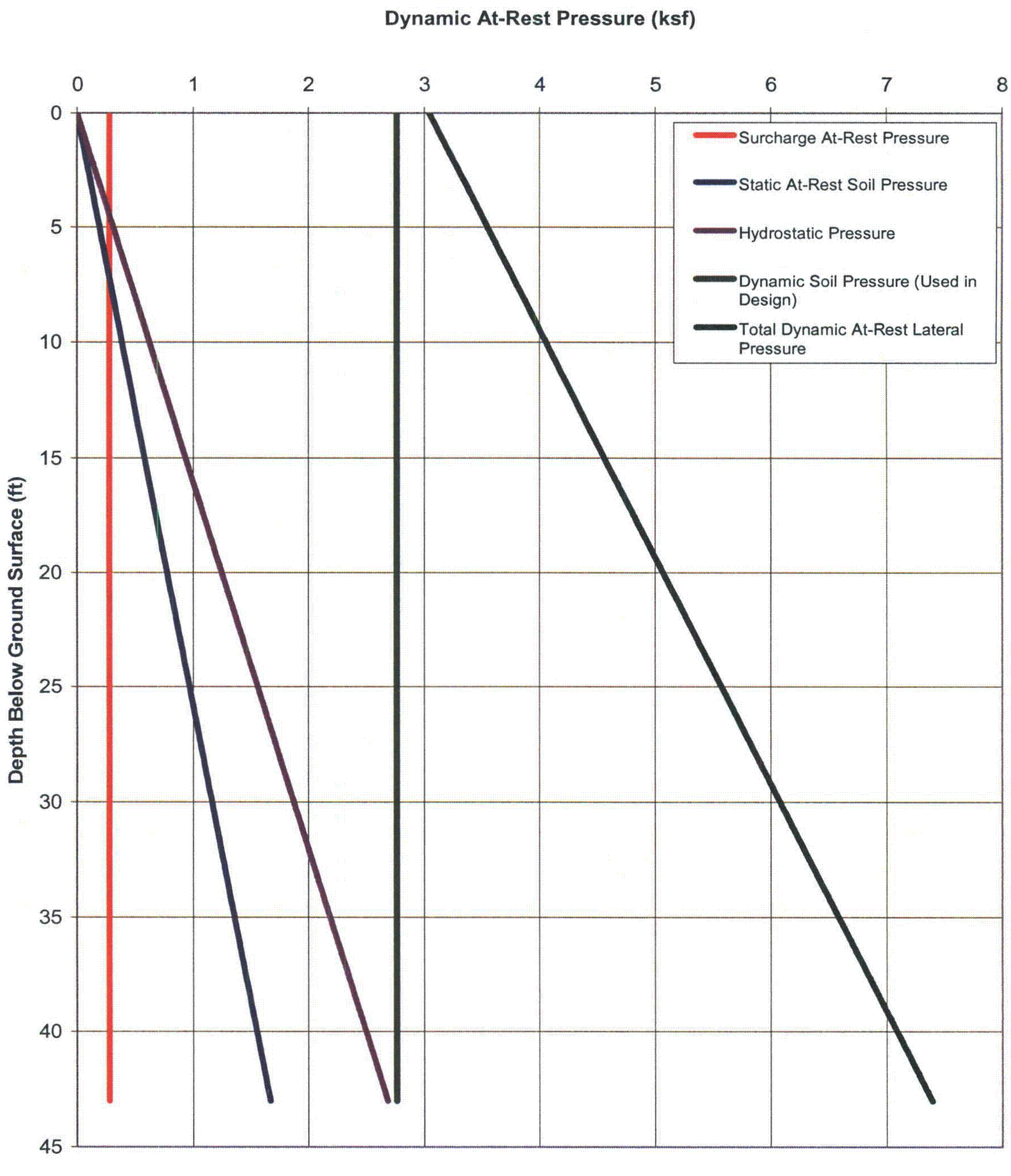


Figure 3H.6-242: Dynamic At-Rest Lateral Earth Pressure on the Diesel Generator Fuel Oil Storage Vault Walls

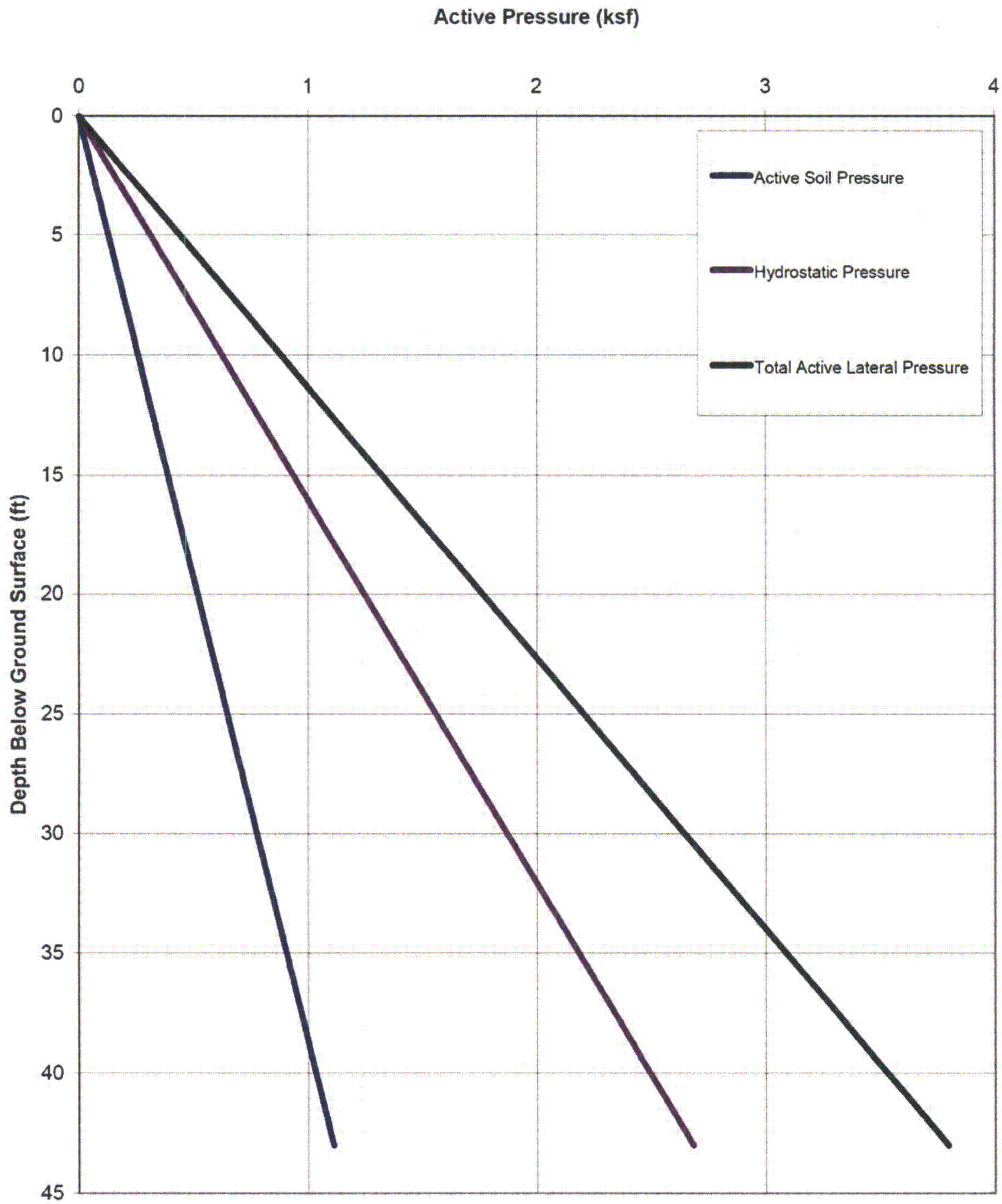


Figure 3H.6-243: Active Lateral Earth Pressure on the Diesel Generator Fuel Oil Storage Vault Walls

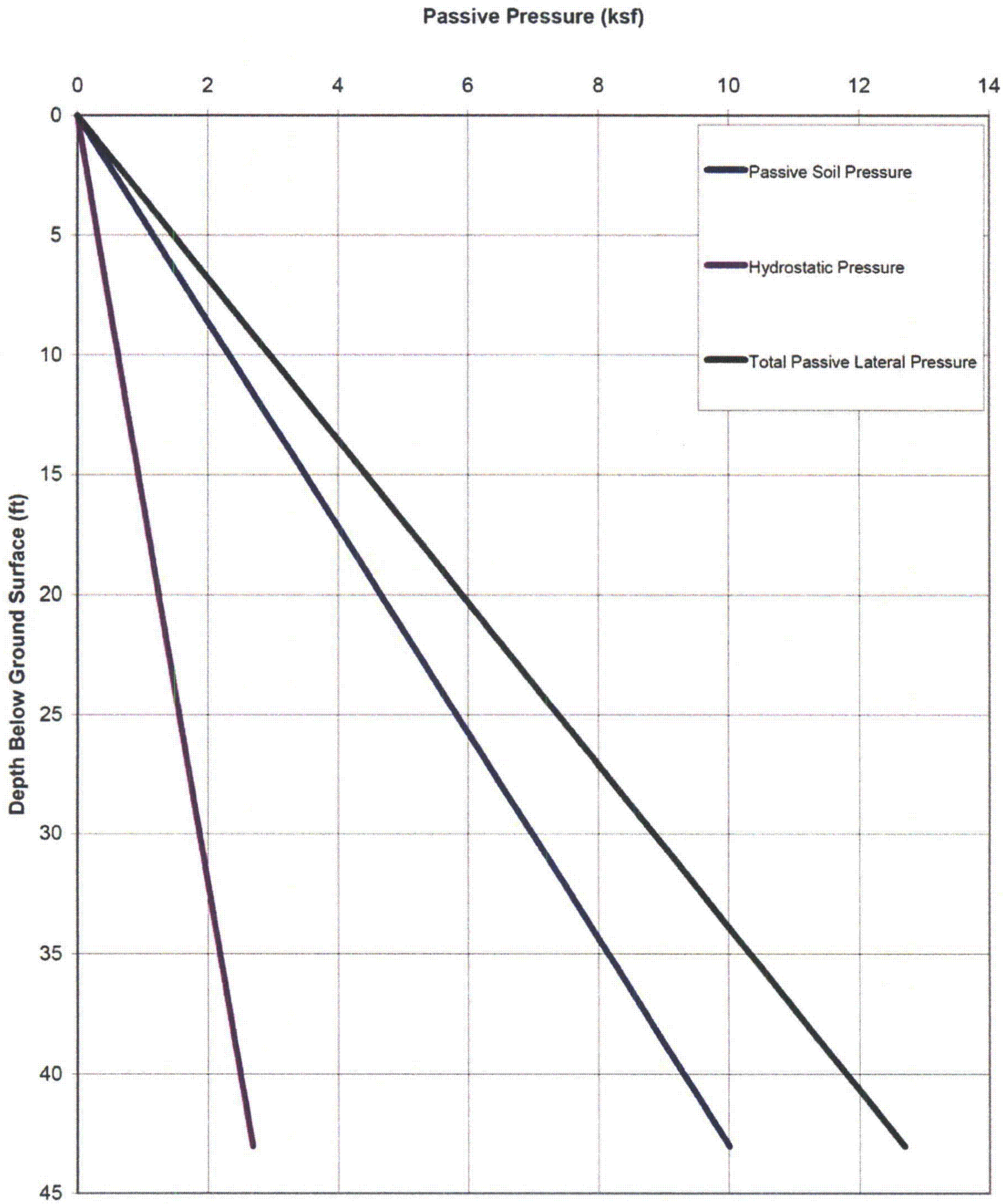


Figure 3H.6-244: Passive Lateral Earth Pressure on the Diesel Generator Fuel Oil Storage Vault Walls

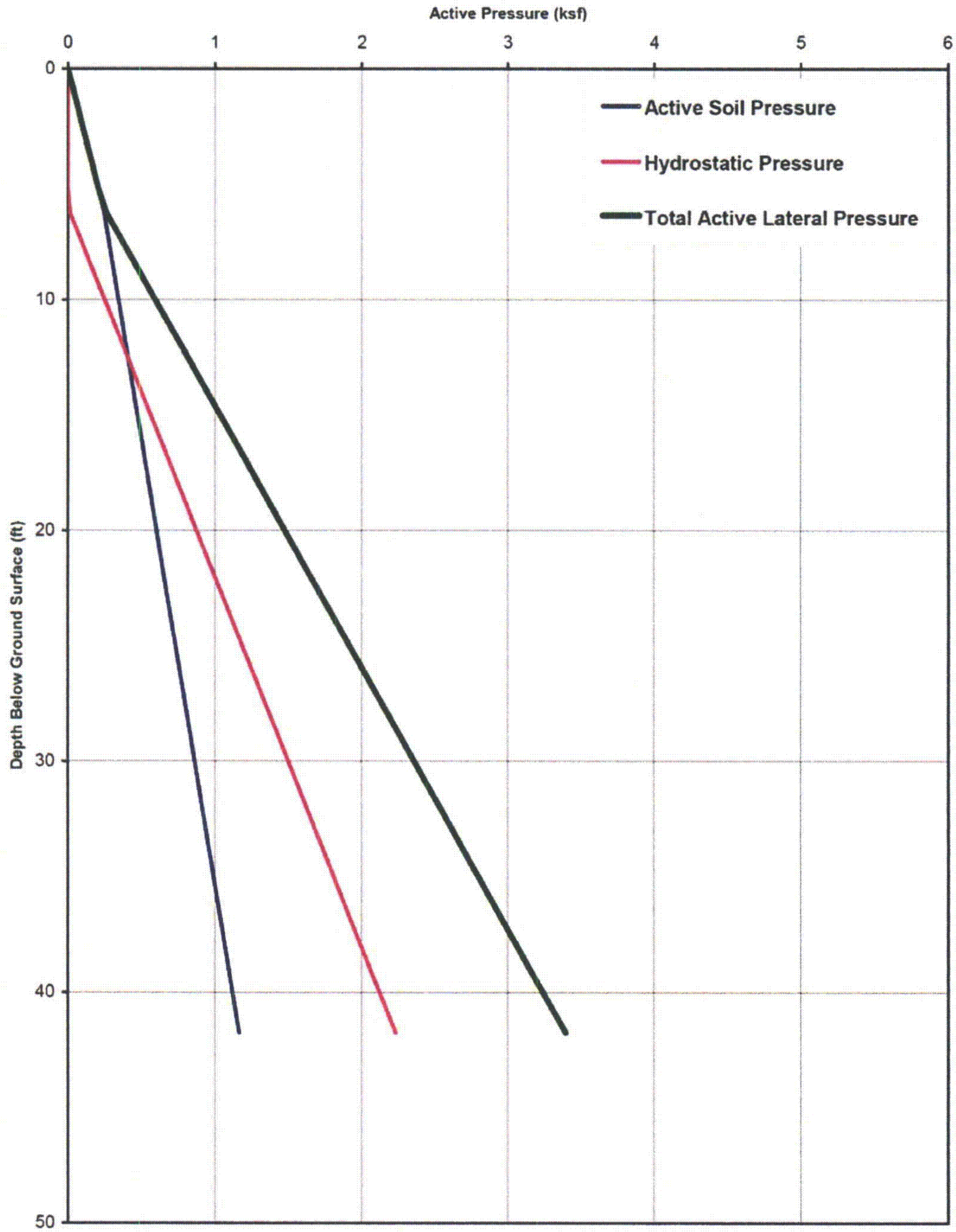


Figure 3H.6-245: Active Lateral Earth Pressure Diagrams for Typical Section of RSW Tunnel

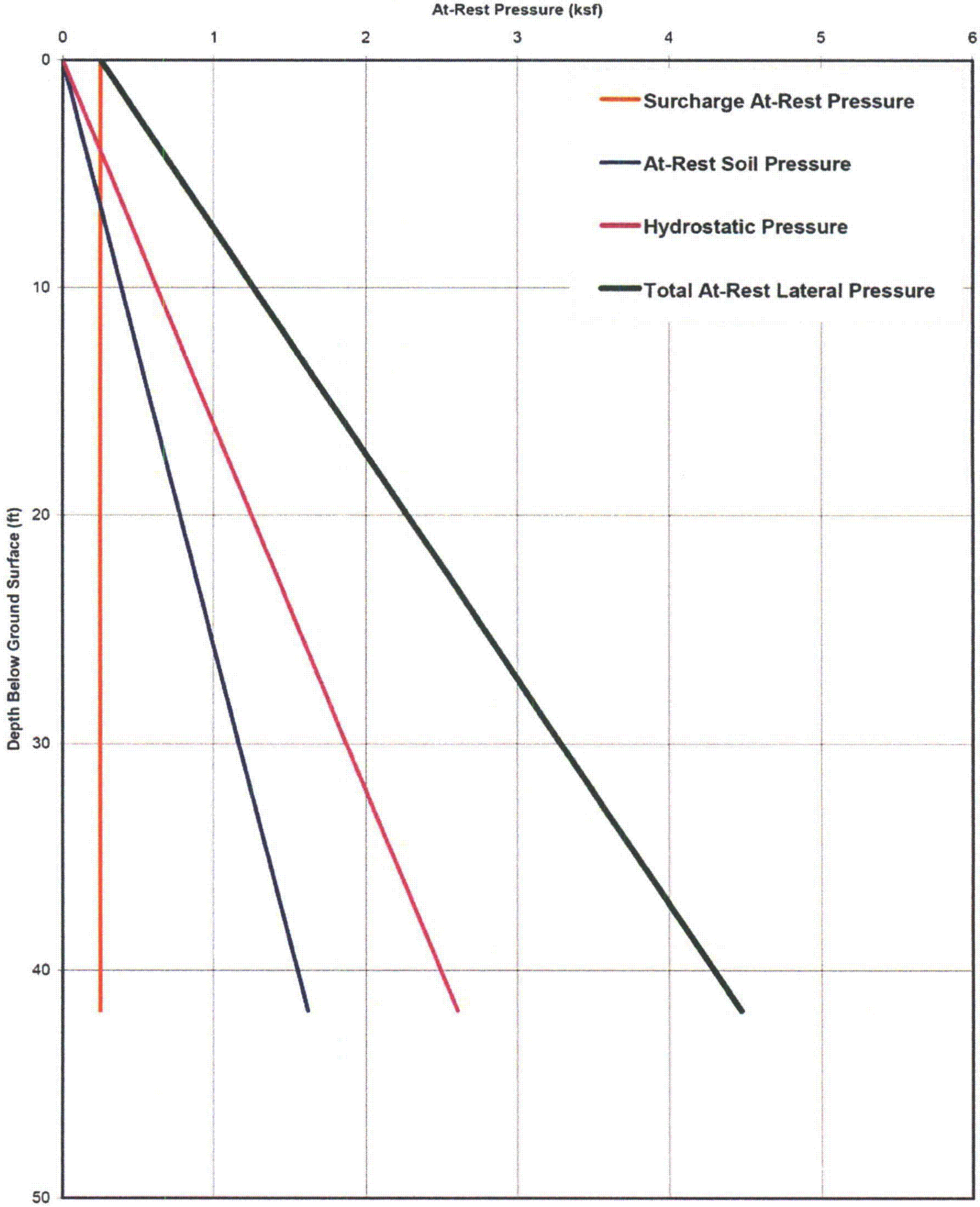


Figure 3H.6-246: At-Rest Lateral Earth Pressure Diagrams for Typical Section of RSW Tunnel

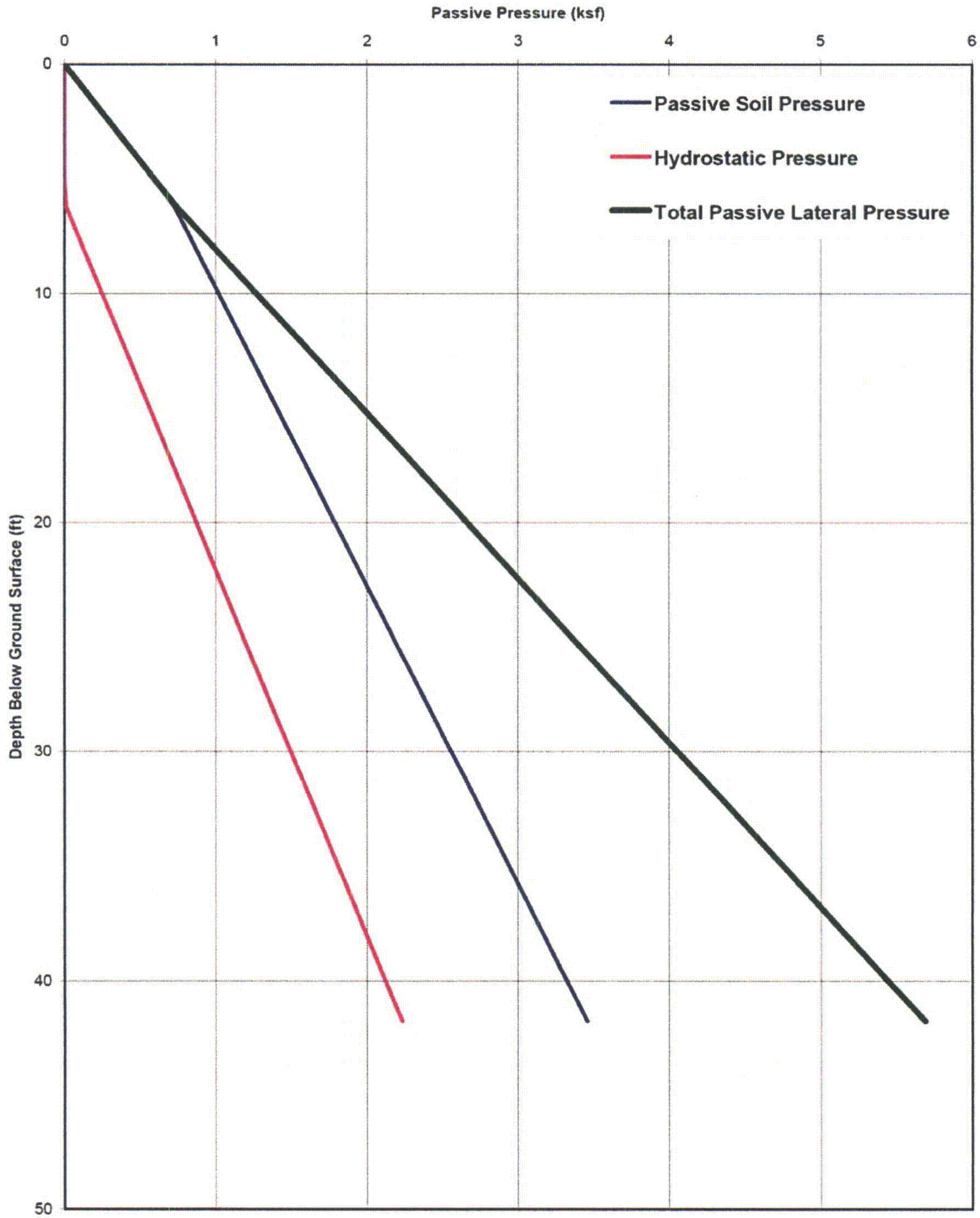


Figure 3H.6-247: Passive Lateral Earth Pressure Diagrams for Typical Section of RSW Tunnel

RAI 03.08.04-17, Revision 1
Enclosure 2

COLA Part 2, Tier 2, Section 3H.7

3H.7.4.3.1.3 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

Lateral soil pressure values are shown in Figures 3H.7-2 through 3H.7-8 and Figures 3H.7-33 through 3H.7-35.

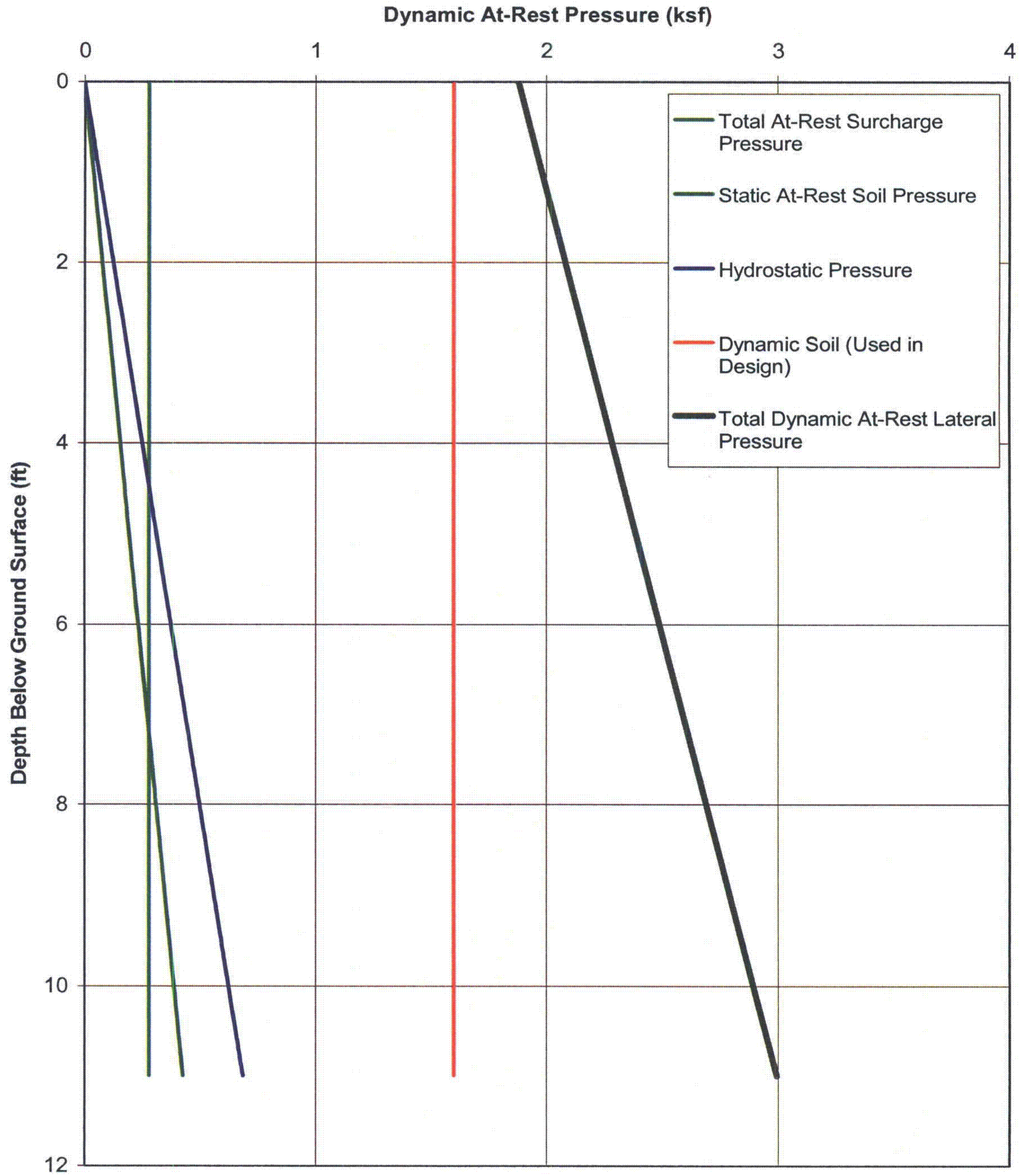


Figure 3H.7-2: Dynamic At-Rest Lateral Earth Pressure ~~(psf) on the Walls of the Fuel Oil Tunnel~~ DGFOT Walls

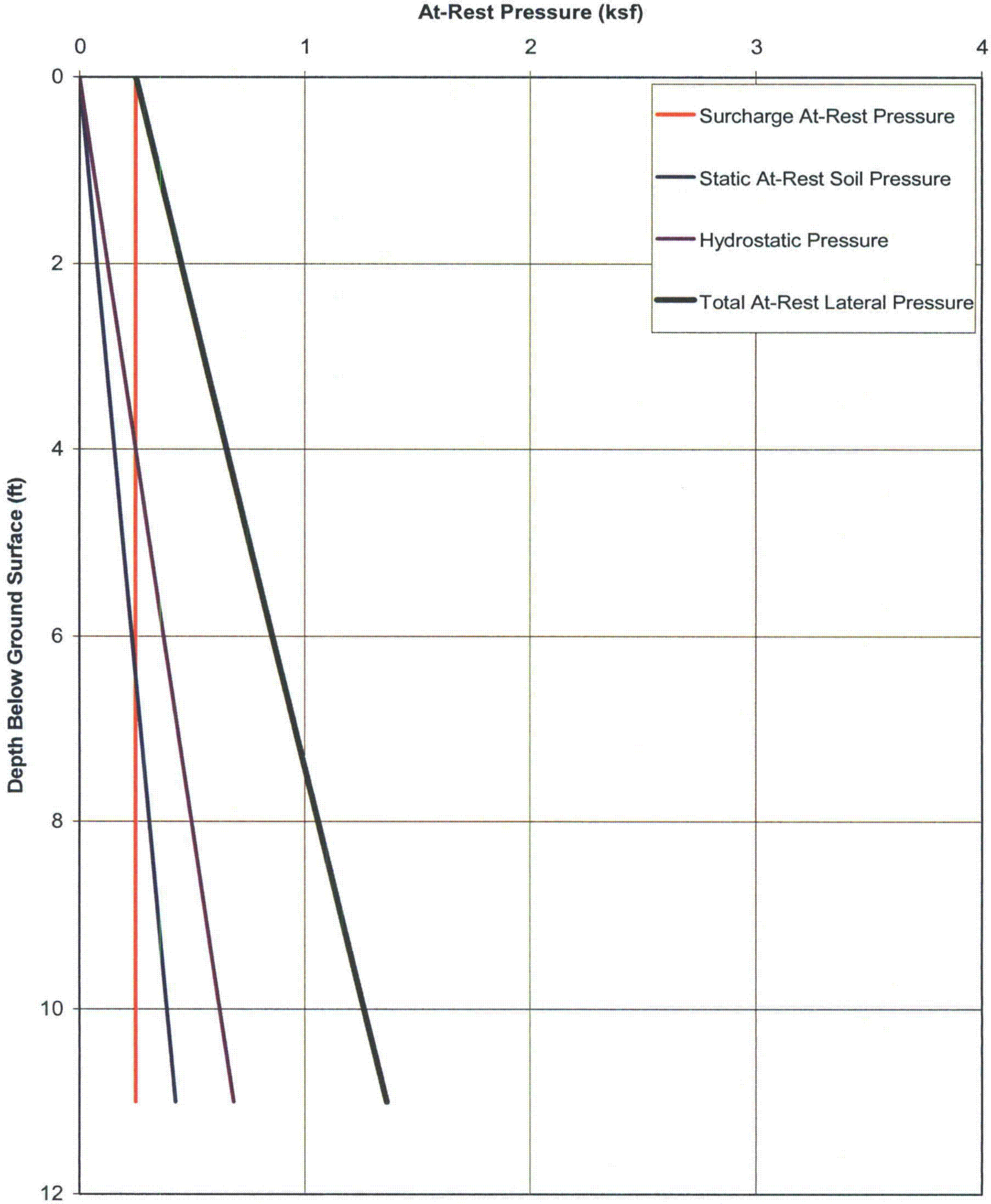


Figure 3H.7-33: At-Rest Lateral Earth Pressure on the DGFOT Walls

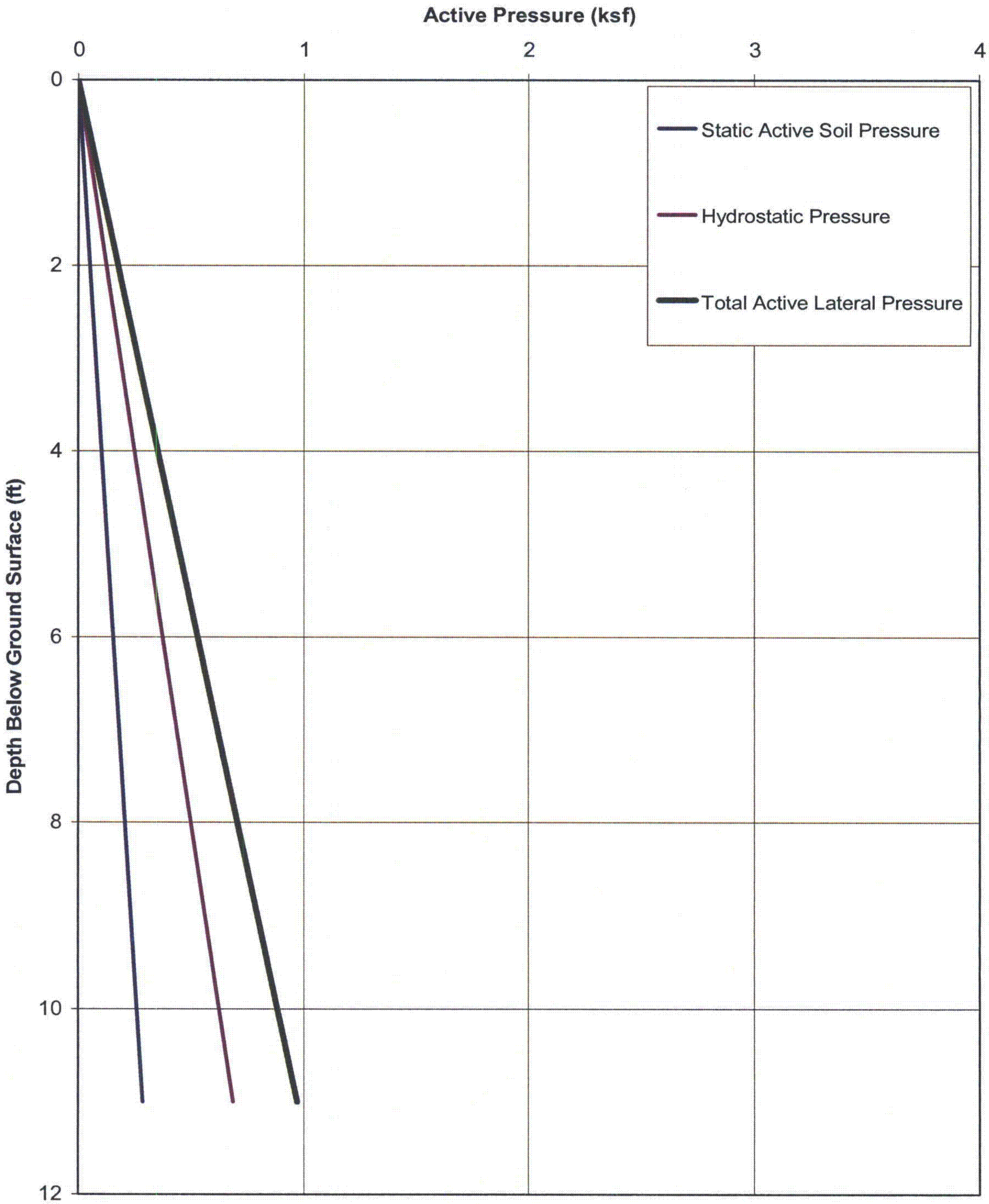


Figure 3H.7-34: Active Lateral Earth Pressure on the DGFOT Walls

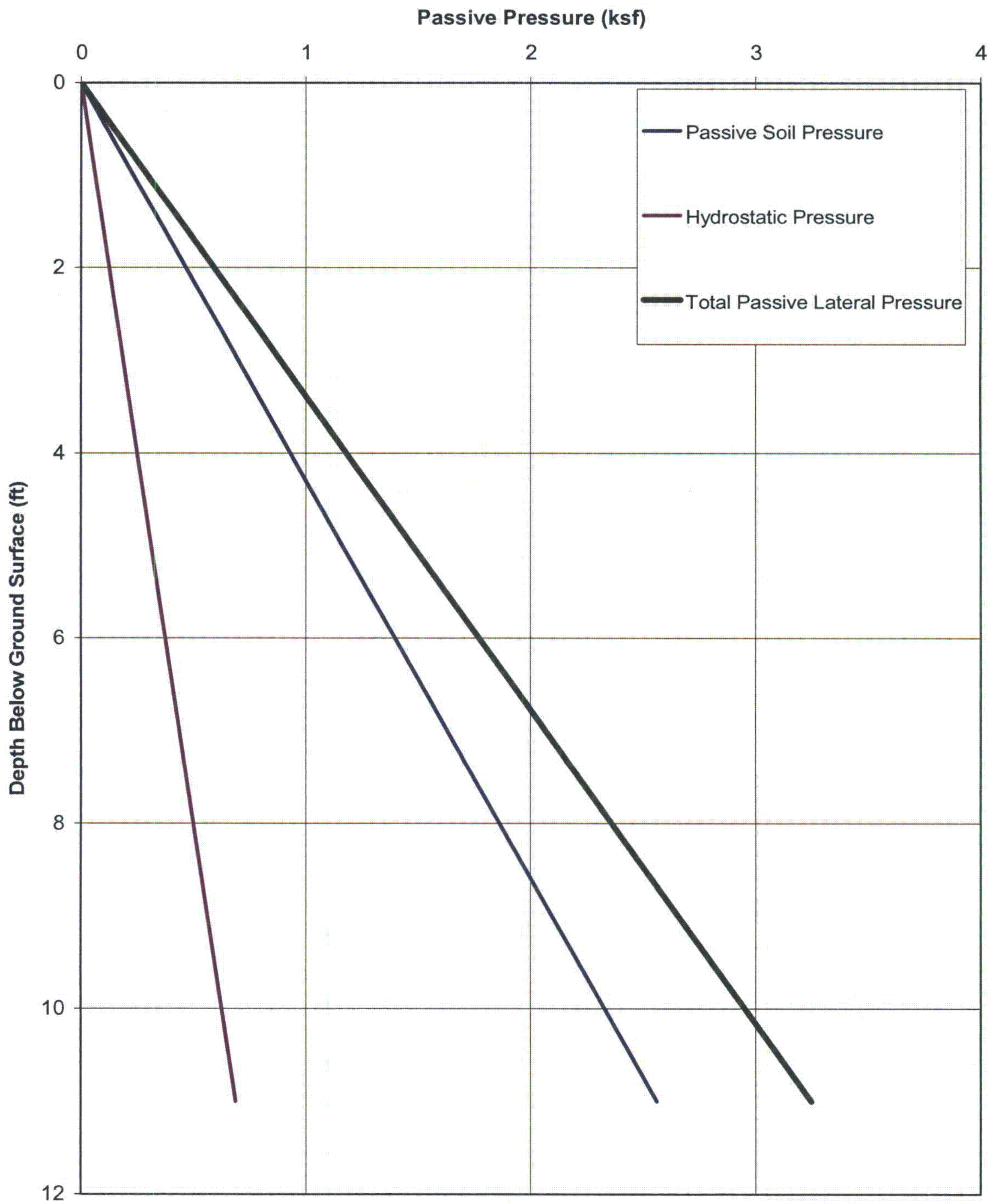


Figure 3H.7-35: Passive Lateral Earth Pressure on the DGFOT Walls

RAI 03.08.04-28, Revision 1

QUESTION:

Follow-up to Question 03.08.04-19

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

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

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

3. In response to the staff's question regarding the coefficient of friction for the waterproofing membrane, the applicant has proposed an ITAAC that states that "Type testing will be performed to determine the minimum coefficient of friction of the type of material used in the mudmat-waterproofing-mudmat interface beneath the basemats of the Category I structures." It is not clear from the description if the thickness of the specimen tested will be the same as that used for the membrane. The applicant is requested to clarify this and revise the ITAAC. Also, the acceptance criteria for the ITAAC states that "A report exists and documents that the waterproof system (mudmat-waterproofing-mudmat) has a coefficient of friction to support the analysis against sliding." The applicant stated in

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

4. The applicant stated in its response (Table RAI 03.08.04-19a) that the coefficient of friction provided at the interface of the bottom of the gravel layer and soil to be the smaller of 0.6 and shear capacity of the soil. Elsewhere in the response, the applicant stated that the soil capacity exceeds the value of 0.47 needed for maintaining minimum factor of safety against sliding of RB and CB. The applicant is requested to clarify the minimum coefficient of friction available at the bottom of gravel and soil interface based on site-specific soil properties and explain how it is determined.

REVISED RESPONSE:

The original response to RAI 03.08.04-28 was submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010. This revision is based on the discussions held in the meeting with NRC on February 2nd and 3rd, 2011. The revisions are indicated by revision bars in the margin.

1. The waterproofing membrane is applied in the structural concrete fill and is in the load path between the basemat and soil, which requires, in addition to water retaining capacity, a sufficient coefficient of friction be maintained at the interface over the life of the plant. Therefore, the waterproofing membrane will be tested under conditions that simulate actual exposure.

   The waterproofing membrane will be tested per ASTM C267 (Standard Test Methods for Chemical Resistance of Mortars, Grouts, and Monolithic Surfacings and Polymer Concretes) for its resistance to the concrete mix chemistry, the actual backfill material chemistry, and groundwater chemistry found on site. Additional testing of the waterproofing membrane's ability to resist the chemical reagents as specified through accelerated aging will be done per ASTM G114 (Standard Practices for Evaluating the Age Resistance of Polymeric Materials Used in Oxygen Service).

   The description and thickness of the membrane material was given in Revision 1 of the response to RAI 03.08.04-19 (see STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The COLA markup for the description and thickness of the membrane material, as well as the requirement to test for resistance to the concrete mix chemistry, the actual backfill material chemistry, and groundwater chemistry found on site is included at the end of this response.

2. The membrane will be tested in accordance with ASTM D5385, "Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes", which requires that the membrane be subjected to a pressure of 100 psi. The acceptance criterion will be that the sample is able to resist the expected hydrostatic pressure. Based on a maximum water head of less than 90 ft (based on the depth of the Reactor Building foundation), the design hydrostatic pressure is less than 40 psi. Accelerated aging test results per ASTM G114 will be used to show that there is negligible change in the material properties or composition for at least the 60 year life of the plant. The margin provided by the test pressure of 100 psi (the design pressure is 40 psi) along with the results from accelerated age testing will ensure that the waterproofing will sufficiently resist the design hydrostatic pressure over its intended lifetime. This is included in the COLA markup included at the end of this response.

   Additional testing on the waterproofing membrane will be required to demonstrate the adequacy of the membrane's performance under applicable mechanical conditions, including pressures from the backfill, hydrostatic pressure, and foundation bearing. Test conditions will simulate the environment at the walls and the base level. The horizontal membrane (located in the structural concrete fill) will also be tested for its resistance to the hydrostatic pressures at the membrane location, as the basic assumption that necessitates the use of waterproofing is that cracks in the concrete fill will allow water to propagate up to the waterproofing membrane.

   Additional evaluations of the projected environmental pressures at the walls and the base level will be evaluated, and, if necessary the test pressures of ASTM D5385 will be adjusted accordingly.

3. The thickness of the membrane to be tested will be the same as the actual nominal thickness used for the membrane. The ITAAC in COLA Part 9, Table 3.0-13 is revised to state this as shown at the end of this response. The acceptance criterion for the minimum coefficient of friction is 0.6 and the revised ITAAC states this. The COLA markup included at the end of this response indicates that the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface is 0.6.

4. The bottom of gravel and soil interface is governed by the friction forces that develop under the Reactor Building and Control Building resulting from the properties of the existing materials under the buildings.

   The coefficient of friction and cohesion values for gravel and soil interfaces mobilize the full soil shear strength and require adjustment to account for cyclic loading. The cyclic yield strength of soils are the maximum stress level below which the material exhibits nearly elastic behavior and above which the material exhibits permanent plastic deformation whose magnitude depends on the number of cycles applied. These dynamic effects were taken into account for the cyclic seismic loading conditions based on experimental data from Makdisi and Seed (Makdisi, F.I. and Seed, H.B. 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformations," Journal of the Geotechnical Engineering Division, ASCE Vol 104, No. GT7, p 849-867).

Sand layers beneath the Unit 3 and Unit 4 Control Buildings have a coefficient of friction of 0.70, and the clay layers beneath both the Unit 3 and Unit 4 Reactor Buildings have a cohesion value of 3.4ksf according to FSAR Section 2.5S.4. Sliding resistance is provided by both passive lateral soil pressure and friction. Using 67% (dynamic effect reduction) of the sand friction coefficient (0.70 reduces to 0.47) and 80% of the cohesion (3.4ksf reduces to 2.72 ksf) provides sufficient safety margin on the lateral passive earth pressure required to meet the safety factor against sliding.

The soil friction angles and cohesion values that were used in the sliding evaluations of the Control Building and Reactor Building are provided by COLA Part 2, Tier 2, Table 2.5S.4-37B and Table 2.5S.4-38B. The interface between the bottom of gravel and sandy soil for the Control Building will have a coefficient of friction of 0.70 for static loading based on the tangent of the friction angle (φ) as provided by COLA Part 2, Tier 2, Table 2.5S.4-37B for the Reactor Building and COLA Part 2, Tier 2, Table 2.5S.4-38B for the Control Building, but is reduced to two-thirds the value in order to compensate for repeated cyclic (seismic) loading, bringing the resultant coefficient of friction to 0.47. In addition, the gravel to gravel coefficient of friction is 0.75 to 0.84 and gravel to soil friction is governed by the shear resistance of gravel or soil (sand or clay), whichever is less.

The coefficient of friction needed to maintain the minimum factor of safety was reported as 0.47 in Revision 1 of the response to RAI 03.08.04-19 (STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The evaluations were based on the available coefficient of friction and showed sufficient margin in the required passive pressure to be developed.

Part of the Reactor Building will be constructed over clay, rather than sandy soil. The resistance to sliding for these locations is based on cohesion of the clay (3.4 ksf) as provided in COLA Part 2, Tier 2, Table 2.5S.4-37B. The evaluations for this case similarly showed sufficient margin in the required passive pressure to be developed.

Analysis considered a 20% reduction of the cohesion capacity to account for dynamic effects. To achieve a Factor of Safety of 1.11, as reported in DCD Table 3H.1-23, only 40% of the available passive pressure is engaged. The passive pressure contributes 50% of the total sliding resistance. This is conservative for the Unit 3 Reactor Building because the Unit 4 Reactor Building, which is assumed to be founded only on top of clay, results in a lower Factor of Safety than the Unit 3 Reactor Building, which is partially founded on sandy soil.

COLA will be revised as shown below as a result of this response and will completely supersede COLA revisions provided in RAI 03.08.04-19 (see STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The revisions to the COLA markup provided in RAI 03.08.04-19 and RAI 03.08.04-28 are shown by revision bars in the margin.

1. COLA Revision 5, Part 2, Tier 2, 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 waterproofing membrane near the top elevation of the concrete fill. The remainder of the concrete fill is then poured on top of the waterproofing material. A waterproof membrane that could degrade the ability of the foundation to transfer loads is not used.

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 total thickness of the waterproofing membrane will be a nominal 120 mils.

Additional testing on the waterproofing membrane will be required to demonstrate the adequacy of the membrane's performance under applicable mechanical conditions, including pressures from the backfill, hydrostatic pressure, and foundation bearing. Test conditions will simulate the environment at the walls and the base level. The horizontal membrane (located in the structural concrete fill) will also be tested for its resistance to the hydrostatic pressures at the membrane location, as the basic assumption that necessitates the use of waterproofing is that cracks in the concrete fill will allow water to propagate up to the waterproofing membrane.

The membrane will be tested in accordance with ASTM D5385, Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes, which requires that the membrane be subjected to a pressure of 100 psi. The acceptance criterion is that the sample is able to resist the expected hydrostatic pressure.

The waterproofing membrane will be tested per ASTM C267 (Standard Test Methods for Chemical Resistance of Mortars, Grouts, and Monolithic Surfacings and Polymer Concretes) for its resistance to the concrete mix chemistry, the actual backfill material chemistry, and groundwater chemistry found on site. Additional testing of the waterproofing membrane's ability to resist the chemical reagents as specified through accelerated aging will be done per ASTM G114 (Standard Practices for Evaluating the Age Resistance of Polymeric Materials Used in Oxygen Service). The margin provided by the testing, for chemicals and pressure exposures, along with the results from accelerated age testing will ensure that the waterproofing will sufficiently resist the projected environmental pressures over its intended lifetime.

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 of 0.60 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. The tests will be performed with the expected range of normal compressive stresses. The coefficient of friction, as defined in ASTM D1894, is the ratio of the force required to move one surface over another to the total force applied normal to those surfaces. The test fixture assembly will be designed to obtain a series of shear / lateral forces and the corresponding applied normal compressive loads. The test data will be generally represented by a best fit straight line whose slope is the coefficient of friction.

2. COLA, Revision 5, Part 9 will be revised to add the following site-specific ITAAC.

3.0 Site-Specific ITAAC

Table 3.0-13 Waterproofing Membrane		
Design Commitment	**Inspections, Tests, Analyses**	**Acceptance Criteria**
The static friction coefficient to resist sliding beneath the basemat of Category I structures is at least 0.60 ~~meets the required friction coefficient to prevent sliding.~~	Type testing will be performed on a membrane of the material and thickness specified for the waterproof system 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 static friction of at least 0.60 to support the analysis against sliding.

RAI 03.08.04-29, Supplement 1

QUESTION:

Follow-up to Question 03.08.04-22

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

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

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

   - $D + F + L + H + T_a + E'$ – Provide justification for using only lateral soil pressure H, and not H', which includes seismic effects.

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

SUPPLEMENTAL RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010. This supplemental response is being submitted to clarify that pipe break loads are not applicable to the site-specific structures, as discussed in meeting with NRC on February 2nd and 3rd, 2011.

Ta is described in Section 3H.6.4.3.3.6 as the Ultimate Heat Sink (UHS) Basin Water temperature (95°F) during accident conditions. Other loads specified in ACI 349 such as R_a, Y_r, Y_j, Y_m are not included since there are no high energy line breaks associated with the UHS/ Reactor Service Water Pump House or other site-specific Seismic Category I Structures.

COLA Part 2, Tier 2 will be revised as shown in Enclosure 1.

Enclosure 1
Revision to COLA Section 3H.6

3H.6.4.3.4 Load Combinations

The load combinations and structural acceptance criteria used to evaluate the site-specific Category I concrete structures are consistent with the provisions of ACI 349, as supplemented by RG 1.142 as well as ACI 350. Loads ~~T_a,~~ R_a, P_a, Y_r, Y_j and Y_m ~~and E_o,~~ as defined in ACI 349, are not applicable to the evaluation of the site-specific seismic Category I structures since there are no high energy line breaks associated with the site-specific Category I concrete structures; therefore these loads ~~and~~ are not included in the load combinations defined below.

RAI 03.08.04-32, Revision 1

QUESTION:

Follow-up to Question 03.08.04-27

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

REVISED RESPONSE:

This revised response is being submitted based on discussions in meetings with NRC on February 2 and 3, 2011 This revised response completely supersedes the responses to RAI 03.08.04-6 (provided in letter U7-C-STP-NRC-090136 dated September 15, 2009) , RAI 03.08.04-27 (provided in letter U7-C-STP-NRC-100036 dated February 10, 2010), and the previous response to RAI 03.08.04-32 (provided in letter U7-C-STP-NRC-100208 dated September 15, 2010). The revisions are marked by revision bars in the margin. This revised response includes the following as discussed in the February meeting:

- Plans and developed elevation of the containment showing the proposed locations of measurement of displacements and strains.

- Confirmation that the ranges selected for the instrumentation are consistent with the predicted deformation.

- Detailed description of how test results will be evaluated to ensure full compliance with the acceptance criteria per Subarticle CC-6400 of ASME Section III, Division 2 and per the Regulatory Guide.

- How and when crack mapping locations are determined.

- When the calculation for the predictive analyses will be performed.

Details of the Test and Instrument Plan for the Structural Integrity Test (SIT) are provided below. The Unit 3 Reinforced Concrete Containment Vessel (RCCV) is classified as a prototype containment. Therefore, the test and instrument plan for the Unit 3 SIT has been developed to conform to the requirements for prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2. The test and instrument plan for the Unit 4 SIT will conform to the requirements for non-prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2.

The following is a summary of SIT requirements for Units 3&4 based on Article CC-6000 of ASME Section III, Division 2. These will be included in the ASME Construction Specification for the Containment.

I. Details of the Test:

The containment shall be subjected to integrity tests that include both an overall internal pressure test and a differential pressure test. The overall SIT will be performed at a test pressure of at least 1.15 times the containment design pressure in both the drywell and suppression chamber simultaneously. The differential pressure test will be performed at a test pressure of at least 1.0 times the maximum design differential pressure. The test pressure will be held for at least 1 hour. The detailed non-linear finite element analysis for predictions of strains (Unit 3 only) and displacements during SIT will be made after the detailed design of the RCCV is complete and at least 12 months prior to the start of the SIT.

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

1. Test Description & Objectives

   a. The SIT will test the RCCV for structural performance acceptability as a prerequisite for Code Acceptance and stamping. The test will be conducted in accordance with the 2001 Edition, including 2003 addenda, of the ASME Boiler & Pressure Vessel Code, Section III, Division 2, Article CC-6000 (hereinafter referred to as the ASME Code).

   b. The SIT is performed at a test pressure of at least 1.15 times the containment design pressure of 45 psig (1.15x45=51.75 psig) to demonstrate the quality of construction and to verify the acceptable performance of new design features. The structural response of the system under the required maximum test pressure - measured in terms of displacements, strain (Unit 3 only) and cracking - shall be recorded and the data shall be presented in a final report.

c. Evaluation of SIT results will be conducted in accordance with Section CC-6400 of the ASME Code using the acceptance criteria given in Section CC-6410.

d. The SIT shall be performed using atmospheric air.

2. Test Parameters:

a. Loading

i. Pressurization/depressurization of the RCCV

The SIT will subject the RCCV to a pressurization/depressurization sequence during which the internal pressure is increased from atmospheric pressure to the test pressure at which point pressure inside the RCCV will be held at maximum test pressure for at least 1 hour. Afterwards, the internal pressure is decreased from the maximum test pressure to atmospheric pressure. A detailed description of the test pressurization sequence is provided in Section I.2.a.iii below.

ii. Differential pressurization/depressurization of drywell and suppression chamber

The SIT will subject the drywell of the RCCV to a differential pressurization/depressurization sequence while the suppression chamber is at the atmospheric pressure. For this test, the internal pressure of the drywell is set to 25 psig and held at this level for at least 1 hour.

iii. Pressurization Sequence

The pressurization/depressurization rate during the test shall not exceed 20% of the maximum test pressure per hour, or 10.35 psig per hour. The pressurization and depressurization shall be performed using a minimum of 5 pressure steps. At the end of each step, the pressure shall be held for a minimum of 1 hour to collect a full set of strains (Unit 3 only), displacements, and temperatures. Once the full SIT test pressure is obtained, the pressure shall be held for a minimum of 2 hours to perform crack mapping in addition to collecting a full set of strains (Unit 3 only), displacements, and temperatures. The same process shall be used during the depressurization phase of the test.

b. Response

i. Displacement

Displacement measurements shall be taken at the following locations (as shown in Figures 03.08.04-32.1 through 03.08.04-32.3):

1 Radial displacements in the drywell: top of the upper drywell, mid-height of the upper drywell, and above the diaphragm floor. Radial displacements in the suppression chamber (SC): top of the SC, mid-height of the SC, and above the basemat. Measurements shall be made at a minimum of four approximately equally spaced azimuths and should be perpendicular to the containment centerline.

2 Radial displacements of the containment wall adjacent to the largest opening, at a minimum of 12 points, four equally spaced on each of three concentric circles. The diameter for the inner circle shall be large enough to permit measurements to be made on the concrete rather than on the steel sleeve; the middle approximately 1.75 times the diameter of the opening; and the outer approximately 2.5 times the diameter of the opening. The change in the diameter of the opening shall be measured on the horizontal and vertical axes.

3 Vertical displacement of the RCCV walls at the top of the drywell relative to the basemat–wall junction, measured at a minimum of four approximately equally spaced azimuths.

4 Vertical displacement of the drywell top slab relative to the basemat near the reactor shield wall, and vertical displacement of the drywell top slab relative to the basemat at two other approximately equally spaced locations between the reactor shield wall and the primary vertical wall of the RCCV on a common azimuth.

ii. Strain (Unit 3 only)

Per requirements of Section CC-6370 of ASME code, the Unit 3 prototype containment shall be instrumented to measure strain. At a minimum, strain measuring instrumentation will be located at two azimuths, 90 degrees apart, to demonstrate the structural behavior of the following areas of the RCCV (as shown in Figures 03.08.04-32.4 through 03.08.04-32.7):

- the intersection of the shell and the basemat.
- near mid-height on the suppression chamber.
- near mid-height on the upper drywell.
- the vicinity of the lower drywell access tunnel at azimuth 180 deg.
- the intersection of the shell and the top slab.
- the intersection of the shell and the diaphragm floor.
- the intersection of the top slab and the drywell head.

iii. Temperature

Ambient temperature shall be measured inside and outside the RCCV. In addition, per requirements of Section CC-6380 of ASME code, for the Unit 3 prototype containment, temperatures shall be measured at all strain gage locations to establish representative temperatures for strain measurements. Temperature measurements shall be used to correct measured strain values for thermal effects.

iv. Crack mapping

Per requirements of Section CC-6350 of ASME code, concrete surface cracks shall be mapped. The patterns of cracks that exceed 0.01 in (0.25 mm) in width and 6 in. (152 mm) in length shall be mapped at specified locations before the test, at maximum pressure, and after the test. At each location, an area of at least 40 sq ft (3.7 m^2) shall be mapped. Locations for crack mapping will be finalized after the completion of the RCCV construction and SIT prediction analysis as well as the completion of engineering for placement of the equipment, piping, cables, and steel frames and galleries so that locations selected will:

1. include areas with physical crack that exceed 0.01in. in width and 6 in. in length.
2. include areas where high surface tensile strain is predicted.
3. be easily accessed before, during, and after the SIT.

v. Post-test examination

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

II. Instrumentation:

Instrumentation for the measurement of pressure, displacement, strain (for Unit 3), crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing.

The instrumentation ranges selected were based on the anticipated test conditions and the ASME Section III, Division 2 Code (Article CC-6000) requirements and are deemed adequate based on SIT analyses and test results from similar ABWR plants in Japan. In addition, the accuracy and sensitivity of the gages were selected to meet the code requirements and with due considerations to the incremental accuracy to capture fine variation of the measured quantity.

Displacement, strain (for Unit 3), and temperature measurements will be made in accordance with CC-6300 of ASME Section III, Division 2. Test data will be collected in accordance with CC-6340 of ASME Section III, Division 2. For the prototype Unit 3 Containment, strains and associated temperatures will be measured for a minimum period of 24 hours prior to the SIT to evaluate the strain variations resulting from temperature change.

1. <u>Equipment Description</u>

   a. Pressurization system

      (a) The pressurization system shall be capable of attaining and holding the maximum test pressure of 51.75 psig during the pressurization/ depressurization of the RCCV and a test pressure of 25 psig during the differential pressurization/depressurization of the drywell and suppression chamber.

      (b) Equipment inside the RCCV that will be subject to pressure from the SIT sequence shall be prepared for the test appropriately, including potential for water vapor condensation.

   b. Data acquisition system specifications

      (a) Data loggers will be used to collect data from various system components including thermometers, strain gages, pressure gages, and displacement transducers. Input/output measurement and control modules, multiplexers, communication interface equipment, battery backup power supplies and signal conditioning equipment shall be supplied as necessary based upon the configuration and features of the instrumentation equipment used.

(b) The data loggers shall have appropriate non-volatile on-board memory to minimize inadvertent loss of data. Sufficient data storage capacity will be provided to store data collected from all gages during the structural integrity test without interruption.

(c) Data collected from all gages shall have a time stamp.

c. Specifications for instrumentation

(a) Sister bar strain gages
Sister bar strain gages are the preferred choice for measurement of strain in reinforcing steel.

1 Sister bar strain gages will be properly secured to the rebar cage at pre-defined locations (indicated in Section I.2.b.ii above) and embedded in the concrete during concrete placement. The end-to-end length of the bar segment used for the sister bar strain gages shall be two times the development length of the sister bar plus either 4 in. or the protected length of the sister bar, whichever is greater. The sensing components shall be foil type resistance strain gages as described below. The foil type resistance strain gages shall be installed in a full bridge, 4-arm configuration for improved stability. The gages shall be mounted at two locations around the circumference of the sister rebar at mid-length. The two locations shall be positioned at ± 180 degrees from each other. The strain gages shall be bonded to the sister bar by strain gage epoxy if directly attached to the rebar, or spot welded if previously encapsulated inside a stainless steel shim. The rebar surface at the location of the strain gage attachment shall be prepared according to the strain gage manufacturer installation requirements. A thermistor shall also be attached to the rebar, near the strain gages, to permit the differentiation of thermally induced strains from load induced strains. The strain gages and thermistor shall be protected against moisture and chemical and mechanical damage. Moisture protective material shall be a type used for underwater applications such as silicone. A protective coating such as polysulfide shall be applied over the water proofing material to protect the strain gages against mechanical and chemical damages. A heat shrinkage protector shall be further applied over the

protective coatings for further reinforcement. Each fabricated sister bar strain gage shall be tested by complete water immersion for at least 24 hrs. The sister bar element shall be supplied with an appropriate cable as defined in Section II.1.d. with an appropriate length of cable such that there are no cable splices inside the concrete. In addition, when splices are required outside the concrete, all connections shall be soldered and then protected from moisture and other contamination with a suitable cable splice sealant. The cables shall be waterproofed and sealed as an integral part of the assembly.

2 The foil type strain gages shall have following characteristics:

a.	Standard Range	3000 micro strain
b.	Sensitivity	1 micro strain
c.	Accuracy	5% of the maximum anticipated strain or 10 microstrain, whichever is greater

(b) Displacement transducer

1 Linear variable displacement transducers (LVDTs) shall be used for both vertical and horizontal displacement measurements. Inside the suppression chamber submersible LVDTs shall be used for measurement locations that are below the water line.

2 LVDTs shall have the following minimum characteristics:

a.	Travel Range	0.5 in
b.	Output	4-20 mA
c.	Minimum Linearity	$\pm$0.30% full scale
d.	Min Repeatability	$\pm$0.015% full scale

(c) Temperature gage

1 Temperature devices shall be resistance type and shall be sealed against moisture. Thermistors used in fabrication of sister bar gages shall have diffusivity approximately that of steel.

2 Temperature sensing element shall be supplied with an appropriate cable as defined in Section II.1.d. The cables shall be waterproofed and sealed as an integral part of the assembly.

(d) Pressure gage

1 Pressure gages used in pressure testing shall be connected directly to the internal environment of the containment, and measure the differential pressure between the internal and external environments. This shall be accomplished either by using an absolute pressure gage inside and another absolute gage outside of the RCCV or by using a gage-pressure gage directly attached to the pressurizing pump outlet outside of the RCCV right after the shut-off valve. The pressure gages shall be voltage output (as compared to millivolt output type) with integrated signal conditioning electronics included. The pressure gages shall be supplied with an appropriate cable as defined in Section II.1.d. The pressure gage cables shall be waterproofed and sealed as an integral part of the assembly.

2 The pressure gages shall have the following characteristics:

a. Range	0-200 psi
b. Accuracy	$\pm$0.25 psi

d. Cable specifications

Instrumentation cable type and size shall be shielded 16 AWG twisted paired for all instruments. The shield shall be either braided strands of copper (or other metal), a non-braided spiral winding of copper tape (or other metal), or a layer of conducting polymer. The shield shall be applied across cable splices. In addition, the cable shall have drain wire.

III Evaluation of Test Results:

Crack and strain (for Unit 3) measurements will be reviewed by the Designer for evaluation of the overall test results. The RCCV will be considered to have satisfied the structural integrity test if the test results are evaluated as decribed and meet without exception the following minimum requirements specified in CC-6410 of ASME Section III, Division 2 and also adopted by the Regulatory Guide 1.136:

1. Yielding of conventional reinforcement does not develop as determined from analysis of crack width, strain, or displacement data.

The Designer will analyze the measured crack width, strain and/or displacement data and also review the results of the SIT prediction analysis to determine if yielding of conventional reinforcement has occurred.

2. No visible signs of permanent damage to either the concrete structure or the steel liner are detected. Evidence, resulting from the test, of spalling, laminations, or voids behind the liner are pertinent considerations. Special care shall be exercised in the post-test examinations (CC-6390) to detect evidence of localized distress which may not be revealed by strain or displacement data. The significance of such distress, if detected, must be determined by the Designer and be acceptable to the Owner.

   The Designer will participate in the pretest examination per CC-6210 to establish the baseline conditions, will witness the test, and will participate in the post-test examination per CC-6390. The significance of potential local distress observed, such as concrete spalling, lamination, and liner bulging, etc., will be evaluated by the Designer and reported to the Owner.

3. Residual displacements at the point of maximum predicted radial and vertical displacement at the completion of depressurization or up to 24 hours later shall not exceed 30% of measured or predicted displacement at maximum test pressure, whichever is greater, plus 0.01in. (0.25mm) plus measurement tolerance. This criterion shall apply to the average of radial displacements measured at the same elevation.

   Residual displacements will be recorded and will be monitored by the Designer. Residual displacements at the point of maximum predicted radial and vertical displacement at the completion of depressurization or up to 24 hours later will be compared against the acceptance limit of "30% of the greater of measured or predicted displacement at maximum test pressure, plus 0.01in. plus measurement tolerance" set forth in this code provision. Radial displacements measured at the same elevation will be averaged for this comparison as required by the Code.

4. The measured displacements at test pressure at points of predicted maximum radial and vertical displacements do not exceed predicted values by more than 30% plus measurement tolerance. This criterion shall apply to the average of radial displacement measured at the same elevation. This requirement may be waived if the residual displacements within 24 hours are not greater than 20%.

   The measured displacements at test pressure at points of predicted maximum radial and vertical displacements will be recorded and will be compared by the Designer against the acceptance limit of "1.3 times the predicted values, plus measurement tolerance" set forth in this code provision. Radial displacements measured at the same elevation will be averaged for this comparison as required by the Code. This requirement will be waived if the residual displacements within 24 hours are not greater than 20%.

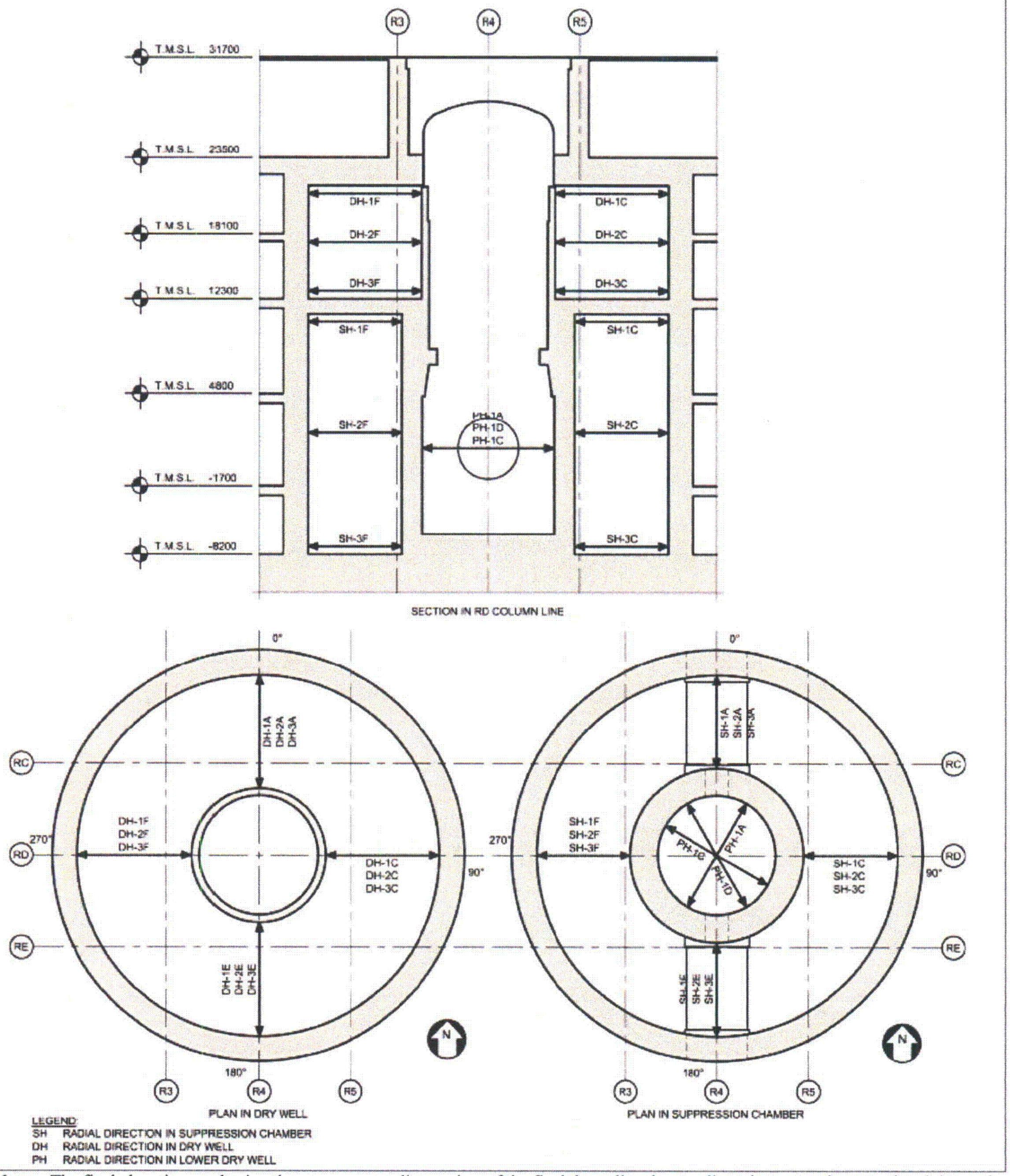
If measurements and studies by the Designer indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2.

IV Test Report:

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

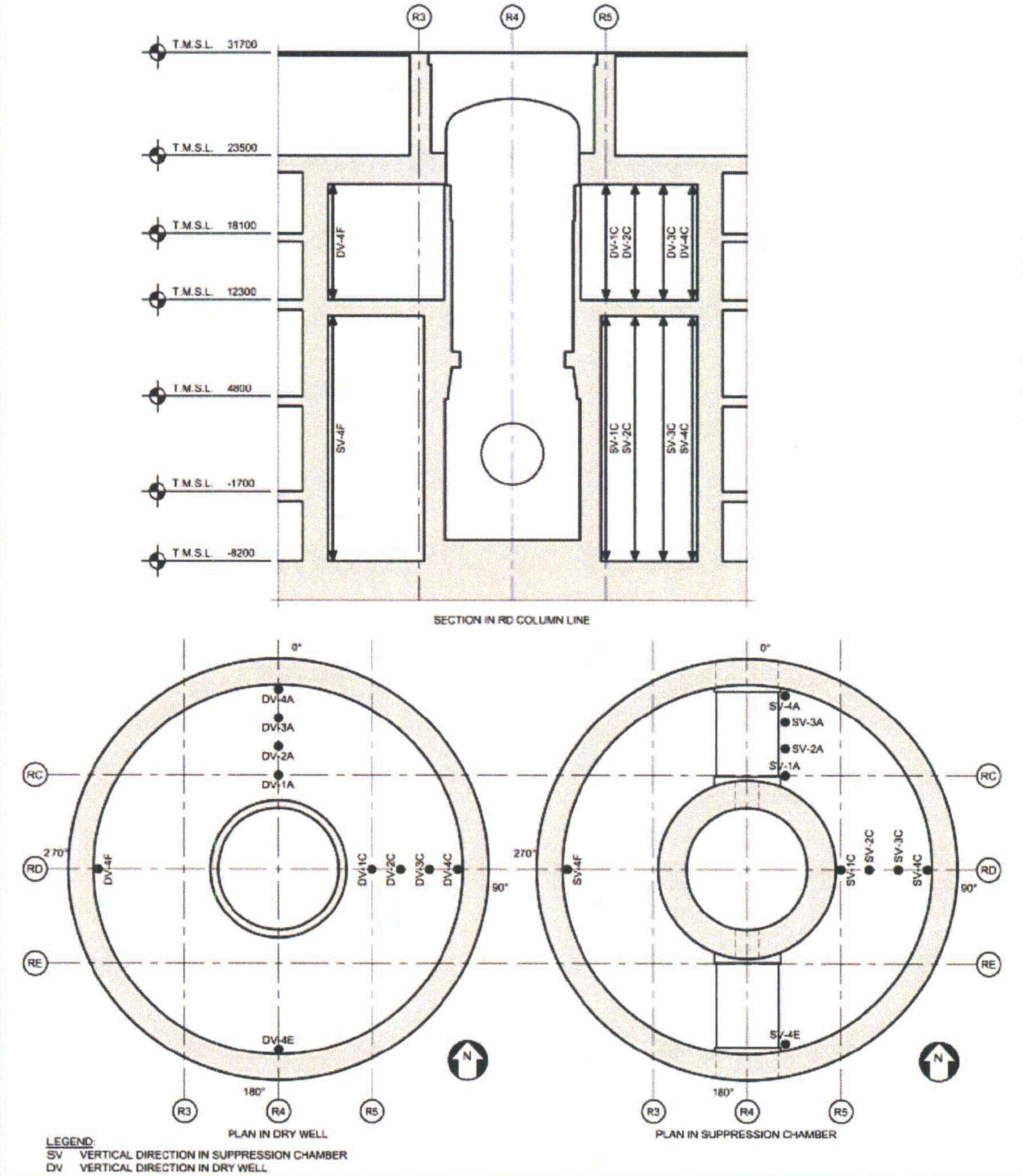
The COLA will be revised as provided in the enclosure to this response.

Figure 03.08.04-32.1: Radial displacement measurement plan in drywell, suppression chamber, and lower drywell



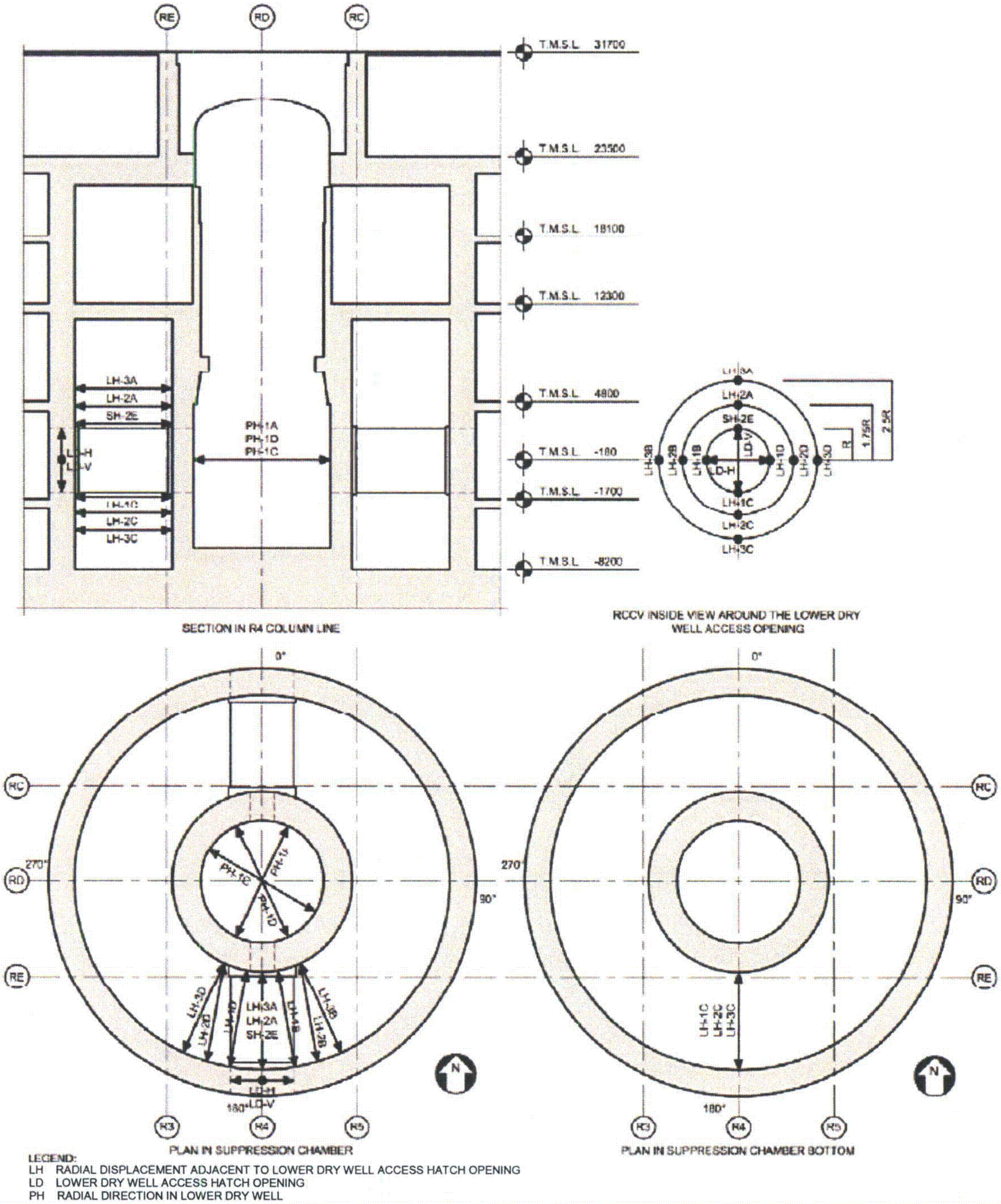
Notes: The final elevations and azimuths may vary pending review of the final drywell and wetwell steel structure layouts, the piping and cable routings, the available liner anchor locations for bracket attachment, and the clearance required for the displacement LVDT transducer and the associated weight-pulley-invar wire arrangements.

Figure 03.08.04-32.2: Vertical displacement measurement plan in drywell and suppression chamber



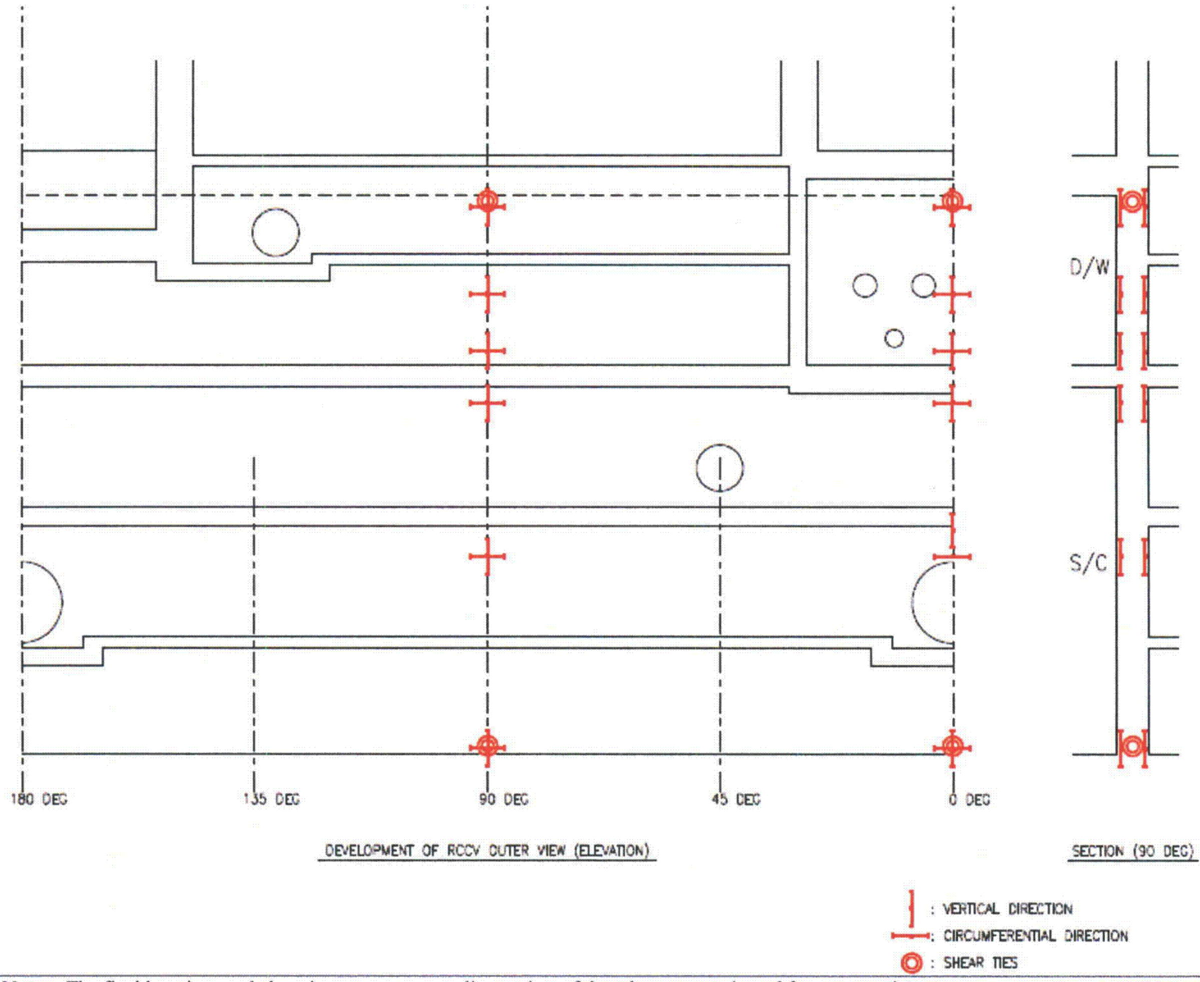
Notes: The final locations and azimuths may vary pending review of the final drywell and wetwell steel structure layouts, the piping and cable routings, the available liner anchor locations for bracket attachment, and the clearance required for the displacement LVDT transducer and the associated weight-pulley-invar wire arrangements.

Figure 03.08.04-32.3: Radial displacement measurement plan adjacent to lower drywell access opening



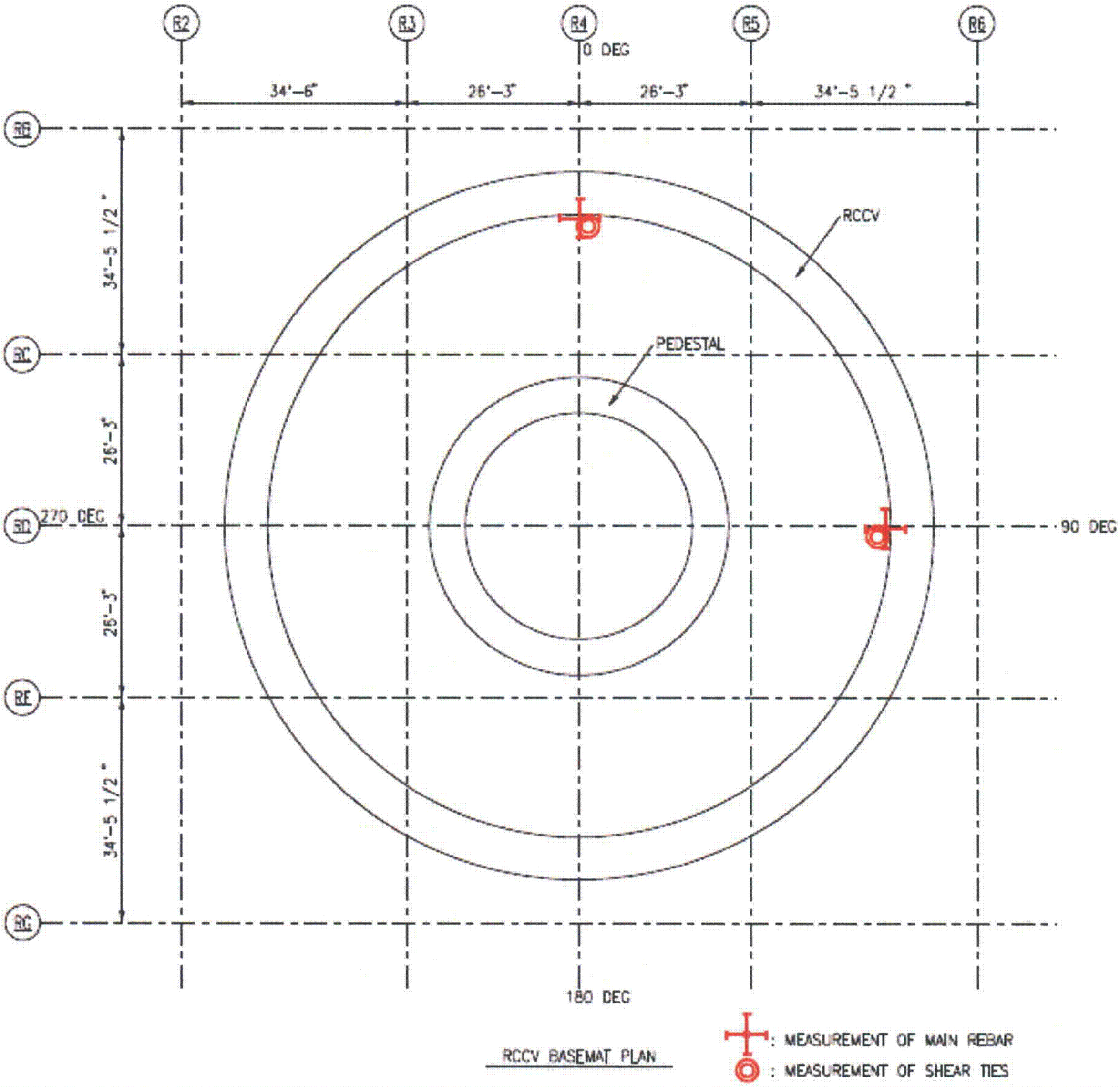
Notes: The final azimuths and distance from the center of the access opening may vary due to local interferences.

Figure 03.08.04-32.4: Rebar strain measurement plan in RCCV shell



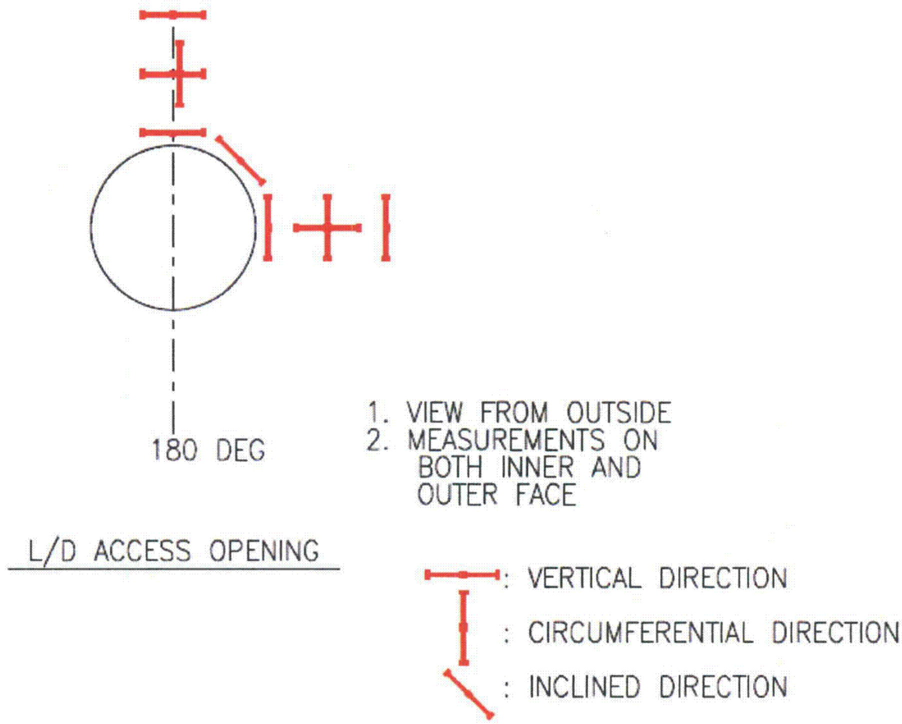
Notes: The final locations and elevations may vary pending review of the rebar patterns issued for construction.

Figure 03.08.04-32.5: Rebar strain measurement plan in basemat



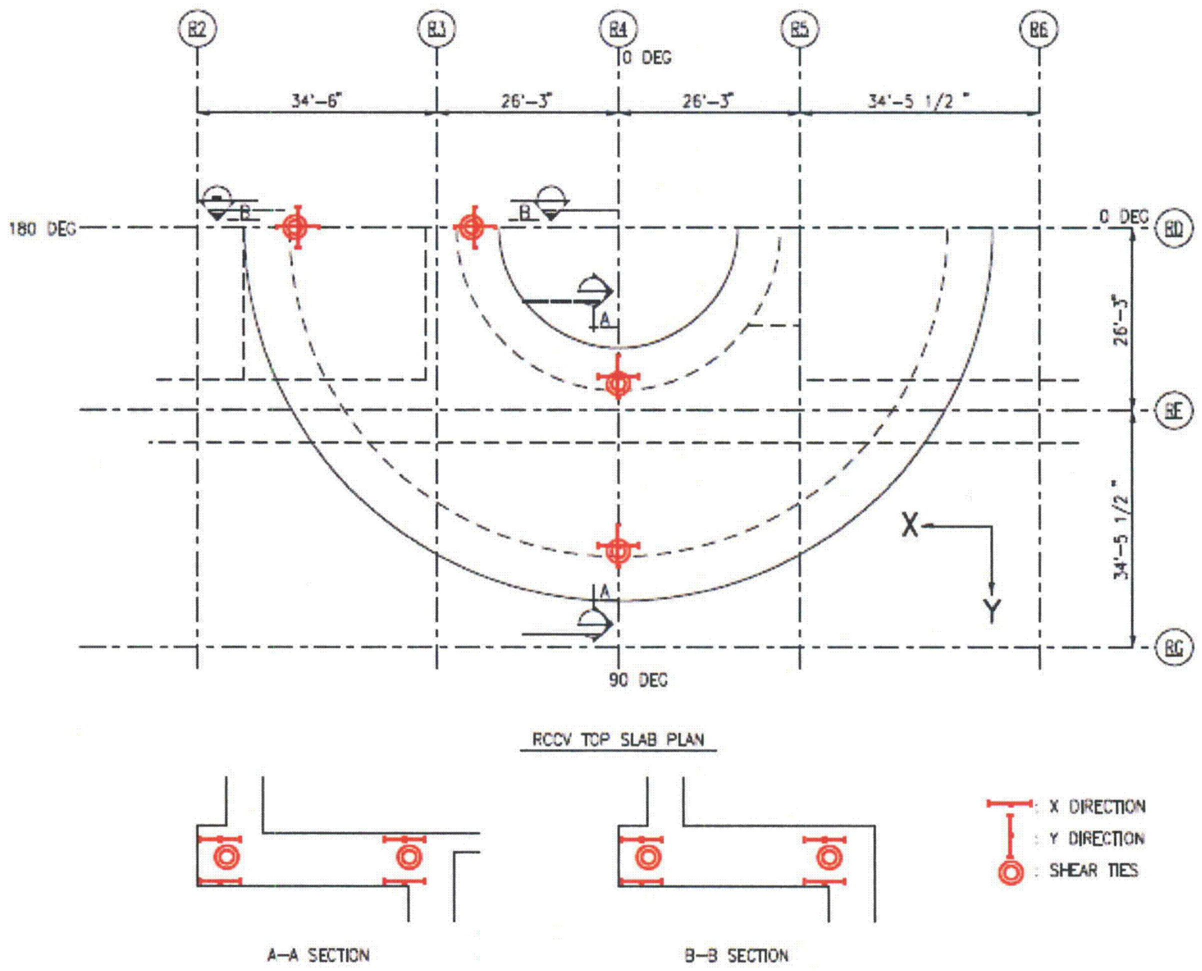
Notes: The final locations and azimuths may vary pending review of the rebar patterns issued for construction.

Figure 03.08.04-32.6: Rebar strain measurement plan at lower drywell access opening



Notes: The final locations and elevations may vary pending review of the rebar patterns issued for construction.

Figure 03.08.04-32.7: Rebar strain measurement plan in RCCV top slab



Notes: The final locations and azimuths may vary pending review of the rebar patterns issued for construction.

Enclosure to RAI 03.08.04-32

Revision to COLA Section 3.8.6.3

Section 3.8.6.3 of the COLA will be revised as follows:

3.8.6.3 Structural Integrity Test Result

The following standard supplement addresses COL License Information Item 3.25.

Structural Integrity Test (SIT) of the containments will be performed in accordance with Subsection 3.8.1.7.1 and ITAAC Table 2.14.1 Item #3. ~~The first~~The Unit 3 containment will be considered a prototype and its SIT performed accordingly. The details of the test and the instrumentation~~, as required for such a test, will be provided in the ASME Construction Specification~~ are provided in the following subsections. The test and instrument plan for the Unit 3 SIT will conform to the requirements for prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2. The test and instrument plan for the Unit 4 SIT will conform to the requirements for nonprototype containments as delineated in Article CC-6000 of ASME Section III, Division 2.

3.8.6.3.1 Details of the Test:

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

Predictions of displacements and strains will be made prior to the start of the Unit 3 test. During the SIT ~~tests~~, the suppression chamber and spent fuel pool will be filled with water to the normal operational water level. Atmospheric air will be used as the testing medium for both the overall and the differential pressure test. The Designer or his designee will perform a pretest visual examination of the accessible portions of the ~~primary containment vessel~~ Reinforced Concrete Containment Vessel (RCCV) prior to the ~~s~~Structural ~~i~~Integrity ~~(SI)~~ ~~t~~Test (SIT) in accordance with CC-6210 of ASME Section III, Division 2. The Designer or his designee will witness the SIT~~test~~ and will monitor displacement measurements.

3.8.6.3.1.1 Test Description & Objectives

(1) The SIT will test the RCCV for structural performance acceptability as a prerequisite for Code Acceptance and stamping. The test will be conducted in accordance with the 2001 Edition, including 2003 addenda, of the ASME Boiler & Pressure Vessel Code, Section III, Division 2, Article CC-6000 (hereinafter referred to as the ASME Code).

(2) The SIT is performed at a test pressure of at least 1.15 times the containment design pressure of 45 psig (1.15x45=51.75 psig) (357 kPag) to demonstrate the quality of construction and to verify the acceptable performance of new design features. The structural response of the system under the required maximum test pressure - measured in terms of displacements, strain (Unit 3

only) and cracking - shall be recorded and the data shall be presented in a final report.

(3) Evaluation of SIT results will be conducted in accordance with Section CC-6400 of the ASME Code using the acceptance criteria given in Section CC-6410.

(4) The SIT shall be performed using atmospheric air.

3.8.6.3.1.2 Test Parameters:

(1) Loading

(a) Pressurization/depressurization test of the RCCV

The SIT will subject the RCCV to a pressurization/depressurization sequence during which the internal pressure is increased from atmospheric pressure to the test pressure at which point pressure inside the RCCV will be held at maximum test pressure for at least 1 hour. Afterwards, the internal pressure is decreased from the maximum test pressure to atmospheric pressure. A detailed description of the test pressurization sequence is provided in Subsection 3.8.6.3.1.2(1)(c) below.

(b) Differential pressurization/depressurization of drywell and suppression chamber

The SIT will subject the drywell of the RCCV to a differential pressurization/depressurization sequence while the suppression chamber is at the atmospheric pressure. For this test, the internal pressure of the drywell is set to 25 psig (172 kPag) and held at this level for at least 1 hour.

(c) Pressurization Sequence

The pressurization/depressurization rate during the test shall not exceed 20% of the maximum test pressure per hour, or 10.35 psig per hour. The pressurization and depressurization shall be performed using a minimum of 5 pressure steps. At the end of each step, the pressure shall be held for a minimum of 1 hour to collect a full set of strains (Unit 3 only), displacements, and temperatures. Once the full SIT test pressure is obtained, the pressure shall be held for a minimum of 2 hours to perform crack mapping in addition to collecting a full set of strains (Unit 3 only), displacements, and temperatures. The same process shall be used during the depressurization phase of the test.

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

(b.5) the intersection of the shell and the top slab.
(b.6) the intersection of the shell and the diaphragm floor.
(b.7) the intersection of the top slab and the drywell head.

(c) Temperature

Ambient temperature shall be measured inside and outside the RCCV. In addition, per requirements of Section CC-6380 of ASME code, for the Unit 3 prototype containment, temperatures shall be measured at all strain gage locations to establish representative temperatures for strain measurements. Temperature measurements shall be used to correct measured strain values for thermal effects.

(d) Crack mapping

Per requirements of Section CC-6350 of ASME code, concrete surface cracks shall be mapped. The patterns of cracks that exceed 0.01 inch (0.25 mm) in width and 6 inches (152 mm) in length shall be mapped at specified locations before the test, at maximum pressure, and after the test. At each location, an area of at least 40 sq ft (3.7 m^2) shall be mapped.

Locations for crack mapping will be finalized after the completion of the RCCV construction and SIT prediction analysis as well as the completion of engineering for placement of the equipment, piping, cables, and steel frame and galleries so that locations selected will:

1. include areas with physical cracks that exceed 0.01 inch (0.25 mm) in width and 6 inches (152 mm) in length.
2. include areas where high surface tensile strain is predicted.
3. be easily accessed before, during, and after the SIT.

(e) Post-test examination

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

3.8.6.3.2 Instrumentation:

Instrumentation for the measurement of pressure, displacement, strain (for Unit 3), crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing.

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

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

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

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

3.8.6.3.2.1 Equipment Description

(1) Pressurization system

(a) The pressurization system shall be able to attain and hold the maximum test pressure of 51.75 psig (357 kPag) during the pressurization/ depressurization of the RCCV and a test pressure of 25 psig (172 kPag) during the differential pressurization/depressurization of the drywell and suppression chamber.

(b) Equipment inside the RCCV that will be subject to pressure from the SIT sequence shall be prepared for the test appropriately, including potential for water vapor condensation.

(2) Data acquisition system specifications

(a) Data loggers will be used to collect data from various system components including thermometers, strain gages, pressure gages, and displacement transducers. Input/output measurement and control modules, multiplexers, communication interface equipment, battery backup power supplies and signal conditioning equipment shall be supplied as necessary based upon the configuration and features of the instrumentation equipment used.

(b) The data loggers shall have appropriate non-volatile on-board memory to minimize inadvertent loss of data. Sufficient data storage capacity will be provided to store data collected from all gages during the structural integrity test without interruption.

(c) Data collected from all gages shall have a time stamp.

(3) Specifications for instrumentation

(a) Sister bar strain gages

Sister bar strain gages are the preferred choice for measurement of strain in reinforcing steel.

(a.1) Sister bar strain gages will be properly secured to the rebar cage at pre-defined locations (See Section 3.8.6.3.1.2(2)(b)) and embedded in the concrete during concrete placement. The end-to-end length of the bar segment used for the sister bar strain gages shall be two times the development length of the sister bar plus either 4 in. or the protected length of the sister bar, whichever is greater. The sensing components shall be foil type resistance strain gages as described below. The foil type resistance strain gages shall be installed in a full bridge, 4-arm configuration for improved stability. The gages shall be mounted at two locations around the circumference of the sister rebar at mid-length. The two locations shall be positioned at +180 degrees from each other. The strain gages shall be bonded to the sister bar by strain gage epoxy if directly attached to the rebar, or spot welded if previously encapsulated inside a stainless steel shim. The rebar surface at the location of the strain gage attachment shall be prepared according to the strain gage manufacturer installation requirements. A thermistor shall also be attached to the rebar near the strain gages, to permit the differentiation of thermally induced strains from load induced strains. The strain gages and thermistor shall be protected against moisture and chemical and mechanical damage. Moisture protective material shall be a type used for underwater applications such as silicone. A protective coating such as polysulfide shall be applied over the water proofing material to protect the strain gages against mechanical and chemical damages. A heat shrinkage protector shall be further applied over the protective coatings for further reinforcement. Each fabricated sister bar strain gage shall be tested by complete water immersion for at least 24 hrs. The sister bar element shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) below with an appropriate length of cable such that there are no cable splices inside the concrete. In addition, when splices are required outside the concrete, all connections shall be soldered and then protected from moisture and other contamination with a suitable cable splice sealant. The cables shall be waterproofed and sealed as an integral part of the assembly.

(a.2) The foil type strain gages shall have following characteristics:

a. Standard Range	3000 micro strain
b. Sensitivity	1 micro strain
c. Accuracy	5% of the maximum anticipated strain or 10 microstrain, whichever is greater

(b) Displacement transducer

(b.1) Linear variable displacement transducers (LVDTs) shall be used for both vertical and horizontal displacement measurements. Inside the suppression chamber submersible LVDTs shall be used for measurement locations that are below the water line.

(b.2) LVDTs shall have the following minimum characteristics:

a.	Travel Range	0.5 in
b.	Output	4-20 mA
c.	Minimum Linearity	$\pm$0.30% full scale
d.	Min Repeatability	$\pm$0.015% full scale

(c) Temperature gage

(c.1) Temperature devices shall be resistance type and shall be sealed against moisture. Thermistors used in fabrication of sister bar gages shall have diffusivity approximately that of steel.

(c.2) Temperature sensing element shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) below. The cables shall be waterproofed and sealed as an integral part of the assembly.

(d) Pressure gage

(d.1) Pressure gages used in pressure testing shall be connected directly to the internal environment of the containment, and measure the differential pressure between the internal and external environments. This shall be accomplished either by using an absolute pressure gage inside and another absolute gage outside of the RCCV or by using a gauge pressure gage directly attached to the pressurizing pump outlet outside of the RCCV right after the shut-off valve. The pressure gages shall be voltage output (as compared to millivolt output type) with integrated signal conditioning electronics included. The pressure gages shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) above. The pressure gage cables shall be waterproofed and sealed as an integral part of the assembly.

(d.2) The pressure gages shall have the following characteristics:

a. Range	0-200 psi
b. Accuracy	±0.25 psi

(4) Cable specifications

Instrumentation cable type and size shall be shielded 16 AWG twisted paired for all instruments. The shield shall be either braided strands of copper (or other metal), a non-braided spiral winding of copper tape (or other metal), or a layer of conducting polymer. The shield shall be applied across cable splices. In addition, the cable shall have drain wire.

3.8.6.3.3 Test Acceptance Criteria:

Crack and strain (for Unit 3) measurements will be reviewed by the Designer for evaluation of the overall test results. The primary containment will be considered to have satisfied the structural integrity test if the following minimum requirements specified in CC-6410 of ASME Section III, Division 2 are met:

1. Yielding of conventional reinforcement does not develop as determined from analysis of crack width, strain, or displacement data.

2. No visible signs of permanent damage to either the concrete structure or the steel liner are detected. Evidence, resulting from the test, of spalling, laminations, or voids behind the liner are pertinent considerations. Special care shall be exercised in the post-test examinations (CC-6390) to detect evidence of localized distress which may not be revealed by strain or displacement data. The significance of such distress, if detected, must be determined by the Designer and be acceptable to the Owner.

3. Residual displacements at the point of maximum predicted radial and vertical displacement at the completion of depressurization or up to 24 hours later shall not exceed 30% of measured or predicted displacement at maximum test pressure, whichever is greater, plus 0.01in. (0.25mm) plus measurement tolerance. This

criterion shall apply to the average of radial displacements measured at the same elevation.

4. The measured displacements at test pressure at points of predicted maximum radial and vertical displacements do not exceed predicted values by more than 30% plus measurement tolerance. This criterion shall apply to the average of radial displacement measured at the same elevation. This requirement may be waived if the residual displacements within 24 hours are not greater than 20%.

If measurements and studies by the Designer indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2.