Nuclear Operating Company

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

January 17, 2011
U7-C-STP-NRC-110008

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

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

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

03.07.01-27	03.08.04-30	03.08.04-31

There are no commitments in this response.

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

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

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

Executed on 1/17/2011

M A McBurnett

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

jep

Attachments:

1. RAI 03.07.01-27, Supplement 2
2. RAI 03.08.04-30
3. RAI 03.08.04-31, Supplement 1

D091
NRO

STI 32809048

cc: w/o attachment except*
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RAI 03.07.01-27, Supplement 2

<u>QUESTION:</u>

Follow-up Question to RAI 03.07.01-19 (STP-NRC-100093)

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 the first paragraph of RAI 03.07.01-19, the applicant has presented its approach for developing the input motion for the SSI analysis and design of the DGFOSV that takes into account the impact of the nearby heavy RB and RSW Pump House structures. The applicant also stated that *"Conservatively, a 3-dimensional SAP2000 response spectrum analysis was used to obtain the safe-shutdown earthquake (SSE) design forces due to structure inertia. The seismic induced dynamic soil pressure on DGFOSV walls were computed using the method of ASCE 4-98, Subsection 3.5.3.2"* The response, however, does not provide details as to how the SSI analysis of the DGFOSV are performed and how the input motion developed are subsequently specified in the SSI analysis of DGFOSV to develop the structural response and in-structure response spectra for any equipment and subsystems within DGFOSV. From the response it appears that the applicant has not included explicitly DGFOSV structural model in the SASSI model of the RB and RSW Pump House structures to properly evaluate the SSSI effect on the DGFOSV. In order for the staff to determine if the evaluation of DGFOSV for SSE has appropriately accounted SSI effects, the applicant is requested to provide in the FSAR the following information:

    (a) Describe in detail the method used for the SSI analysis of DGFOSV including the procedures for treatment of strain dependent backfill material properties in the model, input motion used and how it is specified in the analysis, variation of soil properties, and the computer programs used for SSI analysis.

    (b) Describe in detail how SAP2000 analysis of DGFOSV was performed including, how foundation soil/backfill material was represented, how many modes were extracted, what modal damping values were used, how the input motion was specified, and what type of boundary conditions were used.

    (c) Demonstrate that the DGFOSV foundation response spectra and dynamic soil pressure (on DGFOSV basement walls using ASCE 4-98 criteria) used in the design of DGFOSV will envelop the results of structure to structure (SSSI) interaction analysis which explicitly models DGFOSV structure in the SSI model of RB and the RSW Pump House structure.

    (d) Describe in detail if there is any Category I tunnel structure for transporting Diesel Fuel Oil between DGFOSV and the Diesel Generator located in other buildings including its layout and configuration and seismic analysis and design method.

2. In the response to Item 2 of RAI 03.07.01-19, the applicant has stated that the P-wave damping ratios are assigned the same values as those calculated for the S-wave damping ratios because of the **upcoming** recommendations of ASCE 4-09 standards. It is further stated that this recommendation is based on the recent observation of earthquake data and the realization that the waves generated due to SSI effects are mainly surface and shear waves. It is noted that the NRC has not endorsed ASCE 4-09 for estimating the P-wave damping. In general, the P-wave damping is primarily associated with the site response rather than SSI effects. Because the Pwave energy for the most part will travel in water within the saturated soil media at relatively high propagation speed and is not affected by shear strains of degraded soil, the P-wave damping will be small. As such, the applicant is requested to provide quantitative assessment by performing sensitivity analysis that shows that seismic responses of Category I structures are not adversely affected to a lower P-wave damping.

SUPPLEMENTAL RESPONSE:

The response to Part 2 of this RAI was submitted with STPNOC letter U7-C-STP-NRC-100208 dated September 15, 2010. The response to Parts 1(a) through 1(c) was submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010. This supplemental response provides the response to Part 1(d).

1(d). The layout of the Diesel Generator Fuel Oil Tunnels (DGFOTs) is as shown in COLA Part 2, Tier 2 Figure 3H.6-221 provided in response to Part 1(a) of this RAI. There are three (3) reinforced concrete DGFOTs approximately 50 ft, 200 ft, and 220 ft long for each unit. Each DGFOT is connected at one end to the Reactor Building (RB) and at the other end to a Diesel Generator Fuel Oil Storage Vault (DGFOSV). There is a seismic gap between each of the tunnels and the adjoining RB or DGFOSV. For magnitude of the required and provided seismic gaps at interface of DGFOTs and the adjoining RB and DGFOSVs, see the Supplement 1 response to RAI 03.08.04-31 which is being submitted concurrently with this response.

Each DGFOT has two access regions which extend above grade; one access region is located where the tunnel interfaces with the DGFOSV and another where the tunnel interfaces with the RB. The top of the DGFOT is located at grade. The DGFOT No. 1B, which is the shortest tunnel, running approximately 50 ft between the RB and DGFOSV No. 1B, has a wall thickness of 2'-0" on both sides. The interior below grade dimensions of this tunnel are approximately 7 ft high by 3.5 ft wide. The other two longer DGFOTs (approximately 200 ft and 220 ft long) have a wall thickness of 2'-0" on one side and 2'-6" on the other side to allow for placement of embedded conduits. The interior below grade dimensions of these tunnels are approximately 7 ft high by 3 ft wide. Any fuel leak from the fuel oil lines or water infiltration within the tunnels will be collected in a sump and removed by pumps. The tunnels slope away from the DGFOSV and the RB towards the sump located at the center of the tunnel runs. The access regions provide access to the below grade portions of the DGFOTs during maintenance and inspection.

The overall above grade dimensions of the access regions are approximately 7.5 ft wide by 7.5 ft long and 15 ft high.

The details of DGFOT design are provided in the response to Part 10 of RAI 03.08.04-30 which is being submitted concurrently with this response. The following provides details of seismic analysis for DGFOTs.

Seismic Analysis for Generation of In-structure Response Spectra:

The DGFOTs are long reinforced concrete tunnels with above grade access regions at the two ends of each tunnel. The widened envelop spectra of the resulting in-structure response spectra from the following two seismic analyses are used as the final in-structure response spectra for these tunnels and their access regions.

- Two dimensional (2D) soil-structure-interaction (SSI) analysis of a typical cross section of the DGFOT
- Three dimensional (3D) fixed base seismic analysis of the DGFOT No. 1B (approximately 50 ft long) including its access regions at the two ends of the tunnel.

The details of the above two seismic analyses are provided below.

A. 2D SSI Analysis of a Typical Cross section of DGFOT

SASSI2000 computer code is used for the SSI analysis, using the direct method. Figure 3H.7-20 shows the structural part of the 2D plane-strain model of the DGFOT with 2 ft thick mud mat under the base mat. The top of the tunnel is at the grade elevation. The specifics of the 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 out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 74 ft depth (more than two times the horizontal cross section dimension of the tunnel plus its embedment depth) with halfspace below it.
- Sixteen cases of strain dependent soil properties representing the in-situ lower bound, mean and upper bound; lower bound backfill over in-situ lower bound, mean backfill over in-situ mean and upper bound backfill over in-situ upper bound; cracked concrete wall with in-situ upper bound soil, soil separation with in-situ upper bound soil; ABWR DCD/Tier 2 generic soil profiles UB1D, VP3D, VP4D, VP5D, VP7D, R , R with soil separation and R with cracked wall.

- Concrete and mud mat damping are assigned 4% for all cases (conservatively 4% damping is also used for cracked concrete cases).

- Groundwater is considered at 8 ft depth for site-specific soil and backfill cases and 2 ft depth for DCD cases. In site-specific and backfill cases, the groundwater effect is included by using minimum P-wave velocity of 5000 ft/sec with Poisson's ratio capped at 0.495. In DCD cases, the groundwater effect is included by using minimum P-wave velocity of 4800 ft/sec with Poisson's ratio capped at 0.495 (per Section 3A.3.3 of DCD, the compression wave velocity of water is 1463 m/sec, i.e. 4800 ft/sec).

- The models are capable of passing frequencies up to at least 33 Hz, in both the vertical and horizontal directions.

- For all SSI cases analyzed, a cut-off frequency of 35 Hz is used for transfer function calculations.

- Acceleration time histories consistent with Regulatory Guide 1.60 response spectra anchored at 0.3g peak ground acceleration are used as input at the grade elevation.

- Since the tunnels run along both East-West and North-South directions, the horizontal input motions from both East-West and North-South time histories are considered. East-West input motion is applied to the tunnel sections running North-South and North-South input motion is applied to the tunnel sections running East-West. The input motions consistent with RG 1.60 response spectra anchored at 0.3g peak ground acceleration envelop both the site-specific input motions and the amplified site-specific motions considering the impact of nearby heavy RB and Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) Pump House.

- In-structure response spectra are generated at the top of floor slab (middle of span), at the top of the roof slab (middle of span) and at the mid-height of two walls of the tunnel cross-section.

- The responses from the horizontal and vertical directions are combined using the square-root-of-sum-of-square (SRSS) method.

- The responses from all SSI analyses cases are enveloped.

- The in-structure response spectra at the top of the floor slab (middle of span), at the roof of slab (middle of span) and at the mid-height of two walls of the tunnel cross-section are enveloped to conservatively provide the in-structure response spectra for the entire 2D cross-section of the tunnel.

In response to an action item from the NRC's audit performed during the week of October 18, 2010, the following additional information is also included:

The foundation input response spectra (FIRS) for the DGFOT were calculated and were compared to the outcrop spectra at the foundation level of the DGFOT. The outcrop spectra were calculated from a deconvolution analysis performed in the SHAKE program with the site-specific SSE motion applied at the free field ground surface. Figures 3H.7-22 through 3H.7-32 show the comparison of the outcrop response spectra and the FIRS, in the two horizontal directions and the vertical direction for the lower bound, mean and upper bound in-situ soil properties. These figures show that the FIRS are enveloped by the foundation outcrop spectra in all cases. The figures also show that the response spectra at the SHAKE outcrop of DGFOT foundation level also envelop a broad band spectrum anchored at 0.1g. This is the minimum requirement as stated in SRP 3.7.1 and Appendix S to 10 CFR 50. The broadband spectrum used in this comparison is conservatively defined as the Regulatory Guide 1.60 spectrum anchored at 0.1g.

B. 3D Fixed Base Analysis of DGFOT No. 1B Including its Two Access Regions

A 3D fixed base seismic (basemat fixed) analysis of DGFOT No. 1B running between the RB and DGFOSV No. 1B is performed. The following provides the details of this fixed base analysis:

- SAP2000 computer code is used to perform the seismic analysis.
- Modal time history method of analysis is used.
- Shell elements are used for modeling the reinforced concrete tunnel section and the access regions at the two ends of the tunnel.
- 4% damping is used for the shell elements.
- Acceleration time histories (two horizontal directions and a vertical direction) consistent with Regulatory Guide 1.60 response spectra anchored at 0.3g peak ground acceleration are used as input motions.
- Nodal acceleration time history responses obtained from the SAP2000 analysis are processed using the RSG computer code to calculate in-structure response spectra at selected nodes. The nodes selected for the in-structure response spectra generation are; four nodes on top of each access regions (middle of four walls) and three nodes at the top of tunnel (middle of the tunnel).
- The maximum co-directional responses from each of the three directions of excitations are combined using the SRSS method.

- The in-structure response spectra at the selected nodes are enveloped to conservatively provide the in-structure response spectra from fixed base analysis, for the entire tunnel and the access regions.

The corresponding in-structure response spectra obtained from the 2D SSI analysis and in-structure response spectra obtained from the 3D fixed base analysis described in parts A and B above are enveloped and peak widened by ± 30%. The 30% peak widening is used to cover any frequency shift due to the foundation soil flexibility, which is not included in the fixed base seismic analysis. The final widened in-structure response spectra for the horizontal and vertical directions of the DGFOTs and their access regions are provided in Figures 3H.7-31 and 3H.7-32, respectively. The spectra in Figures 3H.7-31 and 3H.7-32 provide the in-structure response spectra for the entire SGFOTs and their access towers at the two ends.

<u>Structure-Soil-Structure Interaction (SSSI) Analysis to Obtain Seismic Soil Pressures:</u>

Two 2D section cuts are taken for site-specific SSSI analyses; one East-West section cut through DGFOT No. 1C, DGFOSV No. 1A and the Crane Foundation Retaining Wall (CFRW) and one East-West section cut through the RB, DGFOT No. 1A and the CFRW. These SSSI analyses are used to obtain seismic soil pressures on the walls of DGFOT considering the effect of nearby structures.

The SSSI model and analyses details for the section cut through DGFOT No. 1C, DGFOSV No. 1A and the CFRW have been provided in the response to Part 1(a) of this RAI which was submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010.

The structural part of the SSSI model for the section cut through the RB, DGFOT No. 1A and the CFRW is shown in Figure 3H.7-21. The methodology for the SSSI model including strain dependent soil properties; soil cases analyzed; and method of analyses are the same as those for the section cut through DGFOT No. 1C, DGFOSV No. 1A and the CFRW described in the response to Part 1(a) of this RAI. This SSSI model is capable of passing frequencies up to at least 33 Hz in both the vertical and horizontal directions and the analysis uses a cut-off frequency of 33 Hz for calculation of transfer functions.

Figures 3H.7-5 through 3H.7-8 show a comparison of the SSI, SSSI, ASCE 4-98 seismic soil pressures and the enveloping seismic soil pressures used for the design of the DGFOT walls.

The design of the DGFOTs also accounts for the axial tensile strain and the seismic induced forces at the tunnel bends due to SSE wave propagation. For more information on this subject, see the response to Part 10 of RAI 03.08.04-30 which is being submitted concurrently with this response.

Revision 4 of COLA Part 2, Tier 2 Section 3H will be revised as shown in Enclosure 3 of the response to RAI 03.08.04-30 which is being submitted concurrently with this response.

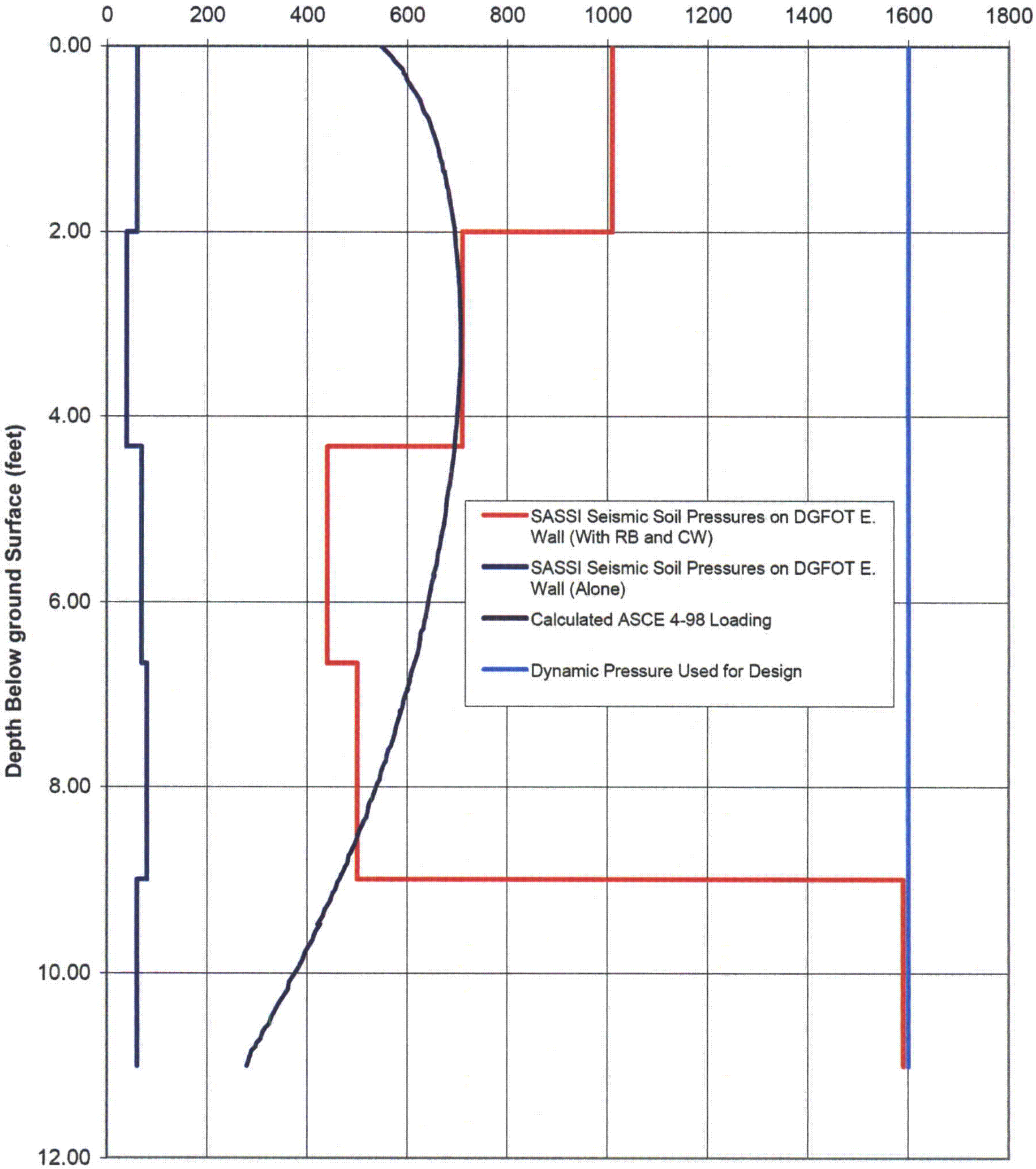


Figure 3H.7-5: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel East Wall with Reactor Building and Crane Wall

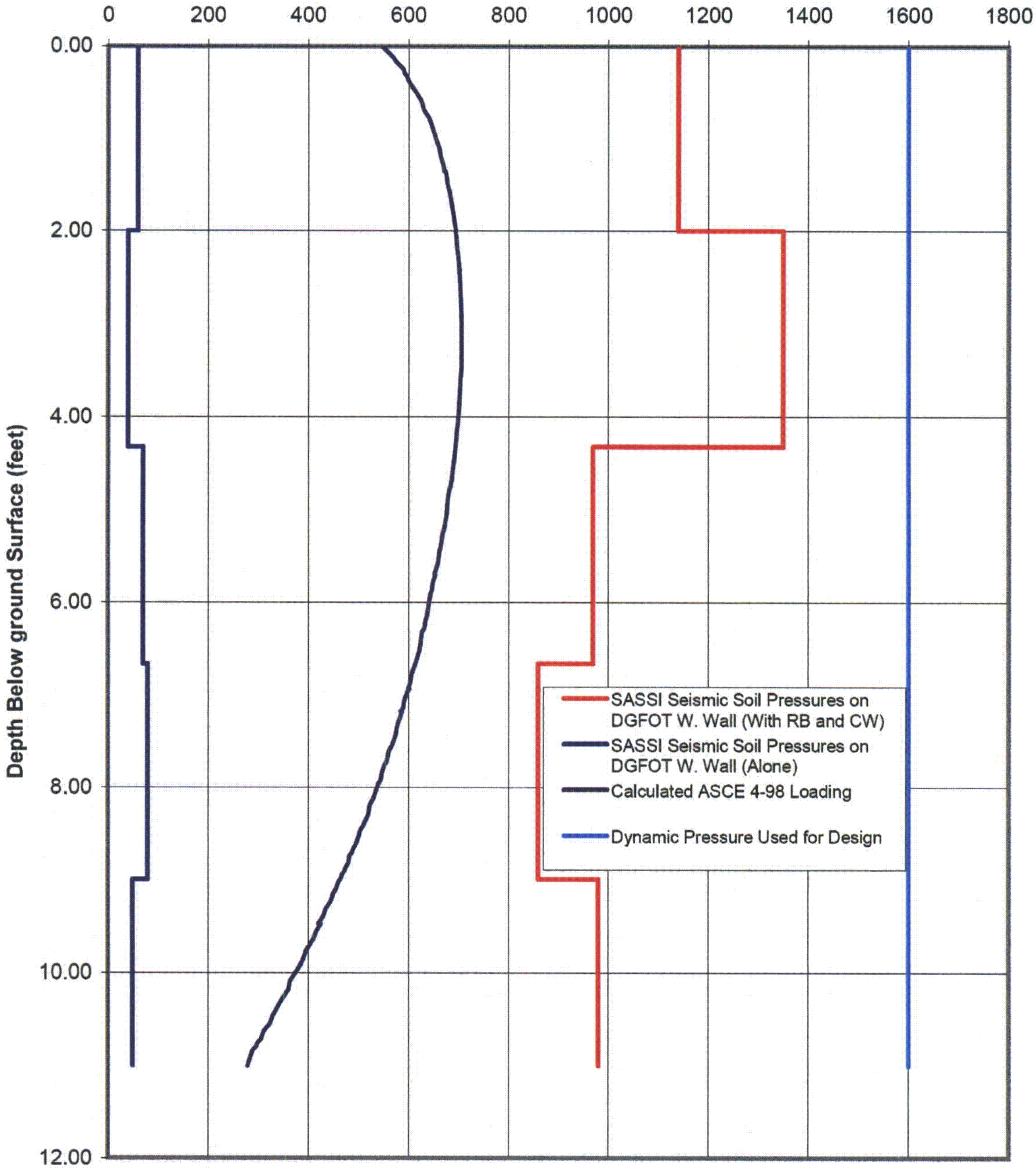


Figure 3H.7-6: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel West Wall with Reactor Building and Crane Wall

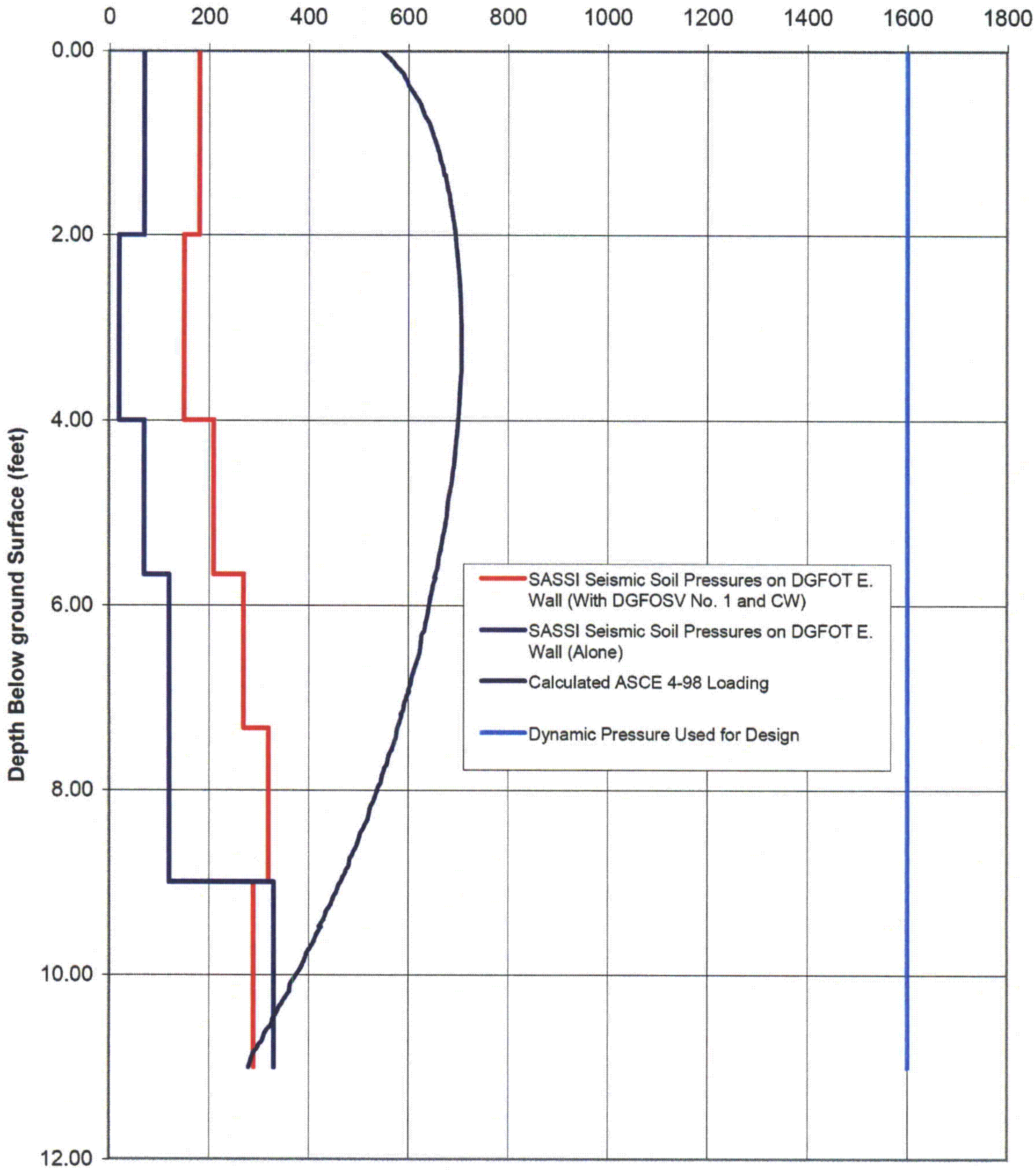


Figure 3H.7-7: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel East Wall with Diesel Generator Fuel Oil Storage Vault and Crane Wall

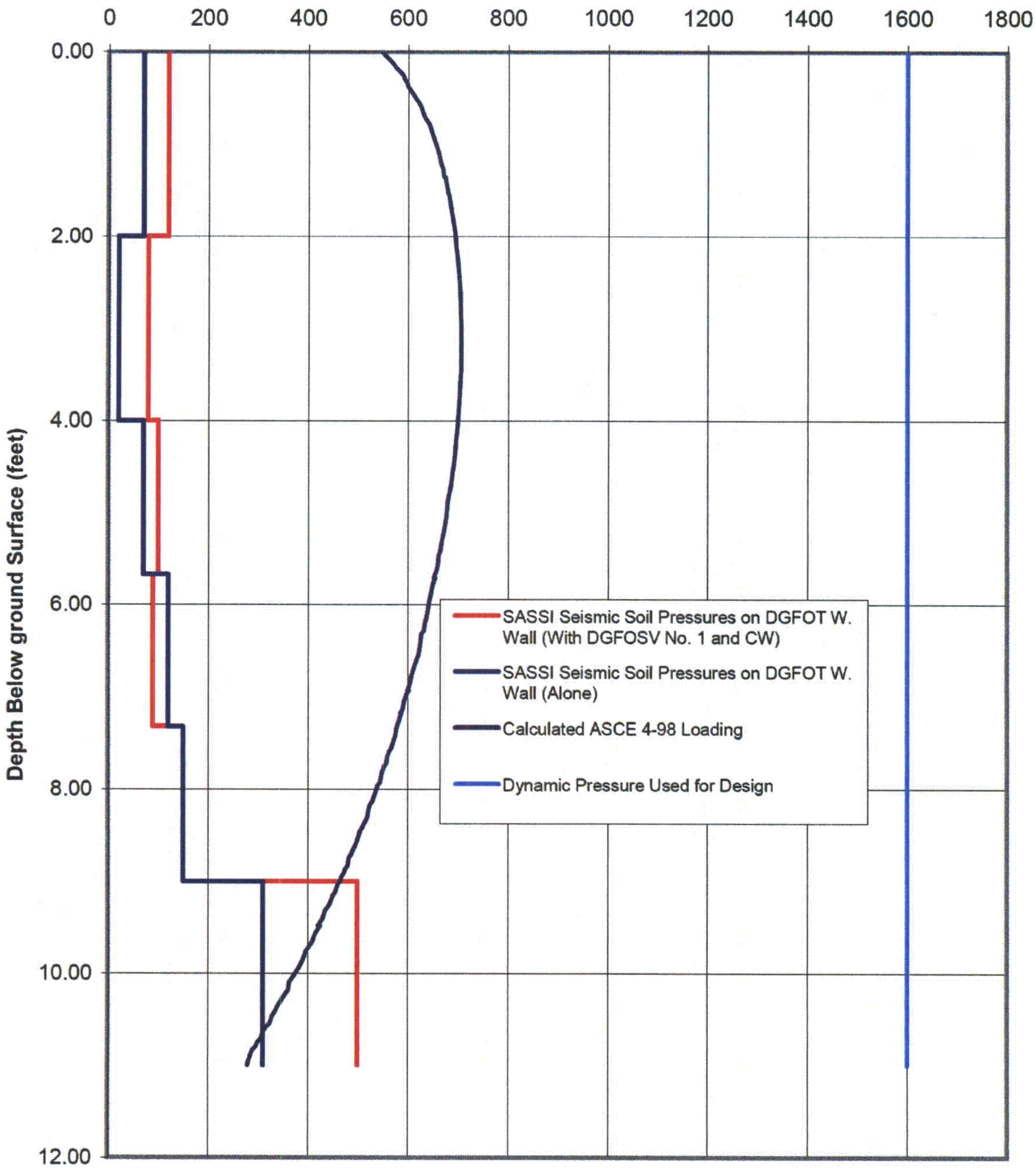


Figure 3H.7-8: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel West Wall with Diesel Generator Fuel Oil Storage Vault and Crane Wall

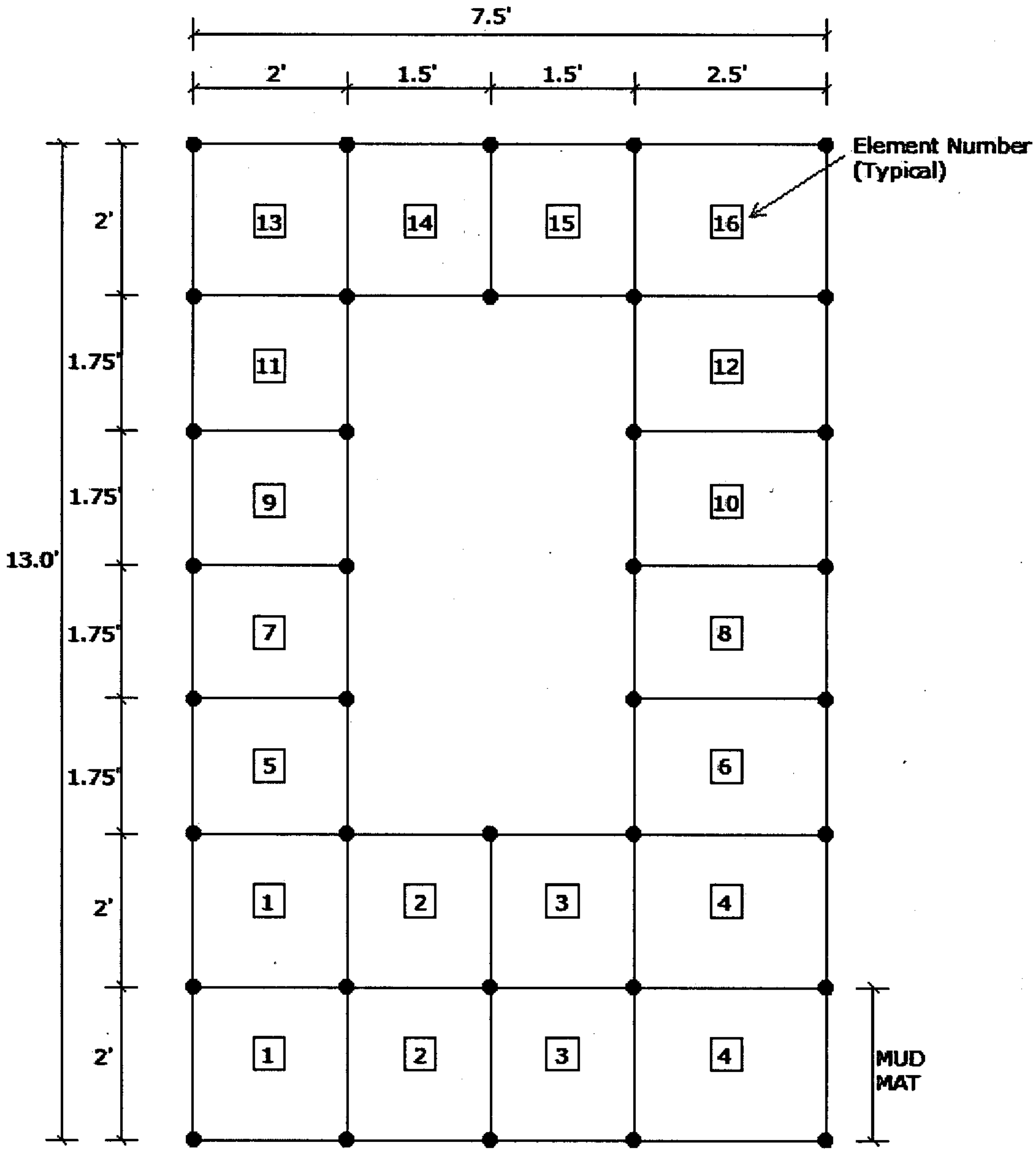


Figure 3H.7-20: 2D Model for SSI Analysis of a Typical Cross section of DGFOT

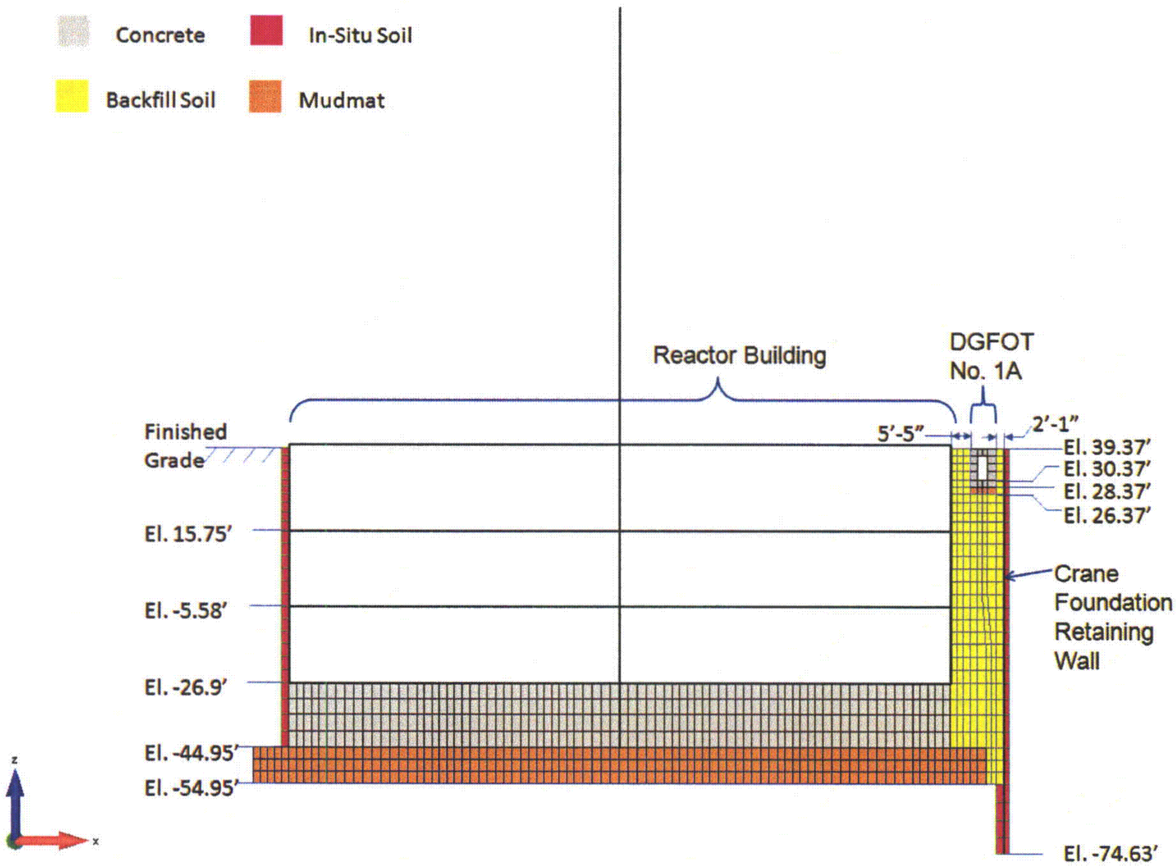


Figure 3H.7-21: 2D SSSI Model of RB, DGFOT and Crane Foundation Retaining Wall

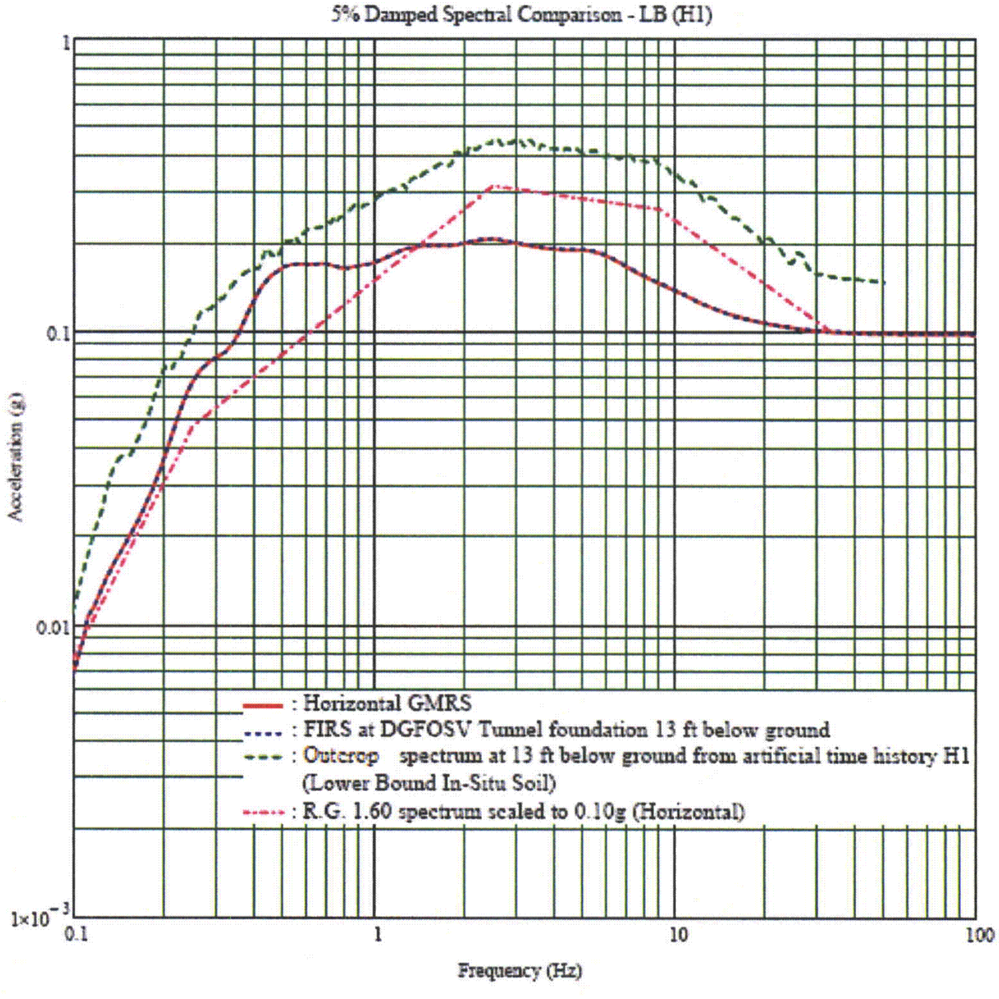


Figure 3H.7-22: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Horizontal X Direction

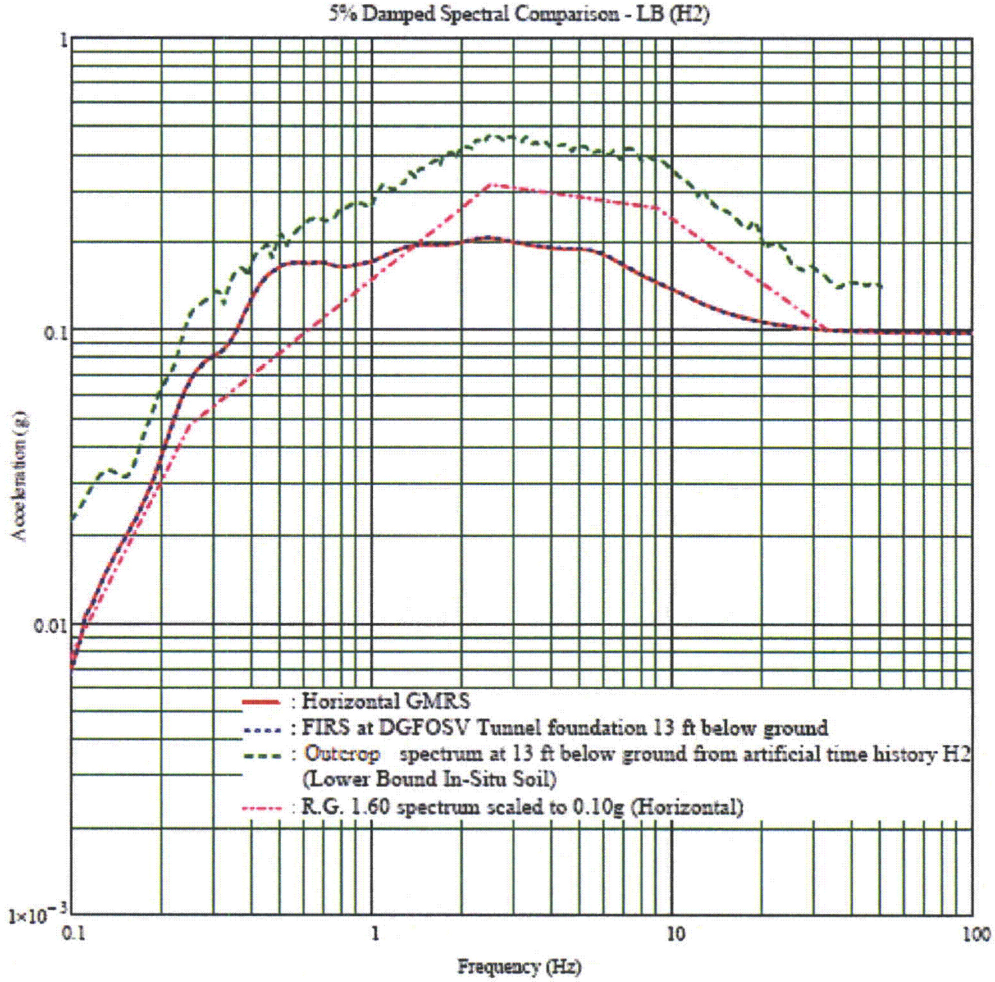


Figure 3H.7-23: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Horizontal Y Direction

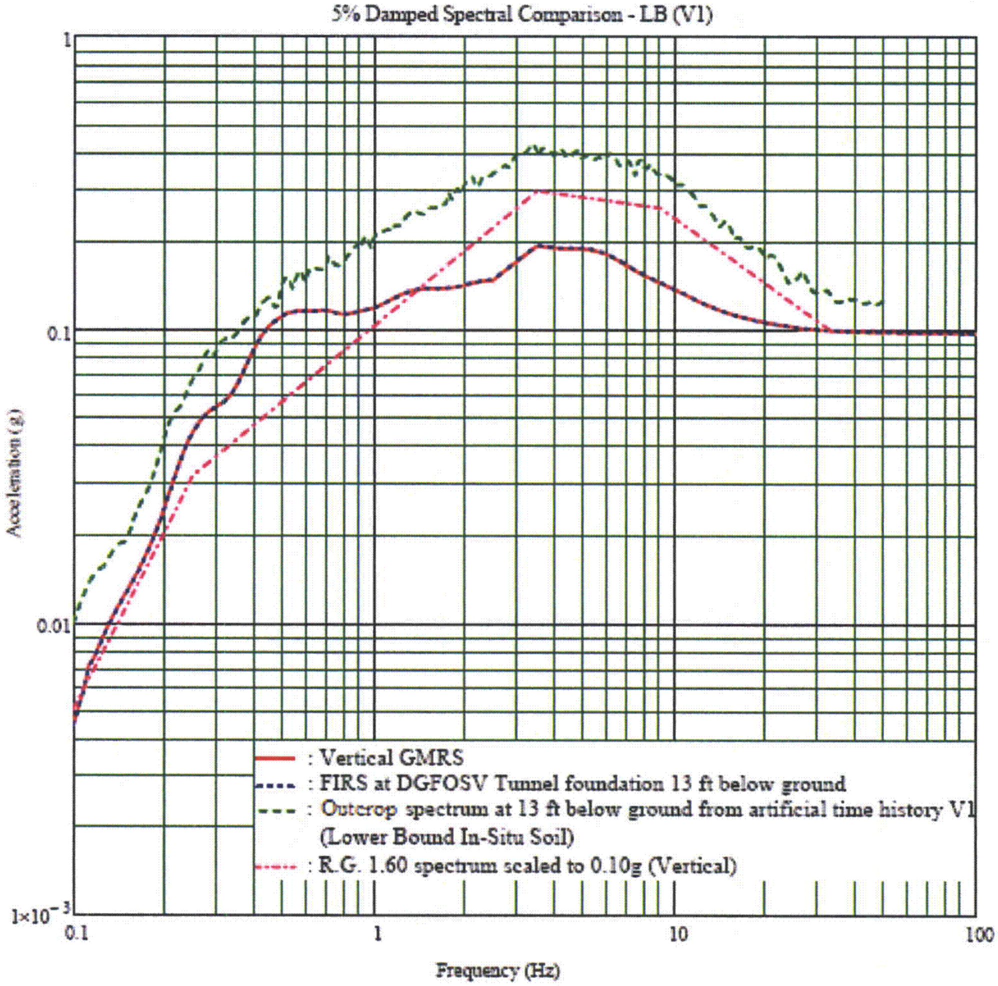


Figure 3H.7-24: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Vertical Direction

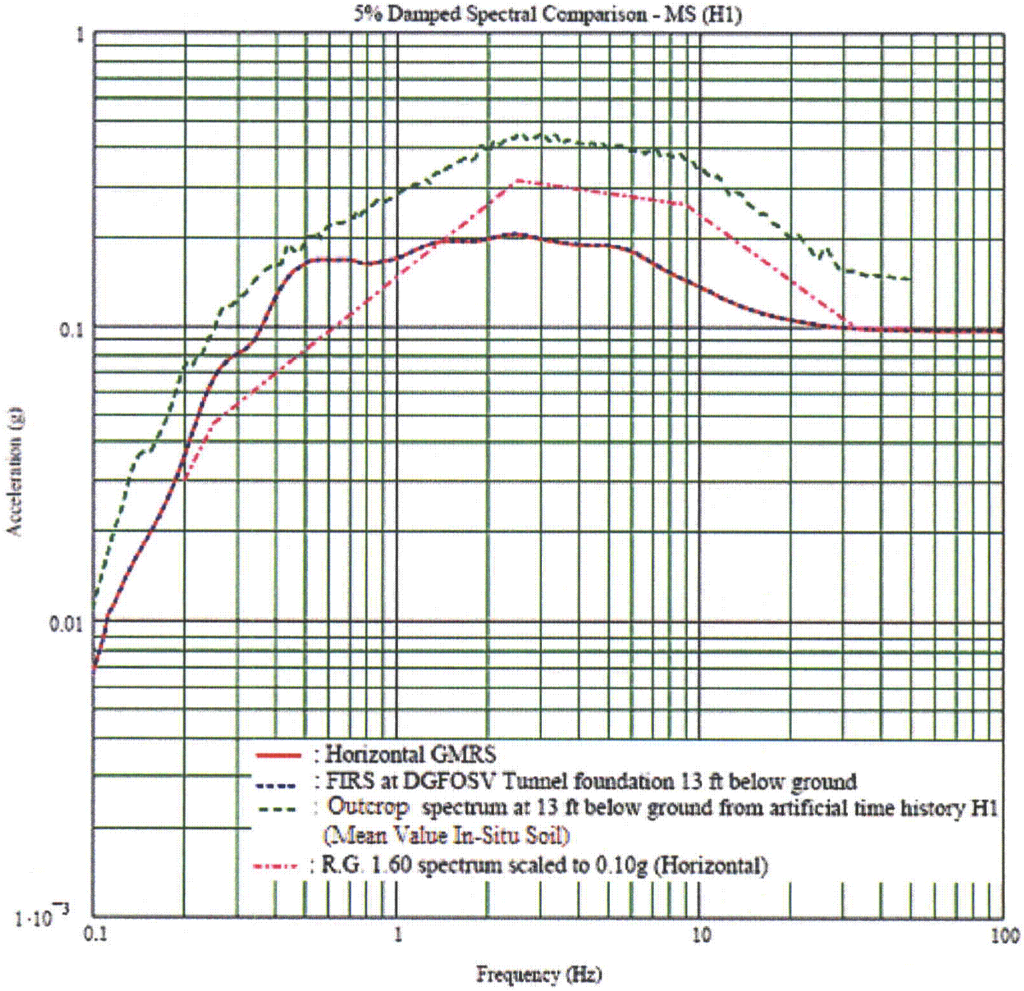


Figure 3H.7-25: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Horizontal X Direction

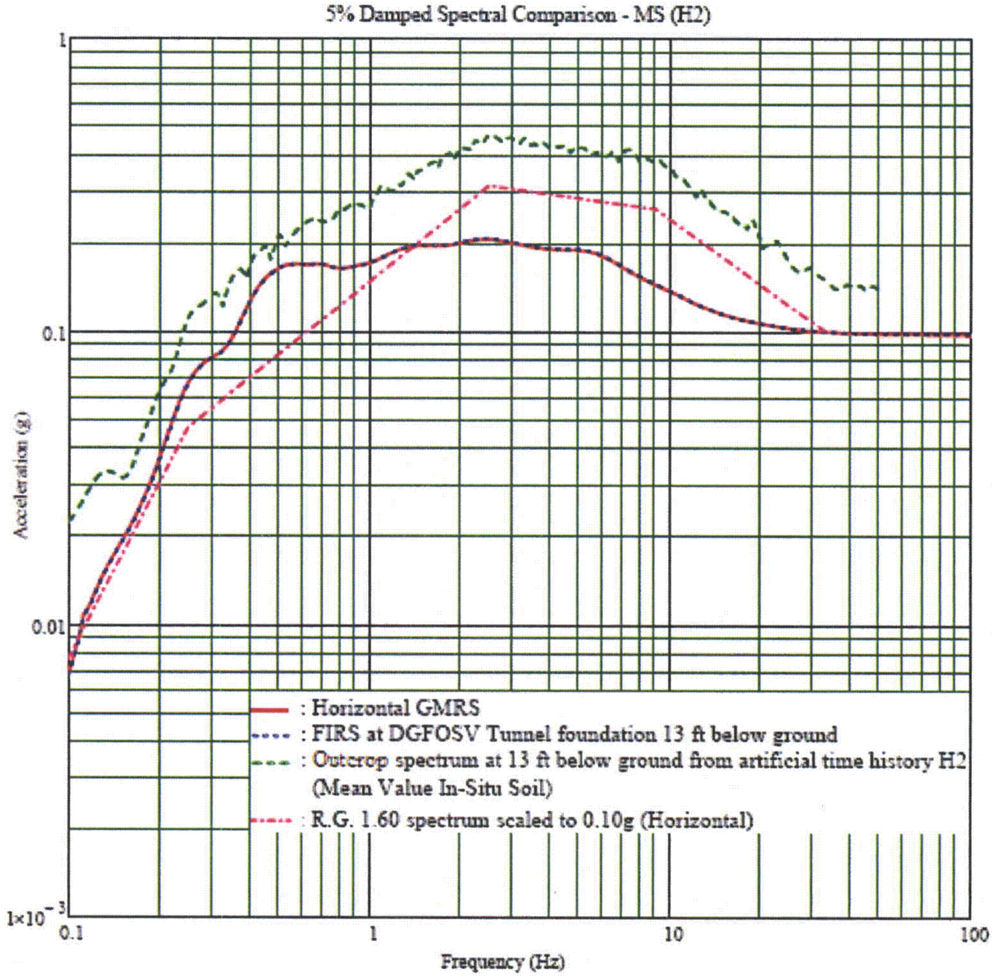


Figure 3H.7-26: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Horizontal Y Direction

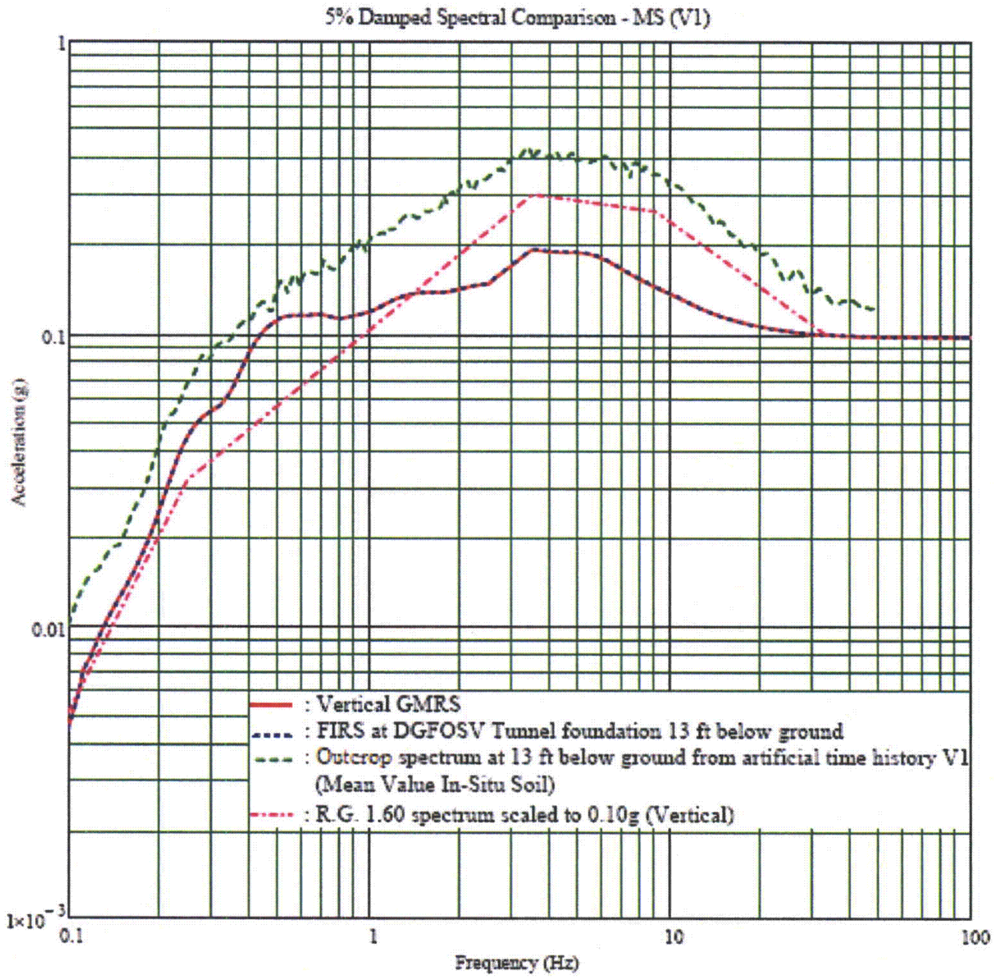


Figure 3H.7-27: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Vertical Direction

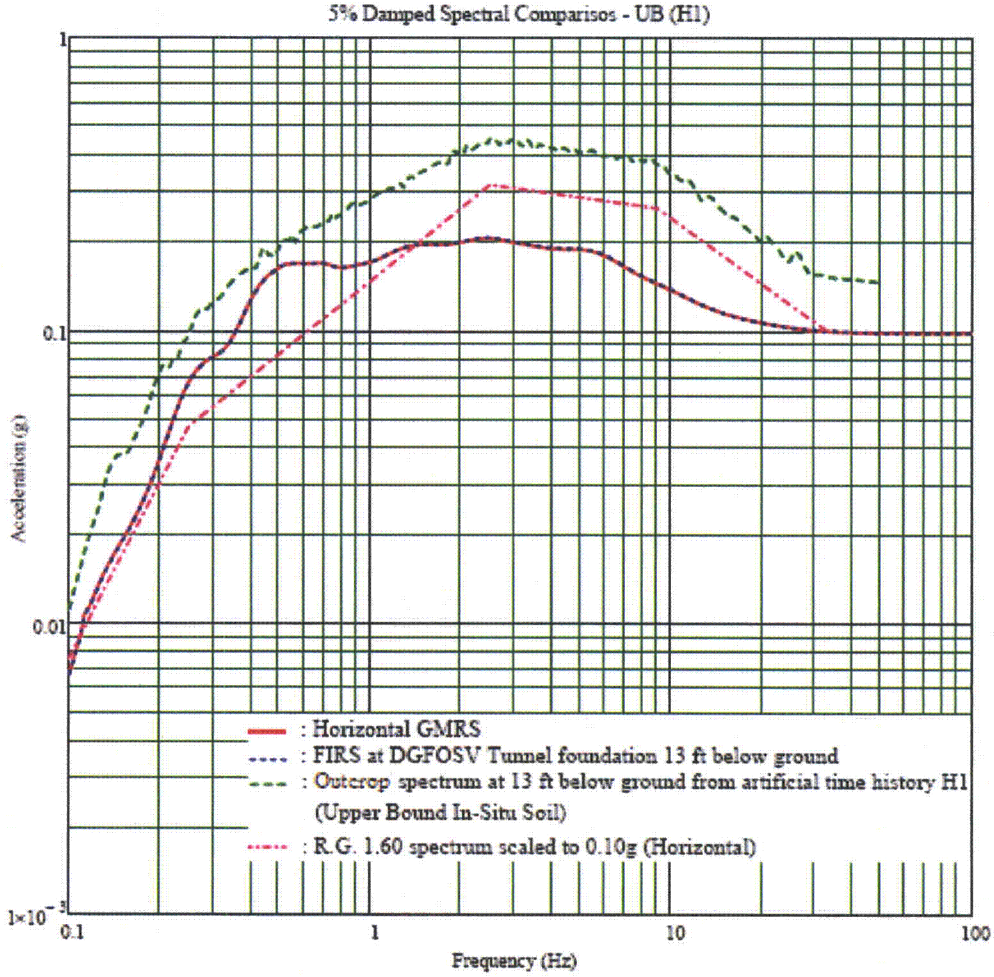


Figure 3H.7-28: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Horizontal X Direction

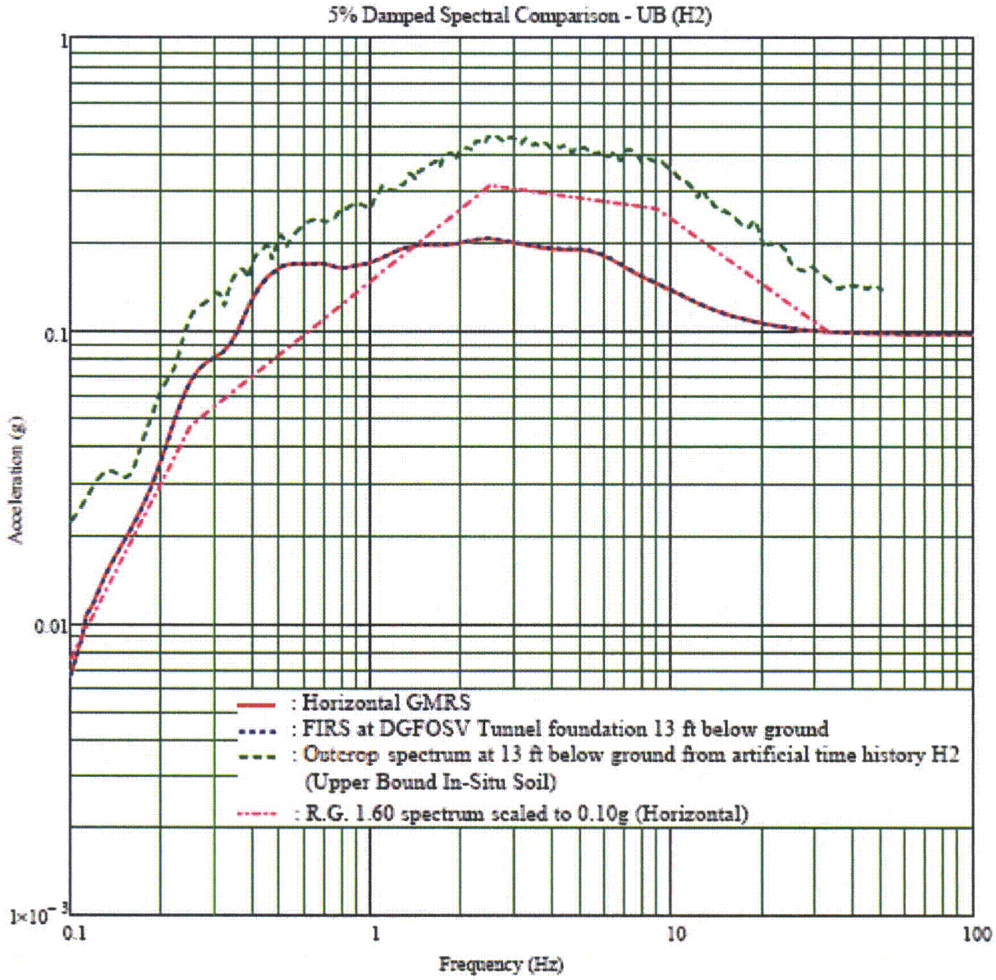


Figure 3H.7-29: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Horizontal Y Direction

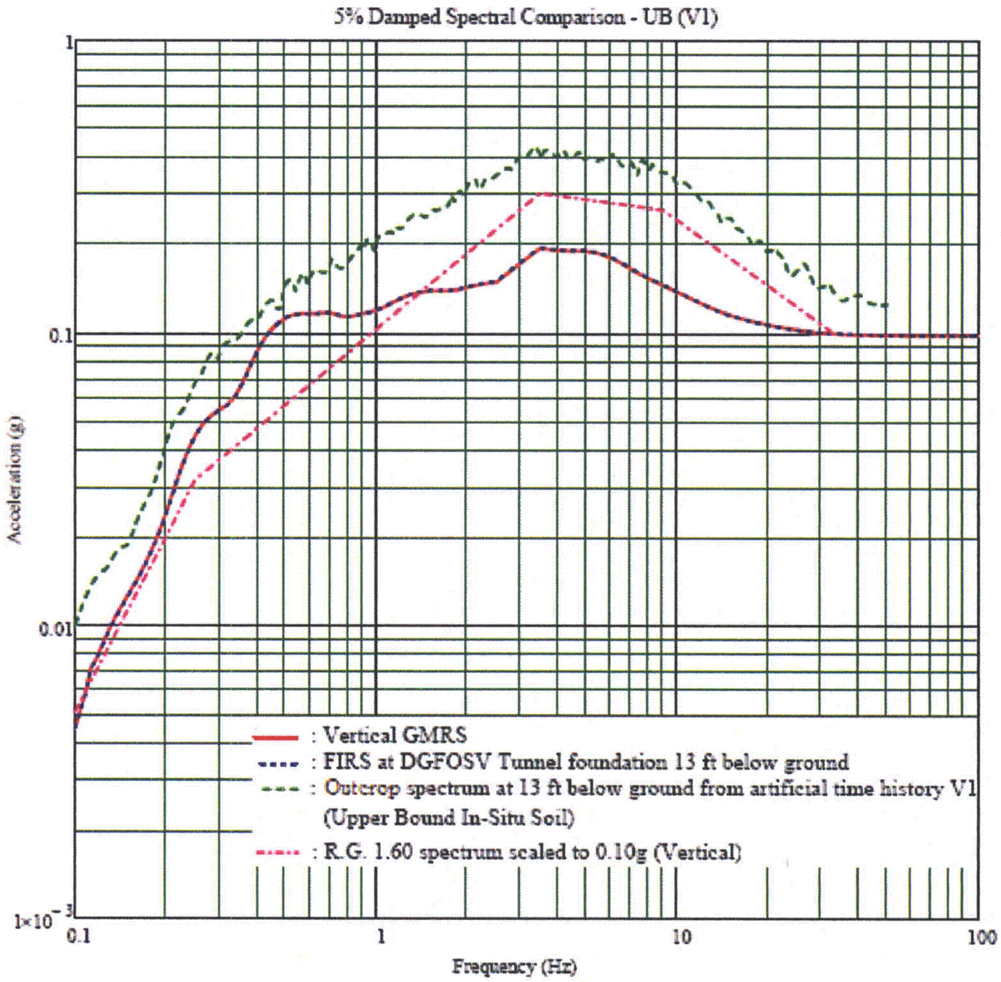


Figure 3H.7-30: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Vertical Direction

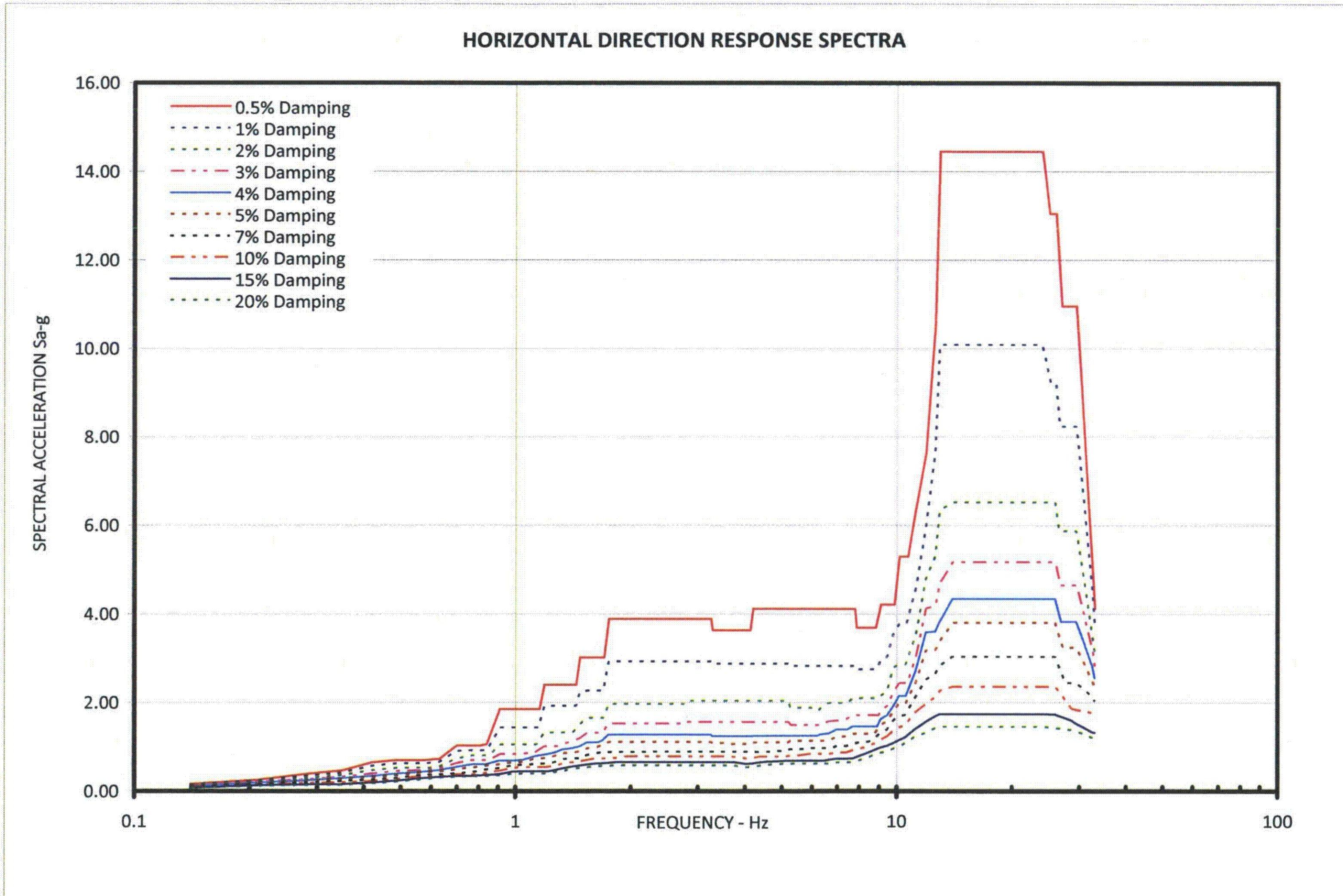


Figure 3H.7-31: Enveloped, Broadened Horizontal Response Spectra for DGFOTs

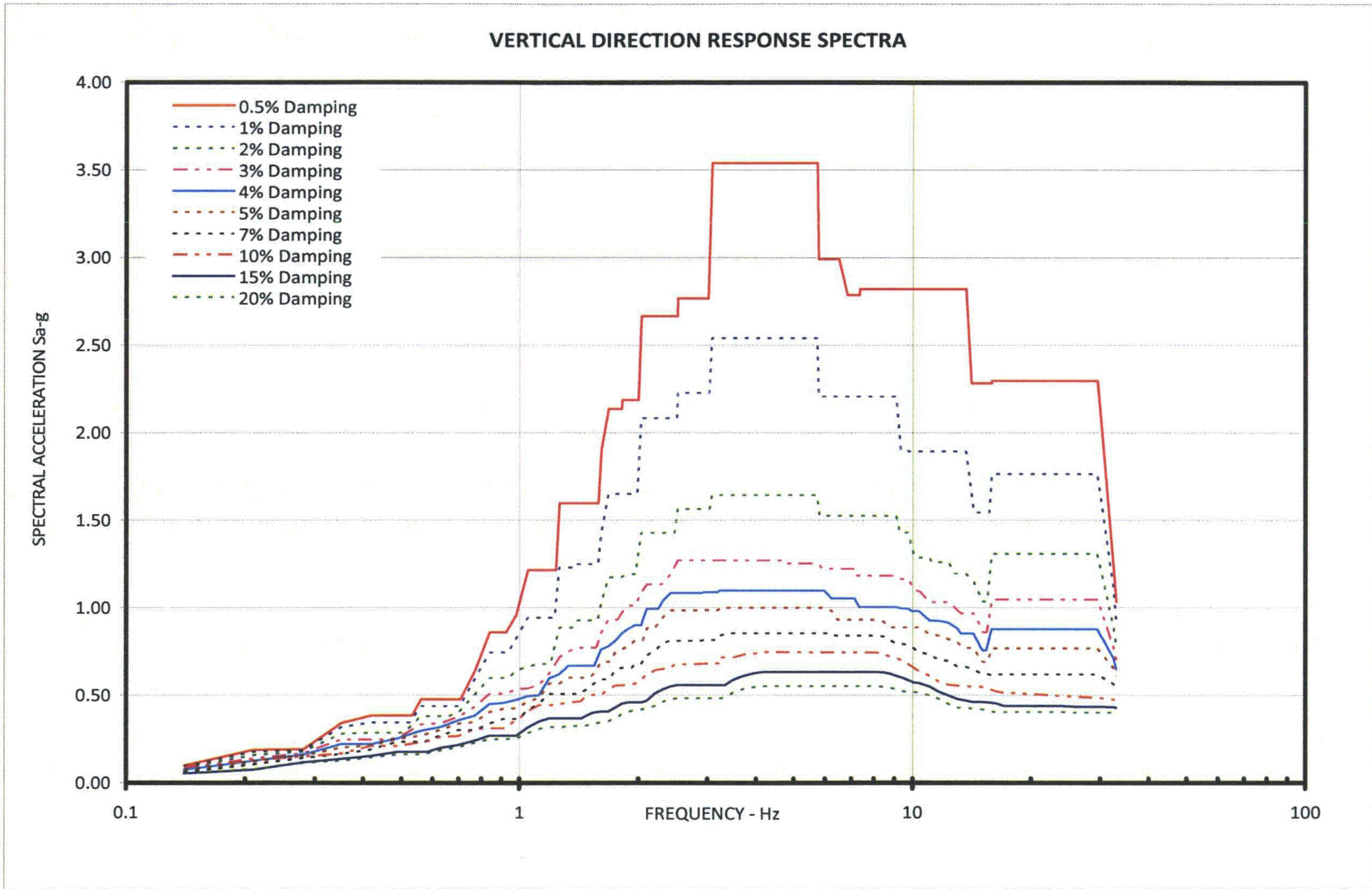


Figure 3H.7-32: Enveloped, Broadened Vertical Response Spectra for DGFOTs

RAI 03.08.04-30

QUESTION:

Follow-up to Question 03.08.04-23

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

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

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

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

justification for not considering the effects of pressure distribution and system frequency in developing the foundation dynamic springs including describing the impact on the calculated results.

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

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

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

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

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

applicant is requested to provide complete structural analysis and design information for the DGFOSV to ensure it meets acceptance criteria 1 through 7 of SRP 3.8.4 and 3.8.5. The staff needs this information to conclude that the DGFOSV is designed to withstand seismic loads and meet GDC 2. Include in the response an updated version of Appendix 3H where structural analysis and design information for all seismic category I structures can be found.

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

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

RESPONSE:

The response to Parts 1 through 7 of this RAI is currently scheduled to be submitted by January 31, 2011. The following is the response to Parts 8 through 10. In addition the following COLA mark-ups are provided based on discussions during the NRC audit performed during the week of October 18, 2010.

- Mark-up for Section 3.7.2.8 provides a summary of the seismic input motion used for seismic II/I evaluation of Non-Seismic Category I structures.
- Mark-up for Section 3.8.6.1 clarifies that the minimum required coefficient of friction for waterproof membrane is determined based on sliding stability of the structure considering the site-specific SSE motion.

8. The following response is broken into sub-sections a through g to address the design of the Diesel Generator Fuel Oil Storage Vaults (DGFOSV).

   a) Soil-Structure Interaction Analyses:

   The structural analysis and design of the Diesel Generator Fuel Oil Storage Vault (DGFOSV) described in the response to RAI 03.07.01-19, Revision 2 (submitted with letter U7-C-STP-NRC-100129, dated June 7, 2010), has been revised.

The revised soil-structure interaction (SSI) analyses for generation of in-structure response spectra considering both full and empty fuel oil tanks, and the two (2) new two-dimensional (2D) structure-soil-structure interaction (SSSI) analyses for obtaining seismic soil pressures were provided in the response to RAI 03.07.01-27, Supplement 1, submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010.

b) Equivalent Static Method Used for Design:

The design of the DGFOSVs has been revised. In the revised design, the seismic loads are conservatively determined using the equivalent static method described below.

The structural analysis and design of the Diesel Generator Fuel Oil Storage Vault (DGFOSV) was performed using a finite element analysis (FEA). The finite element model (FEM) for this FEA is shown in COLA Part 2, Tier 2 Figure 3H.6-140. The maximum nodal accelerations from the SSI analysis in the X, Y, and Z direction for the subgrade and above grade roofs were averaged and used as the accelerations in the X, Y, and Z directions for the entire structure to obtain the equivalent static seismic loads. The induced forces due to the X, Y, and Z seismic excitations were combined using the square-root-sum-of-squares (SRSS) method.

In order to demonstrate that the above equivalent static method is conservative, the seismic in-plane shear forces, axial forces and in-plane moments for the shear walls of this structure from the equivalent static method and those from the SSI analyses were compared at a section cut just above the basemat (see Figure 03.08.04-30.1 for location of this section cut). Tables 03.08.04-30.1 and 03.08.04-30.2 provide the results of this comparison. As seen from these tables, the use of equivalent static method for determination of seismic loads yields seismic loads in excess of those from the SSI analyses. Thus, use of the equivalent static method is conservative.

The design of the DGFOSV meets Acceptance Criteria 1 through 7 of Standard Review Plans 3.8.4, Revision 2, and 3.8.5, Revision 2, as noted in the referenced sections of COLA Part 2, Tier 2 and the COLA mark-ups provided in Enclosure 3 of this response as described below:

1. Description of the Structures and Foundation: Refer to COLA Part 2, Tier 2, Section 3H.6.7 and Section 3H.6.7.3 provided in Enclosure 3.
2. Applicable Codes, Standards and Specifications: Refer to Section 3H.6.7.1 provided in Enclosure 3.
3. Loads and Load Combinations: Refer to Section 3H.6.7.1 provided in Enclosure 3.
4. Design and Analysis Procedures: Refer to Section 3H.6.7.2 provided in Enclosure 3.
5. Structural Acceptance Criteria: Refer to COLA Part 2, Tier 2, Section 3H.6.4.3.4.
6. Materials, Quality Control and Special Construction Techniques: Refer to Section 3H.6.7.1 provided in Enclosure 3 and COLA Part 2, Tier 2, Section 3H.5.6.
7. Testing and Inservice Surveillance Requirements: Testing and inservice surveillance requirements are not applicable to the DGFOSVs.

c) Design Loads and Load Combinations:

The loads and load combinations used for the design of DGFOSVs are in accordance with those described in Revision 4 of COLA Part 2, Tier 2 Section 3H.6.4.3.

d) Foundation and Soil Springs

The foundation for the DGFOSV consists of a reinforced concrete mat and a lean concrete mud mat. The basemat deflections due to the flexibility of the basemat and supporting soil were accounted for through the use of foundation soil springs in the SAP2000 FEA models. Both the Winkler Method and the Pseudo-Coupled Method were used to model the foundation soil springs, and the results of the two analyses were enveloped for design purposes. Additional information on these two methods for modeling of the foundation soil springs is provided in the response to RAI 03.08.05-4, Supplement 1, submitted with STPNOC letter U7-C-STP-NRC-100248, dated November 17, 2010.

In addition, two different subgrade reactions (soil spring constants) are used, one for seismic loads and one for non-seismic loads. The following soil spring constants were used in the FEA models of the DGFOSVs:

Vertical springs (with static loads)..60 kips/ft/ft^2
Vertical springs (with seismic loads)..314 kips/ft/ft^2
North-south springs (with static and seismic loads)...229 kips/ft/ft^2
East-west springs (with static and seismic loads)..213 kips/ft/ft^2

e) Uplift

The SAP2000 finite element models were checked for uplift effects by reviewing the joint reaction at the basemat. It was determined that under seismic loading the DGFOSV experiences uplift. Using the 100%, 40%, 40% rule for combination of three seismic excitations, non-linear analysis was run on each model with uniform Winkler soil springs and pseudo-coupled soil springs to determine an enveloping adjustment factor for forces and moments from the linear analysis for the foundation mat and the connecting walls. The non-linear analysis iterates multiple times removing soil springs that go into tension during each iteration until no soil springs are in tension. For the directional earthquake loading required for the nonlinear analysis, the DGFOSV critical loading, a safe shutdown earthquake (SSE) from the southwest in combination with static active and passive loads for SSE, is considered (See Figure 03.08.04-30.2 for a schematic view of the directional seismic load).

Comparing resultant foundation mat and wall reactions from the linear analysis with mat and wall reactions from the nonlinear analysis, there is a maximum reaction increase of approximately 67% for the foundation mat shear and axial forces, 17% increase for the foundation mat bending moments, and 6% increase for the connecting walls shear forces, axial forces, and bending moments (enveloping cases with Winkler and pseudo-coupled soil springs) in the nonlinear analysis. To account for this, the resulting forces and moments from the linear analyses were adjusted by applying an increase factor of 1.67 to all forces in the foundation mat, an increase factor of 1.17 to all moments in the foundation mat, and an increase factor 1.06 to all forces and moments in the connecting walls for the DGFOSV design.

f) Stability

Detailed stability evaluations were performed for sliding, overturning, and flotation as described in response to RAI 03.07.01-19, Revision 2 (letter U7-C-STP-NRC-100129, dated June 7, 2010). For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for consideration of the X, Y, and Z seismic excitations. Since the orientation of the DGFOSVs in the horizontal plane can be along the East-West or North-South axes, the horizontal seismic values used in the stability calculation envelope the SSI accelerations in the X and Y directions. The stability safety factors considering the revised SSI analyses are provided in Table 3H.6-12 (see Enclosure 3).

g) Design Results:

The strength design criteria of ACI 349-97, as supplemented by RG 1.142, were used for the design of the reinforced concrete elements of the DGFOSV. Concrete with minimum compressive strength of 4.0 ksi (27.6 MPa) and reinforcing steel with yield strength of 60 ksi (414 MPa) are considered in the design.

Due to difference in soil spring constants for seismic and non-seismic loads, the FEA analyses for the non-seismic loads and equivalent static seismic loads were run on different FEA models and the results from these models were combined and adjusted per paragraph (e) above outside the SAP2000 model to obtain the combined total design forces and moments for the seismic load combinations.

The revised design forces and provided reinforcement for the DGFOSV walls and slabs are shown in Table 3H.6-11 included in Enclosure 2. Each face and each direction of each wall and slab has a corresponding longitudinal reinforcement zone figure. Each wall and slab also has a corresponding transverse shear reinforcement zone figure where transverse shear reinforcement is required. The reinforcement zone figures (Figure 3H.6-142 through 3H.6-208 included in Enclosure 2) show the various zones used to define the provided reinforcement based on the finite element analysis results. Actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The shell forces from every element for every load combination in the finite element analysis were evaluated to determine the provided reinforcement in each reinforcement zone. For each reinforcement zone, the following out-of-plane moment and axial force coupled with the corresponding load combination are reported in Table 3H.6-11 (see Enclosure 3):

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

For each reinforcement zone, the following in-plane and transverse shears with the corresponding load combination are reported in Table 3H.6-11 (see Enclosure 3):

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

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

The provided transverse shear reinforcing (as required) is determined based on the transverse shears and axial forces perpendicular to the shear plane occurring simultaneously for every load combination.

The DGFOSV below grade roof was designed with composite steel beams and concrete slabs for vertical loading. The composite beams span in the SAP2000 model Y-direction with the concrete slab designed as spanning one-way between the composite beams. The below grade roof slab acts as a diaphragm to transfer lateral loads. The provided reinforcing for the below grade roof slab is reported in Table 3H.6-11 (see Enclosure 3).

A Reviewers Guide for Section 3H will be made available upon completion of the changes affecting this section.

9. Sliding Stability Evaluations

Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) Pump House:

The sliding stability of the UHS/RSW Pump House against sliding, overturning and flotation was re-evaluated considering the latest SSI analyses described in the response to RAI 03.07.02-24, Supplement 2, submitted with STPNOC letter U7-C-STP-NRC-100268, dated December 14, 2010 considering both full and empty basin conditions. The UHS/RSW Pump House considering a full basin condition was found to be stable against sliding without utilizing any passive pressure. The UHS/RSW Pump House considering an empty basin condition was found to be stable against sliding by engaging some passive pressure. The at-rest coefficients of friction considered for these stability evaluations were 0.3 for RSW Pump House and 0.4 for UHS Basin. The available at-rest (static) coefficient of friction based on tangent of the soil friction angle (phi) is 0.70.

DGFOSV:

The DGFOSV sliding stability evaluation was based on mobilization of passive pressure using a sliding coefficient of friction of 0.39, which is equal to two-thirds of the minimum available at-rest (static) coefficient of friction of 0.58. The available at-rest (static) coefficient of friction of 0.58 is based on tangent of the soil friction angle (φ).

Control Building:

The Control building stability evaluation is based on mobilization of passive pressure using a sliding coefficient of friction of 0.47, which is equal to two-thirds of the minimum available at-rest (static) coefficient of friction of 0.70. The available at-rest (static) coefficient of friction of 0.70 is based on tangent of the soil friction angle (φ).

Reactor Building:

Please see the response to part 4 of RAI 03.08.04-28 submitted with STPNOC letter U7-C-STP-NRC-100208, dated September 15, 2010.

10. The layout of the Diesel Generator Fuel Oil Tunnels (DGFOTs) is as shown in COLA Part 2, Tier 2 Figure 3H.6-221 provided in response to Part 1(a) of RAI 03.07.01-27, Supplement 1, submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010. There are three (3) reinforced concrete DGFOTs approximately 50 ft, 200 ft, and 220 ft long for each unit. Each DGFOT is connected at one end to the Reactor Building (RB) and at the other end to a Diesel Generator Fuel Oil Storage Vault (DGFOSV). There is a seismic gap between each of the DGFOT and the adjoining RB and DGFOSV. For magnitude of the required and provided seismic gaps at interface of DGFOTs and the adjoining RB and DGFOSVs, see the Supplement 1 response to RAI 03.08.04-31 which is being submitted concurrently with this response.

    Each DGFOT has two access regions which extend above grade; one access region is located where the tunnel interfaces with the DGFOSV and another where the tunnel interfaces with the RB. The top of the DGFOT is located at grade. Any fuel leak from the fuel oil lines or water infiltration within the tunnels will be collected in a sump and removed by pumps. The access regions provide access to the below grade portions of the DGFOTs during maintenance and inspection. The overall above grade dimensions of the access regions are approximately 7.5 ft wide by 7.5 ft long and 15 ft high.

For details of the soil-structure interaction (SSI) analysis for generation of in-structure response spectra and structure-soil-structure (SSSI) analysis for determination of seismic soil pressures, see the response to RAI 03.07.01-27, Supplement 2 which is being submitted concurrently with this response.

The DGFOTs are Seismic Category I structures. The structural analysis and design of the DGFOT is performed using a three-dimensional (3D) SAP 2000 finite element analysis (FEA) with shell elements representing the walls, slabs and mat. The foundation soil is represented by vertical and horizontal springs. The FEA finite element model (FEM) is shown in Figure 3H.7-1 (see Enclosure 4).

The DGFOT No. 1B, which is the shortest tunnel, running approximately 50 ft between the RB and DGFOSV No. 1B, has a wall thickness of 2'-0" on both sides. The interior below grade dimensions of this tunnel are approximately 7 ft high by 3.5 ft wide. The other two longer DGFOTs (approximately 200 ft and 220 ft long) have a wall thickness of 2'-0" on one side and 2'-6" on the other side to allow for placement of embedded conduits. The interior below grade dimensions of these tunnels are approximately 7 ft high by 3 ft wide. DGFOT No. 1B, with a wall thickness of 2'-0" on both sides and shorter tunnel length for resisting torsion effects, is selected as the critical tunnel for the FEA.

The Safe Shutdown Earthquake (SSE) design forces (E') are conservatively determined using equivalent static seismic loads. The mass of the structure, equipment weights, and seismic live loads are excited in the X, Y, and Z directions using the enveloping maximum nodal accelerations in the X, Y, and Z directions from the soil-structure interaction (SSI) analysis. A comparison between the maximum accelerations from the SSI analysis and the design accelerations for the DGFOT shows the design accelerations envelope the SSI analysis accelerations. The comparison is provided in Table 03.08.04-30.3. The resulting element forces and moments due to X, Y, and Z excitations are combined using the SRSS method.

Figures 3H.7-5 through 3H.7-8 (see Enclosure 4) show a comparison of the SSI soil pressures, the SSSI soil pressures, the ASCE 4-98 soil pressures and the total enveloping soil pressure used in design on the walls of the DGFOT.

The codes and standards used for the design of the DGFOT are as outlined in Section 3H.7.4.1 provided in the COLA mark-ups in Enclosure 4 of this response. The loads and load combinations are as noted in Section 3H.7 provided in the COLA mark-ups in Enclosure 4 of this response.

Additionally, the axial strain on the DGFOT due to SSE wave propagation is determined based on the equations and commentary outlined in Section 3.5.2.1 of ASCE 4-98. The maximum curvature is computed based on Equation 3.5-3 in Section 3.5.2.1.3 of ASCE 4-98. The forces at bends due to SSE wave propagation are determined based on Section 3.5.2.2 of

ASCE 4-98. The forces at tunnel bends are included as additional loads in the SAP2000 models.

Multiple SAP2000 FEA models were created to represent different conditions and load combinations for the DGFOTs. Additional information on the SAP2000 models is documented in Section 3H.7.5.1 (see Enclosure 4). The following is a breakdown of the different FEA models:

1. Normal (Operating Condition, Heavy Load Condition, and Flood Load Condition):

   The purpose of these models is to consider the effects of operating load conditions (i.e. dead loads, minimum live loads, etc.), the heavy load condition (when heavy vehicles and cargo are moved across the top of the tunnel), and the flood load condition (the extreme flood loads due to a MCR breach).

2. SSE (SSE loads without SSE Wave Propagation):

   The purpose of these models is to consider the effects of SSE loads without the effects of the SSE wave propagation, which are considered in a separate model. The dead loads, live loads, soil loads, and accidental eccentricity loads are applied to the static (non-seismic) model. The SSE loads are combined using the SRSS method in the dynamic (seismic) model.

3. SSE (SSE loads with SSE Wave Propagation per ASCE 4-98):

   The purpose of these models is to consider the effects of SSE loads with the effects of the SSE wave propagation and additional forces and moments due to bends in the tunnel per ASCE 4-98. The dead loads, live loads, soil loads, accidental eccentricity loads, SSE wave propagation loads and additional forces and moments due to bends in the tunnel are applied to the static (non-seismic) model. The SSE loads are combined using the SRSS method in the dynamic (seismic) model.

4. Tornado Missile:

   The purpose of these models is to consider the effects of tornado missiles. The full tornado load combinations, outlined in Section 3H.7.4.3.4.2 (see Enclosure 4) are applied to the model considering a vertical tornado missile. The results of this SAP2000 model are combined with those from a manual calculation which considers the full tornado load combination and a horizontal tornado missile.

5. Effect of Uplift:

   The purpose of this model is to consider the effects of uplift on the basemat during a seismic event. All loads are simultaneously applied to a single static model.

The models described above are developed to determine the reinforcement required for their specific loading conditions. The results are post-processed as described in Section 3H.7.5.3.1 (see Enclosure 4).

The required reinforcement (longitudinal, in-plane shear and transverse) reported in Table 3H.7-1 (see Enclosure 4) is based on the envelop of the required reinforcement determined from all the SAP2000 FEA analyses and the required reinforcement determined via the manual calculation for the full tornado load combination.

The stability of the DGFOT is evaluated for the various load combinations listed in Section 3H.7.4.5 (see Enclosure 4). The DGFOT factors of safety against sliding, overturning and flotation are provided in Table 3H.7-2 (see Enclosure 4). These factors of safety meet the requirements of Standard Review Plan (SRP) 3.8.5. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for consideration of the X, Y, and Z seismic excitations.

More detailed and specific description of loads, load combinations and results of analysis and design of the DGFOT is provided in the COLA mark-up shown in Enclosure 4.

COLA will be revised as shown in Enclosures 1 through 4 as a result of this response.

		In-plane Shear Force (kip)	In-plane Moment (kip-ft)	Axial Force (kip)
Long Direction (Two Walls)	SAP2000 section cut seismic design forces:	6847	74581	
	Enveloped SSI peak section cut forces:	6451	53060	
	Ratio SAP2000/SSI section cut forces:	1.06	1.41	
Short Direction (Two Walls)	SAP2000 section cut seismic design forces:	5277	32140	
	Enveloped SSI peak section cut forces:	4162	25651	
	Ratio SAP2000/SSI section cut forces:	1.27	1.25	N/A
Total Building	SAP2000 section cut forces due to Z-direction seismic load:	N/A	N/A	2559
	Enveloped SSI peak section cut forces:	N/A	N/A	2488
	Ratio SAP2000/SSI section cut forces:	N/A	N/A	1.03

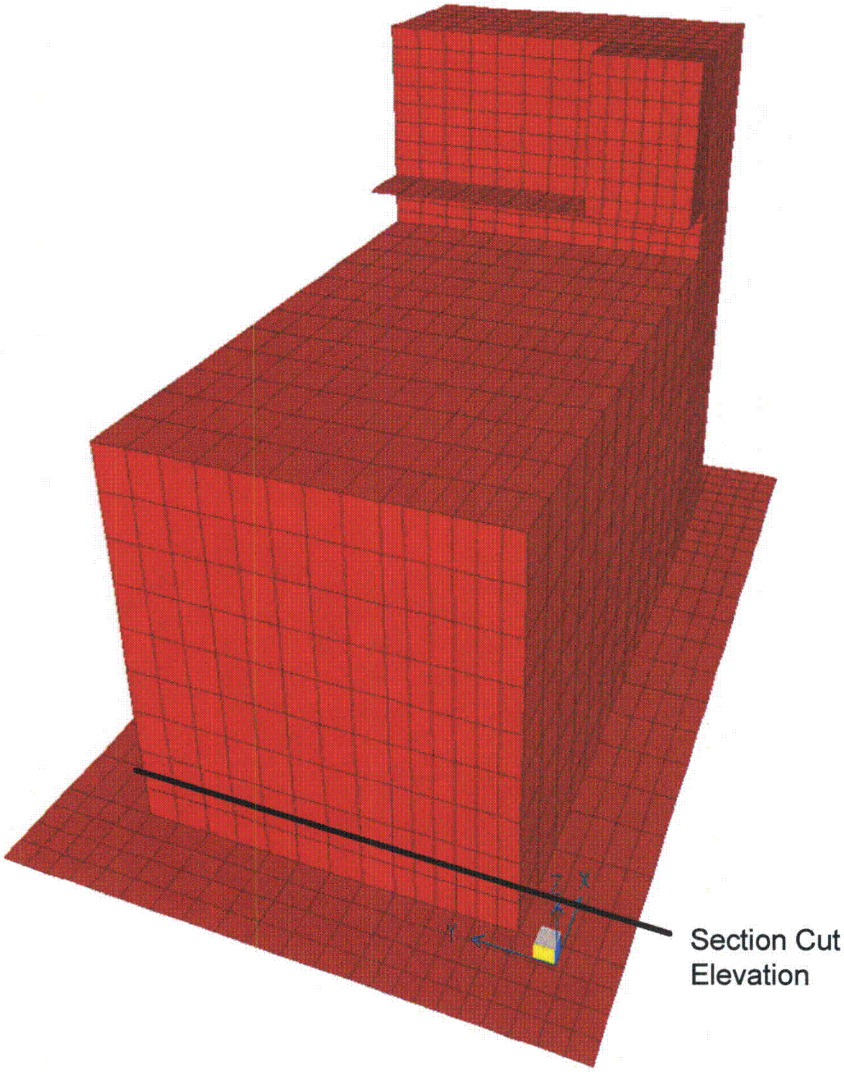
Table 03.08.04-30.1: SAP2000 (Uniform Springs Model) vs SSI Model Section Cut Seismic Force and Moment Comparison for DGFOSV

		In-plane Shear Force (kip)	In-plane Moment (kip-ft)	Axial Force (kip)
Long Direction (Two Walls)	SAP2000 section cut seismic design forces:	6808	71838	
	Enveloped SSI peak section cut forces:	6451	53060	
	Ratio SAP2000/SSI section cut forces:	1.06	1.35	
Short Direction (Two Walls)	SAP2000 section cut seismic design forces:	5239	33571	
	Enveloped SSI peak section cut forces:	4162	25651	
	Ratio SAP2000/SSI section cut forces:	1.26	1.31	N/A
Total Building	SAP2000 section cut forces due to Z-direction seismic load:	N/A	N/A	2559
	Enveloped SSI peak section cut forces:	N/A	N/A	2488
	Ratio SAP2000/SSI section cut forces:	N/A	N/A	1.03

Table 03.08.04-30.2: SAP2000 (Coupled Springs Model) vs SSI Model Section Cut Seismic Force and Moment Comparison for DGFOSV

	Horizontal (X & Y)		Vertical (Z)	
	From SSI Analysis (g)	Used in Design (g)	From SSI Analysis (g)	Used in Design (g)
Tunnel (0.0 <= Z <= 9.0)	0.3591	0.45	0.3078	0.37
Access Regions (9.0 < Z <= 23.17)	0.7324	0.85	0.3286	0.40

Table 03.08.04-30.3: Comparison of SSI Accelerations vs. Design Accelerations for DGFOT



Note:

The first row of elements at the bottom of the DGFOSV walls are 3' link elements that model the distance from the center of the 6' basemat to the bottom of the walls. Therefore, the section cuts are taken at the second row of elements from the bottom of the walls.

Figure 03.08.04-30.1: Location of section cut in SAP2000 design model

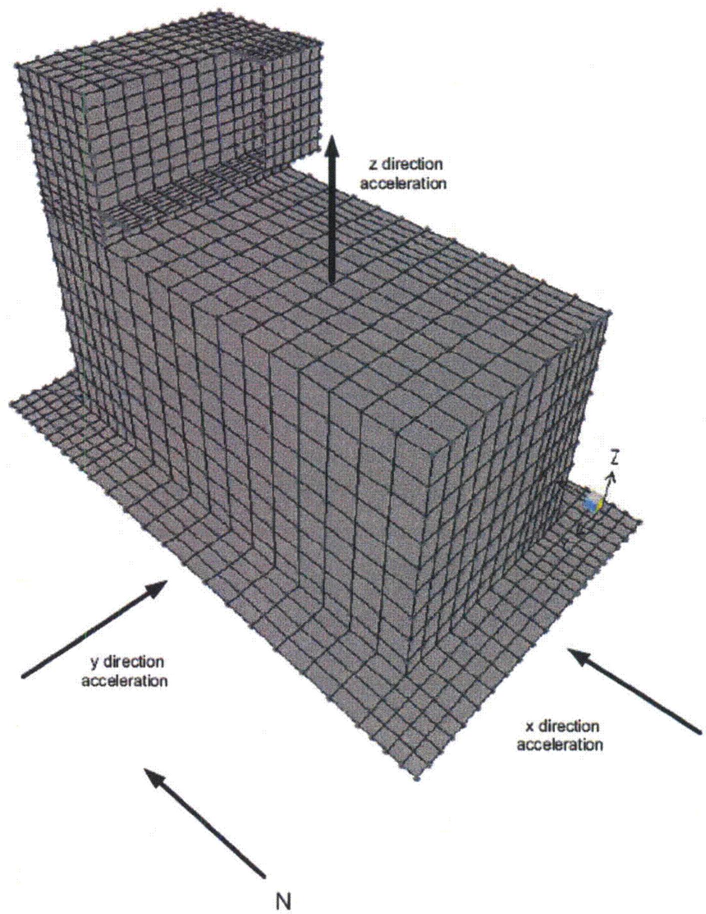


Figure 03.08.04-30.2: Schematic view of the Directional Seismic Load

Enclosure 1

Revision to COLA Section 3.7

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

The Category I structures and their physical proximity to nearby non-Category I structures are shown in Figure 3.7-40. None of the non-Category I structures proposed as part of STP Units 3 and 4 is intended to meet Criterion (2) of DCD Section 3.7.2.8. Rather, for each non-Category I structure, either: (1) it is determined that the collapse of the non-Category I structure will not cause the non-Category I structure to strike a Category I structure; or (2) the non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions in a manner such that the margin of safety of the structure is equivalent to that of Seismic Category I structures. Non-Category I structures that can interact with Seismic Category I structures include the Turbine Building (TB), Radwaste Building (RWB), Service Building (SB), Control Building Annex (CBA) and the stack on the Reactor Building roof. Table 3H.6-14 provides sliding and overturning factors of safety under site-specific SSE for TB, RWB, SB, and CBA.

The seismic input motions for the design of the five non-seismic category I structures noted above are described in the following:

- TB: 0.3g Regulatory Guide 1.60 spectra.
- RWB: as described in Section 3H.3.5.3 and shown in Figures 3.7-40 through 3.7-42.
- SB: 0.3g Regulatory Guide 1.60 spectra.
- CBA: as described in Section 3.7.3.16 and shown in Figures 3.7-38 and 3.7-39.

Stack on the Reactor Building roof: seismic loading at its location, resulting from the SSE analysis of the Reactor Building.

Enclosure 2
Revision to COLA Section 3.8

3.8.6.1 Foundation Waterproofing

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

3.8.6.4 Identification of Seismic Category I Structures

The following site-specific supplement addresses COL License Information Item 3.26.

A complete list of Seismic Category I Structures, Systems, and Components can be found in Table 3.2-1, which includes the following site-specific Seismic Category I Structures:

- Ultimate Heat Sink
- Rector Service Water Piping Tunnel
- Diesel Generator Fuel Oil Storage Vault

A description of these structures can be found in section 3H.6.

Enclosure 3

Revision to COLA Section 3H.6

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

~~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.~~

Stability evaluations were performed for sliding, overturning, and flotation. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for consideration of the X, Y, and Z seismic excitations. Since the orientation of the DGFOSVs in the horizontal plane can be along the East-West or North-South axes, the horizontal seismic values used in the stability calculation envelope the SSI accelerations in the X and Y directions. The calculated factors of safety against sliding, overturning, and flotation for the DGFOSV are included in Table 3H.6-12.

3H.6.7.1 Applicable Codes, Standards, Specifications and Load Combinations and Materials

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 structural materials used in the design of the DGFOSV are specified in Section 3H.6.4.4.

3H.6.7.2 Structural Design

The structural analysis and design of the Diesel Generator Fuel Oil Storage Vault (DGFOSV) was performed using a finite element analysis (FEA). The finite element model (FEM) for this FEA is Figure 3H.6-140. The analysis for the seismic loads was performed using equivalent static seismic loads. The maximum nodal accelerations from the SSI analysis in the X, Y, and Z direction for the subgrade and above grade roofs were averaged and used as the accelerations in the X, Y, and Z directions for the entire structure to obtain the equivalent static seismic loads. The induced forces due to the X, Y, and Z seismic excitations were combined using the square-root-sum-of-squares (SRSS) method.

Comparison of the seismic in-plane shear forces, axial forces and in-plane moments for the shear walls of this structure from the equivalent static method and those from the SSI analyses at a section cut just above the basemat shows that the forces and moments from the equivalent static method are in excess of those from the SSI analyses.

The strength design criteria of ACI 349, as supplemented by RG 1.142, were used for the design of the reinforced concrete elements of the DGFOSV. Concrete with minimum compressive strength of 4.0 ksi (27.6 MPa) and reinforcing steel with yield strength of 60 ksi (414 MPa) are considered in the design.

Due to difference in soil spring constants for seismic and non-seismic loads, the FEA analyses for the non-seismic loads and equivalent static seismic loads were run on different FEA models and the results from these models were combined and adjusted per Section 3H.6.7.3.1 outside the SAP2000 model to obtain the combined total design forces and moments for the seismic load combinations.

3H.6.7.2.1 Wall and Slab Design

The revised design forces and provided reinforcement for the DGFOSV walls and slabs are shown in Table 3H.6-11. Each face and each direction of each wall and slab has a corresponding longitudinal reinforcement zone figure. Each wall and slab also has a corresponding transverse shear reinforcement zone figure where transverse shear reinforcement is required. The reinforcement zone figures (Figure 3H.6-142 through 3H.6-208) show the various zones used to define the provided reinforcement based on the finite element analysis results. Actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The shell forces from every element for every load combination in the finite element analysis were evaluated to determine the provided reinforcement in each reinforcement zone. For each reinforcement zone, the following out-of-plane moment and axial force coupled with the corresponding load combination are reported in Table 3H.6-11:

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

For each reinforcement zone, the following in-plane and transverse shears with the corresponding load combination are reported in Table 3H.6-11:

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

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

The provided transverse shear reinforcing (as required) is determined based on the transverse shears and axial forces perpendicular to the shear plane occurring simultaneously for every load combination.

The DGFOSV below grade roof was designed with composite steel beams and concrete slabs for vertical loading. The composite beams span in the SAP2000 model Y-direction with the concrete slab designed as spanning one-way between the composite beams. The below grade roof slab acts as a diaphragm to transfer lateral loads. The provided reinforcing for the below grade roof slab is reported in Table 3H.6-11.

3H.6.7.3 Foundation

The foundation for the DGFOSV consists of a reinforced concrete mat and a lean concrete mud mat. The basemat deflections due to the flexibility of the basemat and supporting soil were accounted for through the use of foundation soil springs in the SAP2000 FEA models. Both the Winkler and the Pseudo-Coupled Methods were used to model the foundation soil springs, and the results of the two analyses were enveloped for design purposes.

Two different subgrade reactions (soil spring constants) are used, one for seismic loads and one for non-seismic loads. The following soil spring constants were used in the FEA models of the DGFOSVs:

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

Vertical springs (with seismic loads)...314 kips/ft/ft^2

North-south springs (with static and seismic loads)....................................229 kips/ft/ft^2

East-west springs (with static and seismic loads)......................................213 kips/ft/ft^2

3H.6.7.3.1 Uplift Analysis

The SAP2000 finite element models were checked for uplift effects by reviewing the joint reaction at the basemat. It was determined that under seismic loading the DGFOSV experiences uplift. Using the 100%, 40%, 40% rule for combination of three seismic excitations, non-linear analysis was run on each model with uniform Winkler soil springs and pseudo-coupled soil springs to determine an enveloping adjustment factor for forces and moments from the linear analysis for the foundation mat and the connecting walls. The non-linear analysis iterates multiple times removing soil springs that go into tension during each iteration until no soil springs are in tension. For the directional earthquake loading required for the nonlinear analysis, the DGFOSV critical loading, a safe shutdown earthquake (SSE) from the southwest in combination with static active and passive loads for SSE, is considered.

Comparing resultant foundation mat and wall reactions from the linear analysis with mat and wall reactions from the nonlinear analysis, there is a maximum reaction increase of approximately 67% for the foundation mat shear and axial forces, 17% increase for the foundation mat bending moments, and 6% increase for the connecting walls shear forces, axial forces, and bending moments (enveloping cases with Winkler and pseudo-coupled soil springs) in the nonlinear analysis. To account for this, the resulting forces and moments from the linear analyses were adjusted by applying an increase factor of 1.67 to all forces in the foundation mat, an increase factor of 1.17 to all moments in the foundation mat, and an increase factor 1.06 to all forces and moments in the connecting walls for the DGFOSV design.

Table 3H.6-11: Results of DGFOS Vault Concrete Design

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads – Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads – Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Slab 1	6	Near Side	Horizontal	3H.6-142	1-H-L	Max Tension w/ corresponding moment	372	D + F + L + H' +E'	47	-170	D + F + L + H' +E'	19	3.12	-	-	-	
						Max Compression w/ corresponding moment	139	D + F + L + H' +E'	-94	-131							
						Max Moment with axial tension	104	D + F + L + H' +E'	1	-297							
						Max Moment with axial compression	105	D + F + L + H' +E'	-13	-301							
					2-H-L	Max Tension w/ corresponding moment	361	D + F + L + H' +E'	54	-101	D + F + L + H' +E'	30	4.68	-	-	-	
						Max Compression w/ corresponding moment	377	D + F + L + H' +E'	-93	-173							
						Max Moment with axial tension	36	D + F + L + H' +E'	11	-706							
						Max Moment with axial compression	36	D + F + L + H' +E'	-8	-706							
					3-H-L	Max Tension w/ corresponding moment	344	D + F + L + H' +E'	56	-140	D + F + L + H' +E'	33	4.68	-	-	-	
						Max Compression w/ corresponding moment	365	D + F + L + H' +E'	-96	-21							
						Max Moment with axial tension	363	D + F + L + H' +E'	7	-703							
						Max Moment with axial compression	363	D + F + L + H' +E'	-12	-703							
					4-H-L	Max Tension w/ corresponding moment	2182	D + F + L + H' +E'	101	-96	D + F + L + H' +E'	10	3.12	-	-	-	
						Max Compression w/ corresponding moment	2221	D + F + L + H' +E'	-113	-64							
						Max Moment with axial tension	2183	D + F + L + H' +E'	13	-226							
						Max Moment with axial compression	2183	D + F + L + H' +E'	-17	-178							
					5-H-L	Max Tension w/ corresponding moment	2263	D + F + L + H' +E'	103	-88	D + F + L + H' +E'	10	3.12	-	-	-	
						Max Compression w/ corresponding moment	2278	D + F + L + H' +E'	-113	-69							
						Max Moment with axial tension	2249	D + F + L + H' +E'	23	-213							
						Max Moment with axial compression	2249	D + F + L + H' +E'	-6	-163							
			Vertical	3H.6-143	1-V-L	Max Tension w/ corresponding moment	180	D + F + L + H' +E'	38	-286	D + F + L + H' +E'	27	3.12	-	-	-	
						Max Compression w/ corresponding moment	174	D + F + L + H' +E'	-262	-29							
						Max Moment with corresponding axial tension	327	D + F + L + H' +E'	10	-422							
						Max Moment with corresponding axial compression	105	D + F + L + H' +E'	-47	-440							
					2-V-L	Max Tension w/ corresponding moment	2432	D + F + L + H' +E'	63	-78	D + F + L + H' +E'	23	3.12	-	-	-	
						Max Compression w/ corresponding moment	19	D + F + L + H' +E'	-18	-26							
						Max Moment with axial tension	18	D + F + L + H' +E'	27	-398							
						Max Moment with axial compression	18	D + F + L + H +Wt	0	-111							
					3-V-L	Max Tension w/ corresponding moment	396	D + F + L + H' +E'	62	-35	D + F + L + H' +E'	26	3.12	-	-	-	
						Max Compression w/ corresponding moment	397	D + F + L + H' +E'	-21	-11							
						Max Moment with axial tension	381	D + F + L + H' +E'	33	-363							
						Max Moment with axial compression	381	D + F + L + H +Wt	0	-63							
					4-V-L	Max Tension w/ corresponding moment	22	D + F + L + H' +E'	49	-21	D + F + L + H' +E'	34	6.24	-	-	-	
						Max Compression w/ corresponding moment	39	D + F + L + H' +E'	-124	-65							
						Max Moment with axial tension	36	D + F + L + H' +E'	34	-721							
						Max Moment with axial compression	36	D + F + L + H' +E'	-29	-555							
					5-V-L	Max Tension w/ corresponding moment	363	D + F + L + H' +E'	57	-194	D + F + L + H' +E'	37	6.24	-	-	-	
						Max Compression w/ corresponding moment	345	D + F + L + H' +E'	-137	-67							
						Max Moment with axial tension	363	D + F + L + H' +E'	42	-727							
						Max Moment with axial compression	363	D + F + L + H' +E'	-21	-574							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in²/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
Slab 1	6	Far side	Horizontal	3H.6-144	1-H-L	Max Tension w/ corresponding moment	2299	D + F + L + H' +E'	120	209	D + F + L + H' +E'	33	3.12	-	-	-	
						Max Compression w/ corresponding moment	354	D + F + L + H' +E'	-125	333							
						Max Moment with axial tension	99	D + F + L + H' +E'	7	563							
						Max Moment with axial compression	99	D + F + L + H' +E'	-1	563							
			Vertical	3H.6-145	1-V-L	Max Tension w/ corresponding moment	71	D + F + L + H' +E'	59	544	D + F + L + H' +E'	27	3.12	-	-	-	
						Max Compression w/ corresponding moment	231	D + F + L + H' +E'	-456	540							
						Max Moment with corresponding axial tension	58	D + F + L + H' +E'	25	701							
						Max Moment with corresponding axial compression	184	D + F + L + H' +E'	-385	1091							
					2-V-L	Max Tension w/ corresponding moment	2467	D + F + L + H' +E'	116	208	D + F + L + H' +E'	23	3.12	-	-	-	
						Max Compression w/ corresponding moment	19	D + F + L + H' +E'	-18	96							
						Max Moment with axial tension	17	D + F + L + H' +E'	79	377							
						Max Moment with axial compression	3	D + F + L + H' +E'	-1	292							
					3-V-L	Max Tension w/ corresponding moment	2521	D + F + L + H' +E'	118	235	D + F + L + H' +E'	15	3.12	-	-	-	
						Max Compression w/ corresponding moment	2512	D + F + L + H' +E'	-12	25							
						Max Moment with axial tension	2525	D + F + L + H' +E'	97	297							
						Max Moment with axial compression	2483	D + F + L + H' +E'	0	168							
					4-V-L	Max Tension w/ corresponding moment	40	D + F + L + H' +E'	147	631	D + F + L + H' +E'	34	6.24	-	-	-	
						Max Compression w/ corresponding moment	39	D + F + L + H' +E'	-124	210							
						Max Moment with axial tension	40	D + F + L + H' +E'	147	631							
						Max Moment with axial compression	21	D + F + L + H' +E'	-62	519							
					5-V-L	Max Tension w/ corresponding moment	346	D + F + L + H' +E'	187	585	D + F + L + H' +E'	37	6.24	-	-	-	
						Max Compression w/ corresponding moment	345	D + F + L + H' +E'	-137	304							
						Max Moment with axial tension	310	D + F + L + H' +E'	43	716							
						Max Moment with axial compression	378	D + F + L + H' +E'	-66	584							
Roof 2	2	Near Side	Horizontal	3H.6-147	1-H-L	Max Tension w/ corresponding moment	553	D + F + L + H' +E'	61	-25	D + F + L + H' +E'	40	3.12	-	-	-	(9)
						Max Compression w/ corresponding moment	553	D + F + L + H' +E'	-126	-29							
						Max Moment with axial tension	553	D + F + L + H' +E'	21	-52							
						Max Moment with axial compression	539	D + F + L + H' +E'	-92	-68							
			Vertical	3H.6-148	1-V-L	Max Tension w/ corresponding moment	399	D + F + L + H' +E'	34	-53	D + F + L + H' +E'	60	3.12	-	-	-	
						Max Compression w/ corresponding moment	554	D + F + L + H' +E'	-136	-120							
						Max Moment with corresponding axial tension	399	D + F + L + H' +E'	32	-58							
						Max Moment with corresponding axial compression	540	D + F + L + H' +E'	-105	-134							
					2-V-L	Max Tension w/ corresponding moment	566	D + F + L + H' +E'	-	-	D + F + L + H' +E'	22	6.24	-	-	-	(10)
						Max Compression w/ corresponding moment	566	D + F + L + H' +E'	-140	-151							
						Max Moment with corresponding axial tension	566	D + F + L + H' +E'	-	-							
						Max Moment with corresponding axial compression	566	D + F + L + H' +E'	-102	-210							
					3-V-L	Max Tension w/ corresponding moment	553	D + F + L + H' +E'	-	-	D + F + L + H' +E'	22	6.24	-	-	-	(10)
						Max Compression w/ corresponding moment	553	D + F + L + H' +E'	-142	-154							
						Max Moment with corresponding axial tension	539	D + F + L + H' +E'	-	-							
						Max Moment with corresponding axial compression	553	D + F + L + H' +E'	-103	-216							
		Far side	Horizontal	3H.6-149	1-H-L	Max Tension w/ corresponding moment	399	D + F + L + H' +E'	18	6	D + F + L + H' +E'	40	3.12	-	-	-	
						Max Compression w/ corresponding moment	553	D + F + L + H' +E'	-108	56							
						Max Moment with axial tension	556	D + F + L + H' +E'	0	65							
						Max Moment with axial compression	565	D + F + L + H' +E'	-21	78							
			Vertical	3H.6-150	1-V-L	Max Tension w/ corresponding moment	554	D + F + L + H' +E'	17	15	D + F + L + H' +E'	60	3.12	-	-	-	
						Max Compression w/ corresponding moment	565	D + F + L + H' +E'	-114	10							
						Max Moment with corresponding axial tension	566	D + F + L + H' +E'	14	24							
						Max Moment with corresponding axial compression	566	D + F + L + H' +E'	-35	24							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Slab 3	2	Near Side	Horizontal	3H.6-151	1-H-L	Max Tension w/ corresponding moment	650	D + F + L + H' +E'	29	-10	D + F + L + H +Wt	24	1.56	-	-	-	
						Max Compression w/ corresponding moment	638	D + F + L + H +Wt	-58	-21							
						Max Moment with axial tension	643	D + F + L + H +Wt	2	-38							
						Max Moment with axial compression	638	D + F + L + H +Wt	-54	-57							
			Vertical	3H.6-152	1-V-L	Max Tension w/ corresponding moment	574	D + F + L + H +Wt	34	-10	D + F + L + H +Wt	16	1.56	-	-	-	
						Max Compression w/ corresponding moment	574	D + F + L + H' +E'	-81	-10							
						Max Moment with corresponding axial tension	574	D + F + L + H' +E'	31	-35							
						Max Moment with corresponding axial compression	574	D + F + L + H' +E'	0	-35							
		Far side	Horizontal	3H.6-153	1-H-L	Max Tension w/ corresponding moment	638	D + F + L + H +Wt	30	5	D + F + L + H +Wt	24	1.56	-	-	-	
						Max Compression w/ corresponding moment	651	D + F + L + H' +E'	-44	6							
						Max Moment with axial tension	643	D + F + L + H' +E'	3	26							
						Max Moment with axial compression	573	D + F + L + H +Wt	-6	35							
			Vertical	3H.6-154	1-V-L	Max Tension w/ corresponding moment	574	D + F + L + H +Wt	34	7	D + F + L + H +Wt	16	1.56	-	-	-	
						Max Compression w/ corresponding moment	574	D + F + L + H +Wt	-114	41							
						Max Moment with corresponding axial tension	574	D + F + L + H +Wt	1	26							
						Max Moment with corresponding axial compression	574	D + F + L + H +Wt	-114	41							
Roof 5	2	Near Side	Horizontal	3H.6-155	1-H-L	Max Tension w/ corresponding moment	690	D + F + L + H +Wt	44	-12	D + F + L + H +Wt	37	1.56	-	-	-	
						Max Compression w/ corresponding moment	695	D + F + L + H +Wt	-47	-8							
						Max Moment with axial tension	771	D + F + L + H' +E'	0	-24							
						Max Moment with axial compression	768	D + F + L + H' +E'	-8	-39							
			Vertical	3H.6-156	1-V-L	Max Tension w/ corresponding moment	769	D + F + L + H +Wt	63	-5	D + F + L + H' +E'	18	1.56	-	-	-	
						Max Compression w/ corresponding moment	693	D + F + L + H +Wt	-53	-2							
						Max Moment with corresponding axial tension	766	D + F + L + H +Wt	2	-17							
						Max Moment with corresponding axial compression	768	D + F + L + H +Wt	-31	-19							
		Far side	Horizontal	3H.6-157	1-H-L	Max Tension w/ corresponding moment	704	D + F + L + H +Wt	32	5	D + F + L + H +Wt	37	1.56	-	-	-	
						Max Compression w/ corresponding moment	767	D + F + L + H +Wt	-145	16							
						Max Moment with axial tension	698	D + F + L + H +Wt	1	19							
						Max Moment with axial compression	732	D + F + L + H +Wt	-22	49							
			Vertical	3H.6-158	1-V-L	Max Tension w/ corresponding moment	711	D + F + L + H +Wt	27	0	D + F + L + H' +E'	18	1.56	-	-	-	
						Max Compression w/ corresponding moment	732	D + F + L + H +Wt	-170	15							
						Max Moment with corresponding axial tension	732	D + F + L + H' +E'	4	14							
						Max Moment with corresponding axial compression	697	D + F + L + H +Wt	-43	43							
Roof 6	2	Near Side	Horizontal	3H.6-159	1-H-L	Max Tension w/ corresponding moment	684	D + F + L + H +Wt	43	-7	D + F + L + H +Wt	52	1.56	-	-	-	
						Max Compression w/ corresponding moment	689	D + F + L + H +Wt	-107	-29							
						Max Moment with axial tension	687	D + F + L + H +Wt	2	-48							
						Max Moment with axial compression	689	D + F + L + H +Wt	-30	-74							
			Vertical	3H.6-160	1-V-L	Max Tension w/ corresponding moment	689	D + F + L + H +Wt	29	-5	D + F + L + H +Wt	67	1.56	-	-	-	
						Max Compression w/ corresponding moment	689	D + F + L + H +Wt	-86	-2							
						Max Moment with corresponding axial tension	666	D + F + L + H +Wt	5	-24							
						Max Moment with corresponding axial compression	656	D + F + L + H +Wt	-38	-25							
		Far side	Horizontal	3H.6-161	1-H-L	Max Tension w/ corresponding moment	673	D + F + L + H +Wt	45	9	D + F + L + H +Wt	52	1.56	-	-	-	
						Max Compression w/ corresponding moment	657	D + F + L + H +Wt	-230	25							
						Max Moment with axial tension	657	D + F + L + H +Wt	2	53							
						Max Moment with axial compression	666	D + F + L + H +Wt	-21	62							
			Vertical	3H.6-162	1-V-L	Max Tension w/ corresponding moment	663	D + F + L + H +Wt	15	6	D + F + L + H +Wt	67	1.56	-	-	-	
						Max Compression w/ corresponding moment	666	D + F + L + H +Wt	-267	30							
						Max Moment with corresponding axial tension	660	D + F + L + H +Wt	3	17							
						Max Moment with corresponding axial compression	656	D + F + L + H +Wt	-37	75							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number(2)	Maximum Forces(3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial(4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Wall 7	4	Near Side	Horizontal	3H.6-163	1-H-L	Max Tension w/ corresponding moment	843	D + F + L + H' +E'	98	-44	D + F + L + H' +E'	60	3.12	-	-	-	
						Max Compression w/ corresponding moment	1051	D + F + L + H' +E'	-172	-310							
						Max Moment with axial tension	1014	D + F + L + H' +E'	0	-107							
						Max Moment with axial compression	1069	D + F + L + H' +E'	-162	-352							
					2-H-L	Max Tension w/ corresponding moment	811	D + F + L + H' +E'	49	-80	D + F + L + H' +E'	60	7.8	-	-	-	
						Max Compression w/ corresponding moment	799	D + F + L + H' +E'	-190	-815							
						Max Moment with axial tension	803	D + F + L + H' +E'	8	-239							
						Max Moment with axial compression	799	D + F + L + H' +E'	-190	-815							
					3-H-L	Max Tension w/ corresponding moment	891	D + F + L + H' +E'	147	-231	D + F + L + H' +E'	31	6.24	-	-	-	
						Max Compression w/ corresponding moment	1042	D + F + L + H' +E'	-218	-202							
						Max Moment with axial tension	1042	D + F + L + H' +E'	91	-291							
						Max Moment with axial compression	1057	D + F + L + H' +E'	-145	-344							
					4-H-L	Max Tension w/ corresponding moment	1046	D + F + L + H' +E'	20	-77	D + F + L + H' +E'	60	7.8	-	-	-	
						Max Compression w/ corresponding moment	1053	D + F + L + H' +E'	-191	-856							
						Max Moment with axial tension	1017	D + F + L + H' +E'	3	-114							
						Max Moment with axial compression	1065	D + F + L + H' +E'	-183	-897							
			Vertical	3H.6-164	1-V-L	Max Tension w/ corresponding moment	797	D + F + L + H' +E'	106	-127	D + F + L + H' +E'	90	4.68	-	-	-	
						Max Compression w/ corresponding moment	1029	D + F + L + H' +E'	-200	-59							
						Max Moment with corresponding axial tension	837	D + F + L + H' +E'	4	-345							
						Max Moment with corresponding axial compression	891	D + F + L + H' +E'	-118	-407							
					2-V-L	Max Tension w/ corresponding moment	796	D + F + L + H' +E'	151	-170	D + F + L + H' +E'	90	12.48	-	-	-	
						Max Compression w/ corresponding moment	796	D + F + L + H' +E'	-166	-79							
						Max Moment with corresponding axial tension	836	D + F + L + H' +E'	2	-1165							
						Max Moment with corresponding axial compression	852	D + F + L + H' +E'	-53	-1235							
		Far side	Horizontal	3H.6-165	1-H-L	Max Tension w/ corresponding moment	851	D + F + L + H' +E'	100	27	D + F + L + H' +E'	60	3.12	-	-	-	
						Max Compression w/ corresponding moment	891	D + F + L + H' +E'	-298	259							
						Max Moment with axial tension	1047	D + F + L + H' +E'	9	189							
						Max Moment with axial compression	814	D + F + L + H' +E'	-109	403							
			Vertical	3H.6-166	1-V-L	Max Tension w/ corresponding moment	796	D + F + L + H' +E'	130	62	D + F + L + H' +E'	90	6.24	-	-	-	
						Max Compression w/ corresponding moment	1017	D + F + L + H' +E'	-237	183							
						Max Moment with corresponding axial tension	848	D + F + L + H' +E'	0	717							
						Max Moment with corresponding axial compression	856	D + F + L + H' +E'	-19	724							
			Horizontal Plane	3H.6-167	1-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	95	0.31 (#5 @12)	
				3H.6-167	2-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	155	0.62 (#5 @6)	
				3H.6-167	3-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	60	0.31 (#5 @12)	
			Vertical Plane	3H.6-167	1-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	103	0.31 (#5 @12)	
				3H.6-167	2-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	102	0.31 (#5 @12)	
				3H.6-167	3-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	128	0.62 (#5 @6)	

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in²/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
Wall 8	4	Near Side	Horizontal	3H.6-168	1-H-L	Max Tension w/ corresponding moment	1156	D + F + L + H' +E'	97	-35	D + F + L + H' +E'	59	3.12	-	-	-	
						Max Compression w/ corresponding moment	1307	D + F + L + H' +E'	-171	-278							
						Max Moment with axial tension	1188	D + F + L + H' +E'	5	-193							
						Max Moment with axial compression	1183	D + F + L + H' +E'	-157	-351							
					2-H-L	Max Tension w/ corresponding moment	1276	D + F + L + H' +E'	20	-111	D + F + L + H' +E'	59	7.8	-	-	-	
						Max Compression w/ corresponding moment	1305	D + F + L + H' +E'	-190	-870							
						Max Moment with axial tension	1288	D + F + L + H' +E'	3	-119							
						Max Moment with axial compression	1311	D + F + L + H' +E'	-183	-913							
					3-H-L	Max Tension w/ corresponding moment	1108	D + F + L + H' +E'	142	-261	D + F + L + H' +E'	34	6.24	-	-	-	
						Max Compression w/ corresponding moment	1280	D + F + L + H' +E'	-212	-230							
						Max Moment with axial tension	1280	D + F + L + H' +E'	76	-324							
						Max Moment with axial compression	1301	D + F + L + H' +E'	-161	-385							
					4-H-L	Max Tension w/ corresponding moment	1189	D + F + L + H' +E'	27	-92	D + F + L + H' +E'	59	7.8	-	-	-	
						Max Compression w/ corresponding moment	1192	D + F + L + H' +E'	-190	-823							
						Max Moment with axial tension	1196	D + F + L + H' +E'	7	-222							
						Max Moment with axial compression	1192	D + F + L + H' +E'	-190	-823							
			Vertical	3H.6-169	1-V-L	Max Tension w/ corresponding moment	1190	D + F + L + H' +E'	109	-133	D + F + L + H' +E'	92	4.68	-	-	-	
						Max Compression w/ corresponding moment	1281	D + F + L + H' +E'	-198	-42							
						Max Moment with corresponding axial tension	1108	D + F + L + H' +E'	0	-382							
						Max Moment with corresponding axial compression	1108	D + F + L + H' +E'	-105	-429							
					2-V-L	Max Tension w/ corresponding moment	1189	D + F + L + H' +E'	152	-174	D + F + L + H' +E'	92	12.48	-	-	-	
						Max Compression w/ corresponding moment	1189	D + F + L + H' +E'	-164	-88							
						Max Moment with corresponding axial tension	1149	D + F + L + H' +E'	1	-1170							
						Max Moment with corresponding axial compression	1133	D + F + L + H' +E'	-54	-1237							
		Far side	Horizontal	3H.6-170	1-H-L	Max Tension w/ corresponding moment	1148	D + F + L + H' +E'	99	20	D + F + L + H' +E'	59	4.68	-	-	-	
						Max Compression w/ corresponding moment	1108	D + F + L + H' +E'	-286	275							
						Max Moment with axial tension	1275	D + F + L + H' +E'	9	220							
						Max Moment with axial compression	1175	D + F + L + H' +E'	-108	413							
			Vertical	3H.6-171	1-V-L	Max Tension w/ corresponding moment	1189	D + F + L + H' +E'	131	65	D + F + L + H' +E'	92	6.24	-	-	-	
						Max Compression w/ corresponding moment	1269	D + F + L + H' +E'	-233	173							
						Max Moment with corresponding axial tension	1145	D + F + L + H' +E'	2	721							
						Max Moment with corresponding axial compression	1145	D + F + L + H' +E'	-18	723							
			Horizontal Plane	3H.6-172	1-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	96	0.31 (#5 @12)	
				3H.6-172	2-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	155	0.62 (#5 @6)	
				3H.6-172	3-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	60	0.31 (#5 @12)	
			Vertical Plane	3H.6-172	1-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	127	0.62 (#5 @6)	
				3H.6-172	2-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	101	0.31 (#5 @12)	
				3H.6-172	3-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	102	0.31 (#5 @12)	

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads – Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads – Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Wall 9	2	Near Side	Horizontal	3H.6-173	1-H-L	Max Tension w/ corresponding moment	1031	D + F + L + H +Wt	53	-8	D + F + L + H +Wt	58	3.12	-	-	-	
						Max Compression w/ corresponding moment	987	D + F + L + H +Wt	-98	-4							
						Max Moment with axial tension	1018	D + F + L + H' +E'	16	-79							
						Max Moment with axial compression	1035	D + F + L + H' +E'	-36	-101							
					2-H-L	Max Tension w/ corresponding moment	1030	D + F + L + H +Wt	83	-18	D + F + L + H +Wt	58	4.68	-	-	-	
						Max Compression w/ corresponding moment	1030	D + F + L + H' +E'	-160	-8							
						Max Moment with axial tension	1030	D + F + L + H' +E'	53	-94							
						Max Moment with axial compression	1030	D + F + L + H' +E'	-11	-94							
			Vertical	3H.6-174	1-V-L	Max Tension w/ corresponding moment	1006	D + F + L + H' +E'	84	-61	D + F + L + H +Wt	47	3.12	-	-	-	
						Max Compression w/ corresponding moment	1006	D + F + L + H +Wt	-162	-2							
						Max Moment with corresponding axial tension	1031	D + F + L + H' +E'	8	-97							
						Max Moment with corresponding axial compression	1031	D + F + L + H' +E'	-46	-97							
					2-V-L	Max Tension w/ corresponding moment	1030	D + F + L + H' +E'	130	-103	D + F + L + H +Wt	45	6.24	-	-	-	
						Max Compression w/ corresponding moment	1030	D + F + L + H' +E'	-231	-50							
						Max Moment with corresponding axial tension	1030	D + F + L + H' +E'	36	-180							
						Max Moment with corresponding axial compression	1030	D + F + L + H' +E'	-77	-180							
		Far side	Horizontal	3H.6-175	1-H-L	Max Tension w/ corresponding moment	1030	D + F + L + H +Wt	56	9	D + F + L + H +Wt	58	3.12	-	-	-	
						Max Compression w/ corresponding moment	995	D + F + L + H +Wt	-181	16							
						Max Moment with axial tension	955	D + F + L + H +Wt	8	49							
						Max Moment with axial compression	983	D + F + L + H +Wt	-41	68							
			Vertical	3H.6-176	1-V-L	Max Tension w/ corresponding moment	1035	D + F + L + H' +E'	64	4	D + F + L + H +Wt	47	3.12	-	-	-	
						Max Compression w/ corresponding moment	1030	D + F + L + H' +E'	-205	8							
						Max Moment with corresponding axial tension	1003	D + F + L + H' +E'	6	15							
						Max Moment with corresponding axial compression	995	D + F + L + H +Wt	-39	69							
Wall 10	2	Near Side	Horizontal	3H.6-177	1-H-L	Max Tension w/ corresponding moment	1257	D + F + L + H +Wt	71	-16	D + F + L + H +Wt	59	3.12	-	-	-	
						Max Compression w/ corresponding moment	1257	D + F + L + H' +E'	-150	-10							
						Max Moment with axial tension	1257	D + F + L + H' +E'	50	-95							
						Max Moment with axial compression	1197	D + F + L + H' +E'	-36	-98							
			Vertical	3H.6-178	1-V-L	Max Tension w/ corresponding moment	1259	D + F + L + H' +E'	90	-56	D + F + L + H +Wt	38	3.12	-	-	-	
						Max Compression w/ corresponding moment	1259	D + F + L + H' +E'	-132	-6							
						Max Moment with corresponding axial tension	1245	D + F + L + H' +E'	1	-103							
						Max Moment with corresponding axial compression	1245	D + F + L + H' +E'	-34	-103							
					2-V-L	Max Tension w/ corresponding moment	1257	D + F + L + H' +E'	126	-110	D + F + L + H +Wt	35	6.24	-	-	-	
						Max Compression w/ corresponding moment	1257	D + F + L + H' +E'	-199	-57							
						Max Moment with corresponding axial tension	1257	D + F + L + H' +E'	34	-187							
						Max Moment with corresponding axial compression	1257	D + F + L + H' +E'	-60	-187							
		Far side	Horizontal	3H.6-179	1-H-L	Max Tension w/ corresponding moment	1257	D + F + L + H +Wt	51	7	D + F + L + H +Wt	59	3.12	-	-	-	
						Max Compression w/ corresponding moment	1265	D + F + L + H +Wt	-179	19							
						Max Moment with axial tension	1264	D + F + L + H +Wt	0	49							
						Max Moment with axial compression	1232	D + F + L + H +Wt	-41	66							
			Vertical	3H.6-180	1-V-L	Max Tension w/ corresponding moment	1198	D + F + L + H' +E'	60	3	D + F + L + H +Wt	38	3.12	-	-	-	
						Max Compression w/ corresponding moment	1257	D + F + L + H' +E'	-173	4							
						Max Moment with corresponding axial tension	1224	D + F + L + H' +E'	6	17							
						Max Moment with corresponding axial compression	1265	D + F + L + H +Wt	-47	69							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in²/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
Wall 11	2	Near Side	Horizontal	3H.6-181	1-H-L	Max Tension w/ corresponding moment	951	D + F + L + H +Wt	43	-7	D + F + L + H +Wt	55	1.56	-	-	-	
						Max Compression w/ corresponding moment	939	D + F + L + H +Wt	-85	-1							
						Max Moment with axial tension	951	D + F + L + H +Wt	34	-44							
						Max Moment with axial compression	947	D + F + L + H +Wt	-2	-38							
			Vertical	3H.6-182	1-V-L	Max Tension w/ corresponding moment	944	D + F + L + H +Wt	37	-4	D + F + L + H +Wt	43	1.56	-	-	-	
						Max Compression w/ corresponding moment	908	D + F + L + H +Wt	-84	-25							
						Max Moment with corresponding axial tension	935	D + F + L + H +Wt	9	-38							
						Max Moment with corresponding axial compression	907	D + F + L + H +Wt	-80	-33							
		Far side	Horizontal	3H.6-183	1-H-L	Max Tension w/ corresponding moment	934	D + F + L + H +Wt	31	5	D + F + L + H +Wt	55	1.56	-	-	-	
						Max Compression w/ corresponding moment	907	D + F + L + H +Wt	-210	25							
						Max Moment with axial tension	947	D + F + L + H +Wt	5	45							
						Max Moment with axial compression	935	D + F + L + H +Wt	-20	89							
			Vertical	3H.6-184	1-V-L	Max Tension w/ corresponding moment	944	D + F + L + H +Wt	34	4	D + F + L + H +Wt	43	1.56	-	-	-	
						Max Compression w/ corresponding moment	927	D + F + L + H +Wt	-184	23							
						Max Moment with corresponding axial tension	935	D + F + L + H +Wt	0	69							
						Max Moment with corresponding axial compression	907	D + F + L + H +Wt	-79	99							
Wall 12	4	Near Side	Horizontal	3H.6-185	1-H-L	Max Tension w/ corresponding moment	1349	D + F + L + H' +E'	20	-12	D + F + L + H' +E'	107	3.12	-	-	-	
						Max Compression w/ corresponding moment	1345	D + F + L + H' +E'	-197	-365							
						Max Moment with axial tension	1349	D + F + L + H' +E'	14	-207							
						Max Moment with axial compression	1346	D + F + L + H' +E'	-185	-396							
					2-H-L	Max Tension w/ corresponding moment	1341	D + F + L + H' +E'	24	-166	D + F + L + H' +E'	107	6.24	-	-	-	
						Max Compression w/ corresponding moment	1337	D + F + L + H' +E'	-199	-800							
						Max Moment with axial tension	1341	D + F + L + H' +E'	16	-212							
						Max Moment with axial compression	1337	D + F + L + H' +E'	-199	-800							
					3-H-L	Max Tension w/ corresponding moment	1437	D + F + L + H' +E'	23	-163	D + F + L + H' +E'	107	6.24	-	-	-	
						Max Compression w/ corresponding moment	1433	D + F + L + H' +E'	-197	-513							
						Max Moment with axial tension	1445	D + F + L + H' +E'	15	-216							
						Max Moment with axial compression	1441	D + F + L + H' +E'	-197	-794							
			Vertical	3H.6-186	1-V-L	Max Tension w/ corresponding moment	1432	D + F + L + H' +E'	78	-40	D + F + L + H' +E'	99	3.12	-	-	-	
						Max Compression w/ corresponding moment	1440	D + F + L + H' +E'	-174	-73							
						Max Moment with corresponding axial tension	1373	D + F + L + H' +E'	3	-186							
						Max Moment with corresponding axial compression	1373	D + F + L + H' +E'	-18	-207							
					2-V-L	Max Tension w/ corresponding moment	1438	D + F + L + H' +E'	188	-113	D + F + L + H' +E'	99	6.24	-	-	-	
						Max Compression w/ corresponding moment	1438	D + F + L + H' +E'	-258	-16							
						Max Moment with corresponding axial tension	1350	D + F + L + H' +E'	30	-447							
						Max Moment with corresponding axial compression	1350	D + F + L + H' +E'	-15	-447							
					3-V-L	Max Tension w/ corresponding moment	1382	D + F + L + H' +E'	85	-666	D + F + L + H' +E'	89	7.8	-	-	-	
						Max Compression w/ corresponding moment	1406	D + F + L + H' +E'	-93	-43							
						Max Moment with corresponding axial tension	1374	D + F + L + H' +E'	78	-689							
						Max Moment with corresponding axial compression	1406	D + F + L + H' +E'	-7	-483							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Load Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Wall 12	4	Far side	Horizontal	3H.6-187	1-H-L	Max Tension w/ corresponding moment	1349	D + F + L + H' +E'	20	5	D + F + L + H' +E'	107	3.12	-	-	-	
						Max Compression w/ corresponding moment	1409	D + F + L + H' +E'	-192	52							
						Max Moment with axial tension	1349	D + F + L + H' +E'	1	75							
						Max Moment with axial compression	1393	D + F + L + H' +E'	-167	329							
			Vertical	3H.6-188	1-V-L	Max Tension w/ corresponding moment	1430	D + F + L + H' +E'	126	40	D + F + L + H' +E'	99	4.68	-	-	-	
						Max Compression w/ corresponding moment	1438	D + F + L + H' +E'	-258	41							
						Max Moment with corresponding axial tension	1384	D + F + L + H' +E'	51	343							
						Max Moment with corresponding axial compression	1400	D + F + L + H' +E'	-4	312							
		-	Horizontal Plane	3H.6-189	1-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	99	0.31 (#5 @12)	
			Vertical Plane	3H.6-189	1-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	107	0.31 (#5 @12)	
				3H.6-189	2-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	115	0.31 (#5 @12)	
Wall 13	4	Near Side	Horizontal	3H.6-190	1-H-L	Max Tension w/ corresponding moment	1883	D + F + L + H +Wt	2	-42	D + F + L + H' +E'	104	3.12	-	-	-	
						Max Compression w/ corresponding moment	1953	D + F + L + H' +E'	-198	-446							
						Max Moment with axial tension	1883	D + F + L + H' +E'	0	-107							
						Max Moment with axial compression	1953	D + F + L + H' +E'	-198	-446							
					2-H-L	Max Tension w/ corresponding moment	1871	D + F + L + H' +E'	33	-48	D + F + L + H' +E'	104	7.8	-	-	-	
						Max Compression w/ corresponding moment	1942	D + F + L + H' +E'	-198	-575							
						Max Moment with axial tension	1871	D + F + L + H' +E'	13	-325							
						Max Moment with axial compression	1955	D + F + L + H' +E'	-187	-879							
					3-H-L	Max Tension w/ corresponding moment	1884	D + F + L + H' +E'	32	-67	D + F + L + H' +E'	104	7.8	-	-	-	
						Max Compression w/ corresponding moment	1954	D + F + L + H' +E'	-200	-849							
						Max Moment with axial tension	1884	D + F + L + H' +E'	11	-344							
						Max Moment with axial compression	1968	D + F + L + H' +E'	-188	-892							
			Vertical	3H.6-191	1-V-L	Max Tension w/ corresponding moment	1857	D + F + L + H' +E'	144	-67	D + F + L + H' +E'	99	4.68	-	-	-	
						Max Compression w/ corresponding moment	1857	D + F + L + H' +E'	-241	-21							
						Max Moment with corresponding axial tension	1869	D + F + L + H' +E'	34	-271							
						Max Moment with corresponding axial compression	1869	D + F + L + H' +E'	-71	-300							
					2-V-L	Max Tension w/ corresponding moment	1860	D + F + L + H' +E'	80	-225	D + F + L + H' +E'	76	7.8	-	-	-	
						Max Compression w/ corresponding moment	1860	D + F + L + H' +E'	-123	-20							
						Max Moment with corresponding axial tension	1865	D + F + L + H' +E'	73	-723							
						Max Moment with corresponding axial compression	1867	D + F + L + H' +E'	-1	-600							
		Far side	Horizontal	3H.6-192	1-H-L	Max Tension w/ corresponding moment	1871	D + F + L + H' +E'	37	149	D + F + L + H' +E'	104	3.12	-	-	-	
						Max Compression w/ corresponding moment	1945	D + F + L + H' +E'	-193	92							
						Max Moment with axial tension	1883	D + F + L + H' +E'	4	198							
						Max Moment with axial compression	1964	D + F + L + H' +E'	-161	396							
			Vertical	3H.6-193	1-V-L	Max Tension w/ corresponding moment	1857	D + F + L + H' +E'	123	7	D + F + L + H' +E'	99	4.68	-	-	-	
						Max Compression w/ corresponding moment	1857	D + F + L + H' +E'	-241	30							
						Max Moment with corresponding axial tension	1922	D + F + L + H' +E'	52	324							
						Max Moment with corresponding axial compression	1919	D + F + L + H' +E'	-1	315							
		-	Horizontal Plane	3H.6-194	1-H-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	97	0.31 (#5 @12)	
			Vertical Plane	3H.6-194	1-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	132	0.62 (#5 @6)	
				3H.6-194	2-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	113	0.31 (#5 @12)	
				3H.6-194	3-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	95	0.31 (#5 @12)	
				3H.6-194	4-V-T	-	-	-	-	-	-	-	-	D + F + L + H' +E'	124	0.62 (#5 @6)	

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads – Load Combination	In-plane Shear (5) (kips / ft)	Longitudinal Reinforcement Provided (in²/ft)	Transverse Shear Design Loads – Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
Wall 14	2	Near Side	Horizontal	3H.6-195	1-H-L	Max Tension w/ corresponding moment	1579	D + F + L + H +Wt	55	-10	D + F + L + H +Wt	28	3.12	-	-	-	
						Max Compression w/ corresponding moment	1496	D + F + L + H' +E'	-153	-32							
						Max Moment with axial tension	1653	D + F + L + H' +E'	6	-68							
						Max Moment with axial compression	1652	D + F + L + H' +E'	-124	-81							
			Vertical	3H.6-196	1-V-L	Max Tension w/ corresponding moment	1652	D + F + L + H +Wt	102	-27	D + F + L + H +Wt	39	3.12	-	-	-	
						Max Compression w/ corresponding moment	1654	D + F + L + H' +E'	-139	-10							
						Max Moment with corresponding axial tension	1652	D + F + L + H' +E'	66	-71							
						Max Moment with corresponding axial compression	1652	D + F + L + H' +E'	-28	-71							
		Far side	Horizontal	3H.6-197	1-H-L	Max Tension w/ corresponding moment	1496	D + F + L + H' +E'	52	41	D + F + L + H +Wt	28	3.12	-	-	-	
						Max Compression w/ corresponding moment	1503	D + F + L + H +Wt	-174	28							
						Max Moment with axial tension	1652	D + F + L + H' +E'	49	54							
						Max Moment with axial compression	1543	D + F + L + H +Wt	-75	66							
			Vertical	3H.6-198	1-V-L	Max Tension w/ corresponding moment	1654	D + F + L + H' +E'	63	10	D + F + L + H +Wt	39	3.12	-	-	-	
						Max Compression w/ corresponding moment	1652	D + F + L + H +Wt	-204	74							
						Max Moment with corresponding axial tension	1508	D + F + L + H' +E'	1	58							
						Max Moment with corresponding axial compression	1652	D + F + L + H' +E'	-201	96							
Wall 15	2	Near Side	Horizontal	3H.6-200	1-H-L	Max Tension w/ corresponding moment	1808	D + F + L + H +Wt	65	-9	D + F + L + H' +E'	28	1.56	-	-	-	
						Max Compression w/ corresponding moment	1840	D + F + L + H +Wt	-90	-2							
						Max Moment with axial tension	1845	D + F + L + H' +E'	16	-86							
						Max Moment with axial compression	1845	D + F + L + H' +E'	-27	-102							
			Vertical	3H.6-201	1-V-L	Max Tension w/ corresponding moment	1689	D + F + L + H +Wt	75	-26	D + F + L + H +Wt	34	2.08	-	-	-	
						Max Compression w/ corresponding moment	1796	D + F + L + H +Wt	-107	-10							
						Max Moment with corresponding axial tension	1689	D + F + L + H' +E'	49	-38							
						Max Moment with corresponding axial compression	1796	D + F + L + H' +E'	-9	-42							
		Far side	Horizontal	3H.6-202	1-H-L	Max Tension w/ corresponding moment	1843	D + F + L + H +Wt	24	1	D + F + L + H' +E'	28	1.56	-	-	-	
						Max Compression w/ corresponding moment	1696	D + F + L + H +Wt	-194	20							
						Max Moment with axial tension	1728	D + F + L + H' +E'	0	42							
						Max Moment with axial compression	1784	D + F + L + H +Wt	-86	67							
			Vertical	3H.6-203	1-V-L	Max Tension w/ corresponding moment	1702	D + F + L + H' +E'	56	6	D + F + L + H +Wt	34	2.08	-	-	-	
						Max Compression w/ corresponding moment	1796	D + F + L + H +Wt	-106	6							
						Max Moment with corresponding axial tension	1785	D + F + L + H' +E'	0	54							
						Max Moment with corresponding axial compression	1696	D + F + L + H +Wt	-29	79							
Wall 16	2	Near Side	Horizontal	3H.6-204	1-H-L	Max Tension w/ corresponding moment	1455	D + F + L + H' +E'	13	-2	D + F + L + H +Wt	51	1.56	-	-	-	
						Max Compression w/ corresponding moment	1447	D + F + L + H +Wt	-48	-8							
						Max Moment with axial tension	1492	D + F + L + H +Wt	5	-15							
						Max Moment with axial compression	1470	D + F + L + H +Wt	-41	-25							
			Vertical	3H.6-205	1-V-L	Max Tension w/ corresponding moment	1450	D + F + L + H +Wt	61	-6	D + F + L + H +Wt	35	1.56	-	-	-	
						Max Compression w/ corresponding moment	1491	D + F + L + H +Wt	-51	-26							
						Max Moment with corresponding axial tension	1455	D + F + L + H +Wt	6	-25							
						Max Moment with corresponding axial compression	1447	D + F + L + H +Wt	-31	-43							
		Far side	Horizontal	3H.6-206	1-H-L	Max Tension w/ corresponding moment	1447	D + F + L + H' +E'	21	3	D + F + L + H +Wt	51	1.56	-	-	-	
						Max Compression w/ corresponding moment	1490	D + F + L + H +Wt	-185	45							
						Max Moment with axial tension	1489	D + F + L + H +Wt	2	31							
						Max Moment with axial compression	1470	D + F + L + H +Wt	-40	77							
			Vertical	3H.6-207	1-V-L	Max Tension w/ corresponding moment	1451	D + F + L + H +Wt	82	11	D + F + L + H +Wt	35	1.56	-	-	-	
						Max Compression w/ corresponding moment	1478	D + F + L + H +Wt	-138	36							
						Max Moment with corresponding axial tension	1462	D + F + L + H +Wt	0	38							
						Max Moment with corresponding axial compression	1491	D + F + L + H +Wt	-50	79							

Table 3H.6-11: Results of DGFOS Vault Concrete Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads					Longitudinal Reinforcement Provided (in²/ ft)	Transverse Shear Design Loads		Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
								Axial and Flexure Loads			In-Plane Shear Loads						
								Load Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	Load Combination	In-plane (5) Shear (kips / ft)		Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)		
Wall 16	2	-	Horizontal Plane	3H.6-208	1-H-T	-	-	-	-	-	-	-	-	-	-	0.62 (#5 @6)	Transverse shear reinforcement provided due to tornado missile impact evaluation.
				3H.6-208	2-H-T	-	-	-	-	-	-	-	-	-	-	0.62 (#5 @6)	

Notes

(1) The reinforcement layout drawings show the various zones used to define the minimum reinforcement that will be provided based on finite element analysis results. Actual provided reinforcement based on final rebar layout and including development length may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported boundaries. The dimensions in the reinforcement drawings are based on the dimensions of the 2D SAP2000 shell elements, which are modeled at the centerline of the walls and slabs. Therefore, the reinforcement drawing dimensions do not match actual building dimensions.

(2) Each reinforcement layout drawing is divided into reinforcement zones. The reinforcement zone naming convention is as follows: "H" = horizontal, "V" = vertical, "L" = longitudinal reinforcement, "T" = transverse reinforcement. For slabs, vertical corresponds to Y-axis and horizontal corresponds to X-axis as shown on Figure 3H.6-140.

(3) The maximum tension and compression axial forces are provided with the corresponding moment from the same load combination. The maximum moment that has a corresponding tension in the same load combination and the maximum moment that has a corresponding compression in the same load combination are also provided. For zones where either axial tension or axial compression does not occur for any load combination, dashes are input into the corresponding cell.

(4) Negative axial load is compression and positive axial load is tension. Negative moment applies tension to the top face of the shell element and positive moment applies tension to the bottom face of the shell element. For walls or slabs where the same reinforcement is provided on both faces, the moment is shown as absolute value.

(5) The reported in-plane shear is the maximum average in-plane shear along a plane that crosses the longitudinal reinforcement zone.

(6) The reported transverse shear is the maximum average transverse shear along a plane in that transverse reinforcement zone.

(7) In areas where horizontal and vertical transverse shear zones overlap, the total transverse shear reinforcement to be supplied in the overlapping area is the sum of the transverse reinforcement required from the horizontal and vertical zones.

(8) For certain areas of the structure, the standard element post-processing methods were too conservative. For such cases, detailed manual design was performed and the design forces determined by the detailed manual design are provided in the table.

(9) The reported forces are from the FEM analysis. The provided longitudinal reinforcement includes additional reinforcement required due to manual one-way design calculations.

(10) Element 553 and 566 were reported for Maximum Axial Tension w/ Corresponding Moment and Maximum Moment w/ Corresponding Axial Tension based on original analysis results. Element 553 shell element forces were averaged with Element 539 shell element forces as stated in Note 8. Element 566 shell element forces were averaged with Element 552 shell element forces as stated in Note 8. As a result of averaging, there were no PM points in Axial Tension; dashes are input into the corresponding cells.

Table 3H.6-12: Factors of Safety Against Sliding, Overturning, and Flotation for Diesel Generator Fuel Oil Storage Vaults

Load Combination	Calculated Safety Factor			Notes
	Overturning	Sliding	Flotation	
D + F'	---	---	1.28	2, 3
D + H + W	~~73.3~~1.5	~~63.1~~ 45.84	---	2, 3, 4
D + H + Wt	~~32.5~~1.41	~~27.3~~19.75	---	2, 3
D + H + E'	1.1	1.1	---	3, 4

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) Reported safety factors are conservatively based on considering empty weight of the fuel oil tank.

3) Coefficients of friction for sliding resistance are 0.58 for static conditions and 0.39 for dynamic conditions for the Diesel Generator Fuel Oil Storage Vault.

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

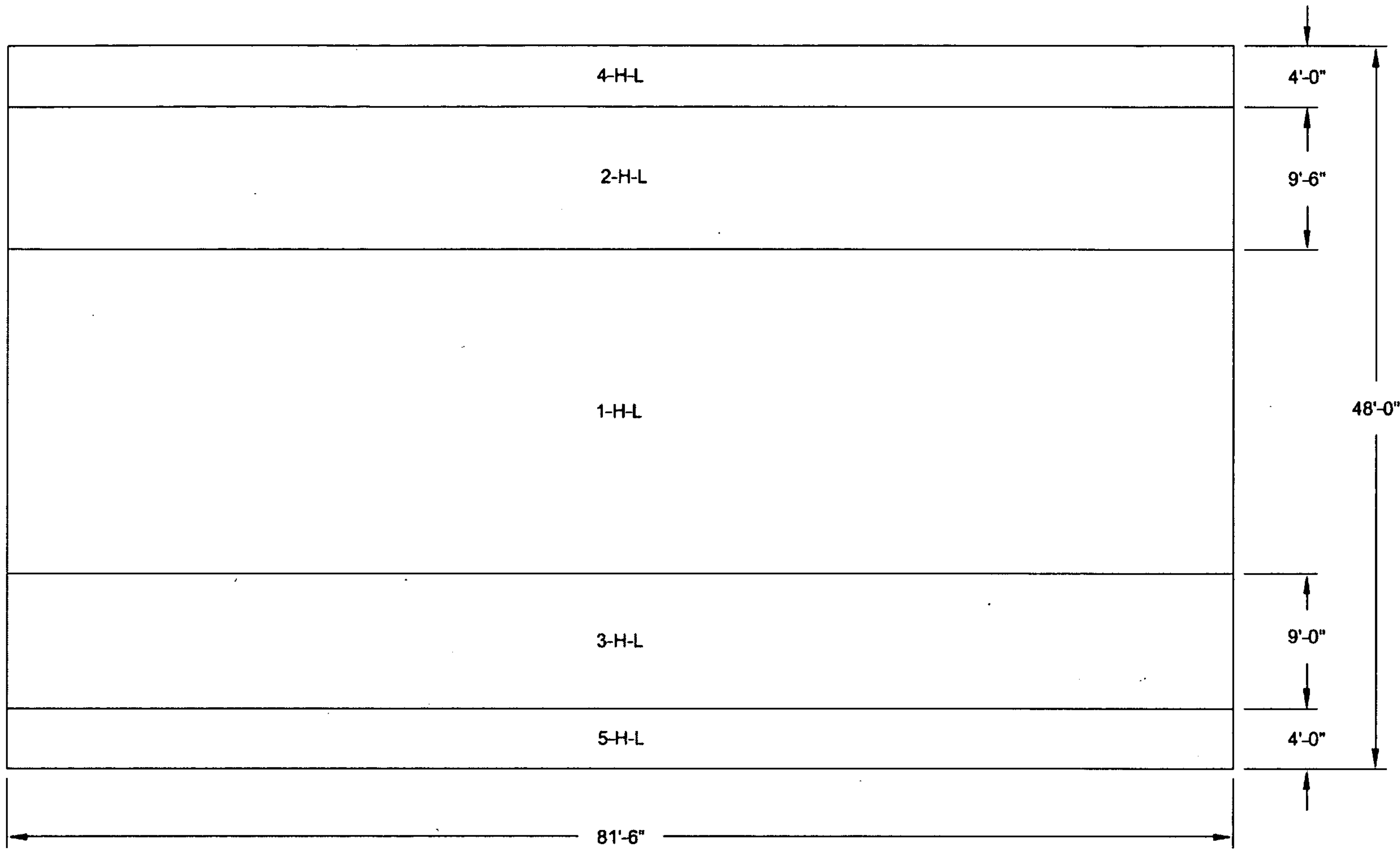


**Figure 3H.6-142 Slab 1 Looking Down
Horizontal Reinforcement Zones
Near Side Face**

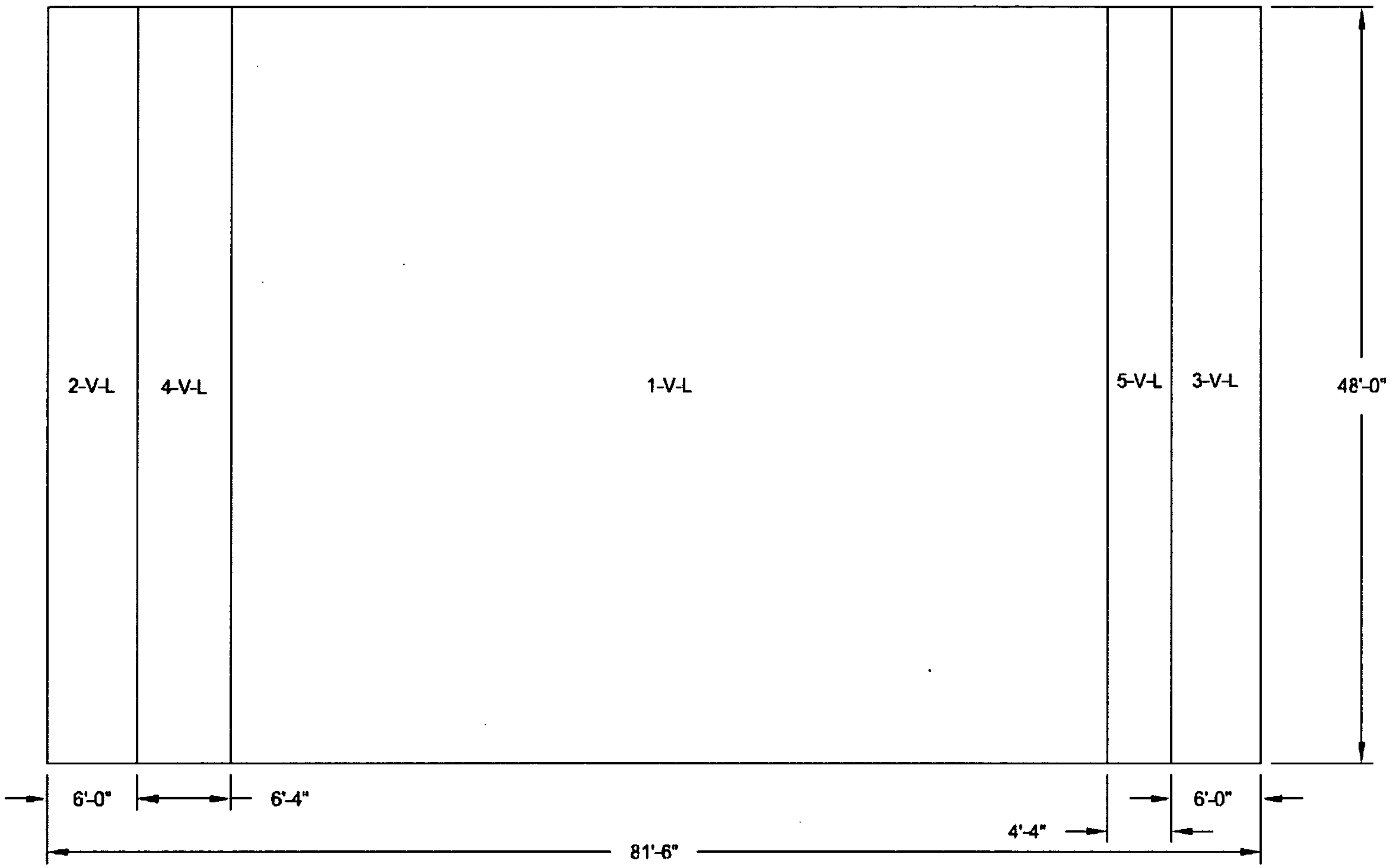


Figure 3H.6-143 Slab 1 Looking Down
Vertical Reinforcement Zones
Near Side Face

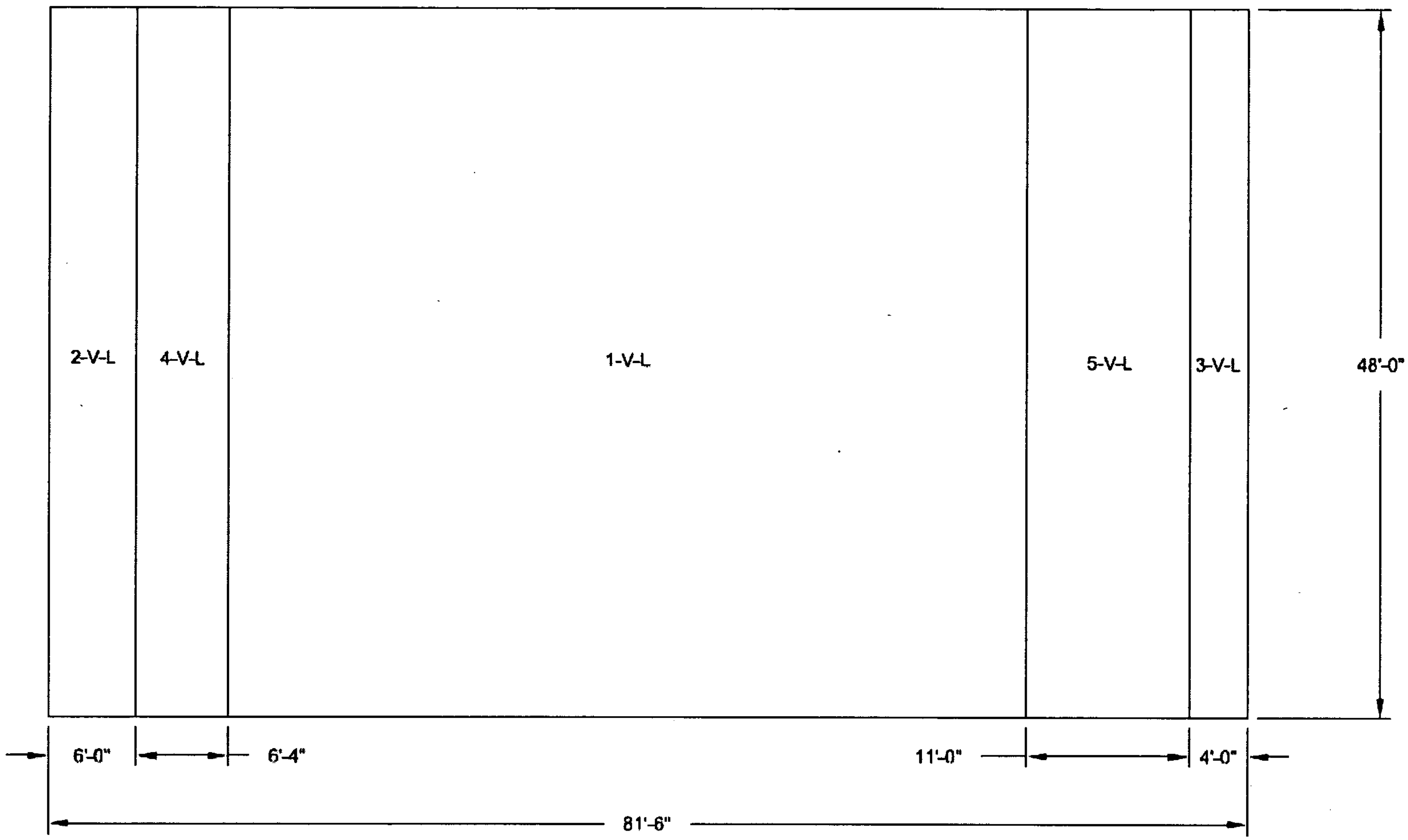


Figure 3H.6-145 Slab 1 Looking Down
Vertical Reinforcement Zones
Far Side Face

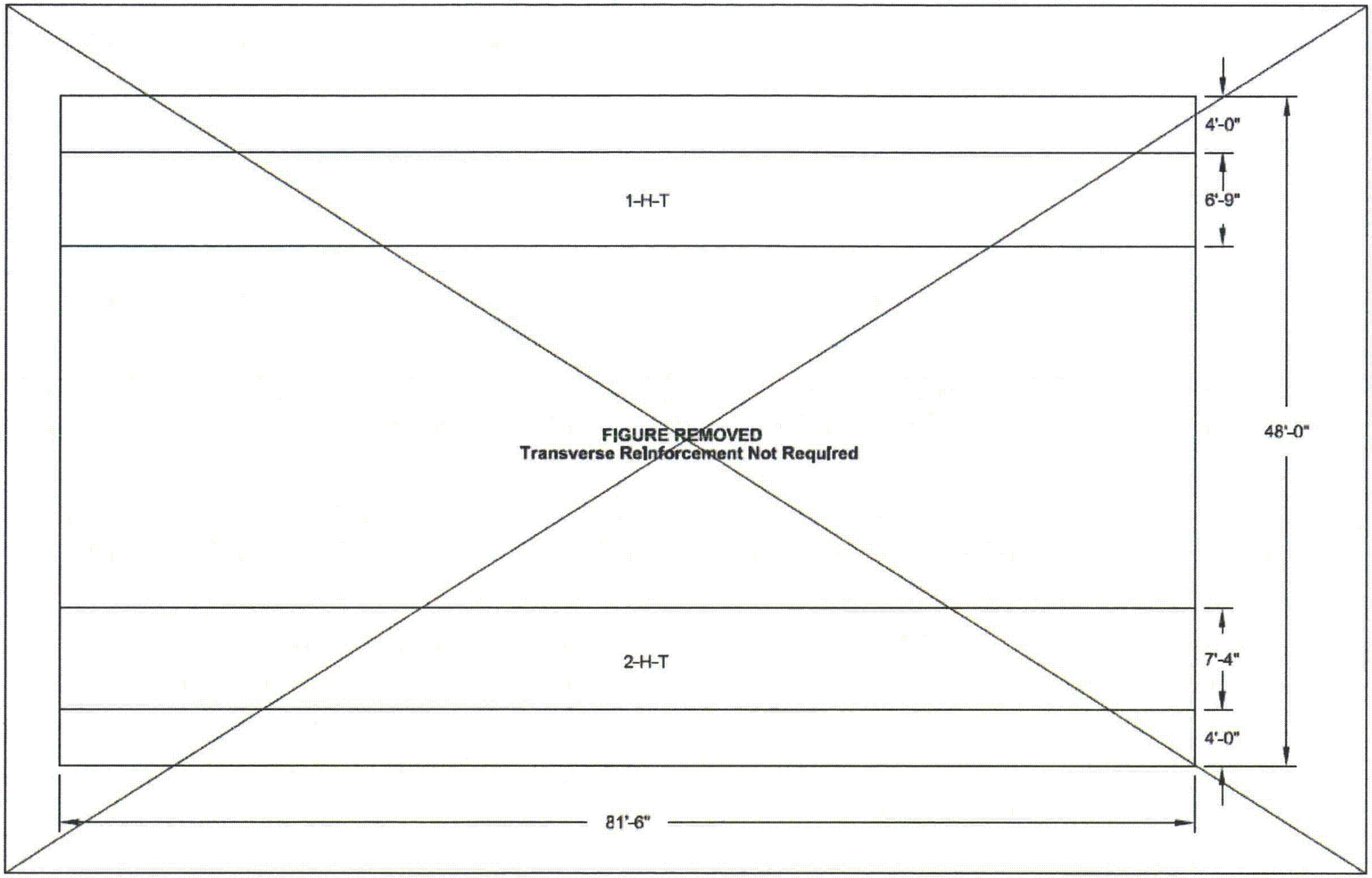


Figure 3H.6-146 ~~Slab 1 Looking Down~~

~~Transverse Reinforcement Zones~~ Not Used

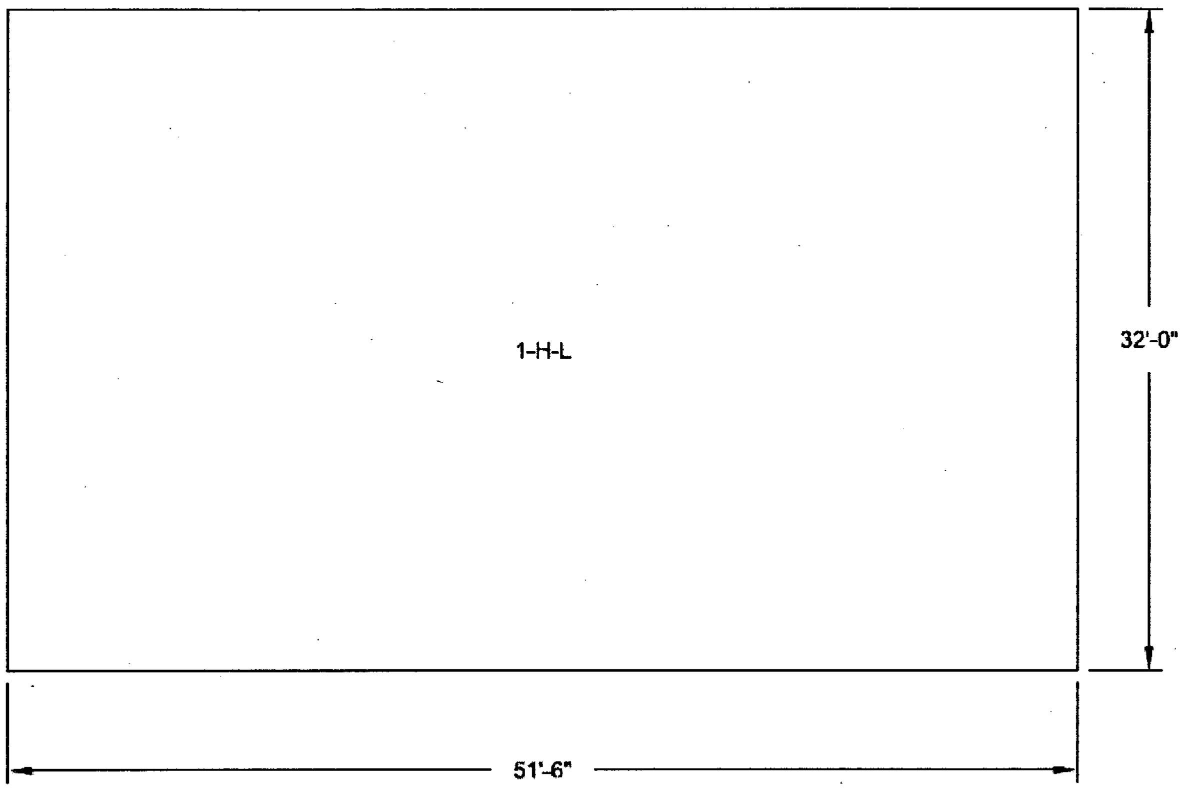


Figure 3H.6-147 Roof 2 Looking Down
Horizontal Reinforcement Zones
Near Side Face

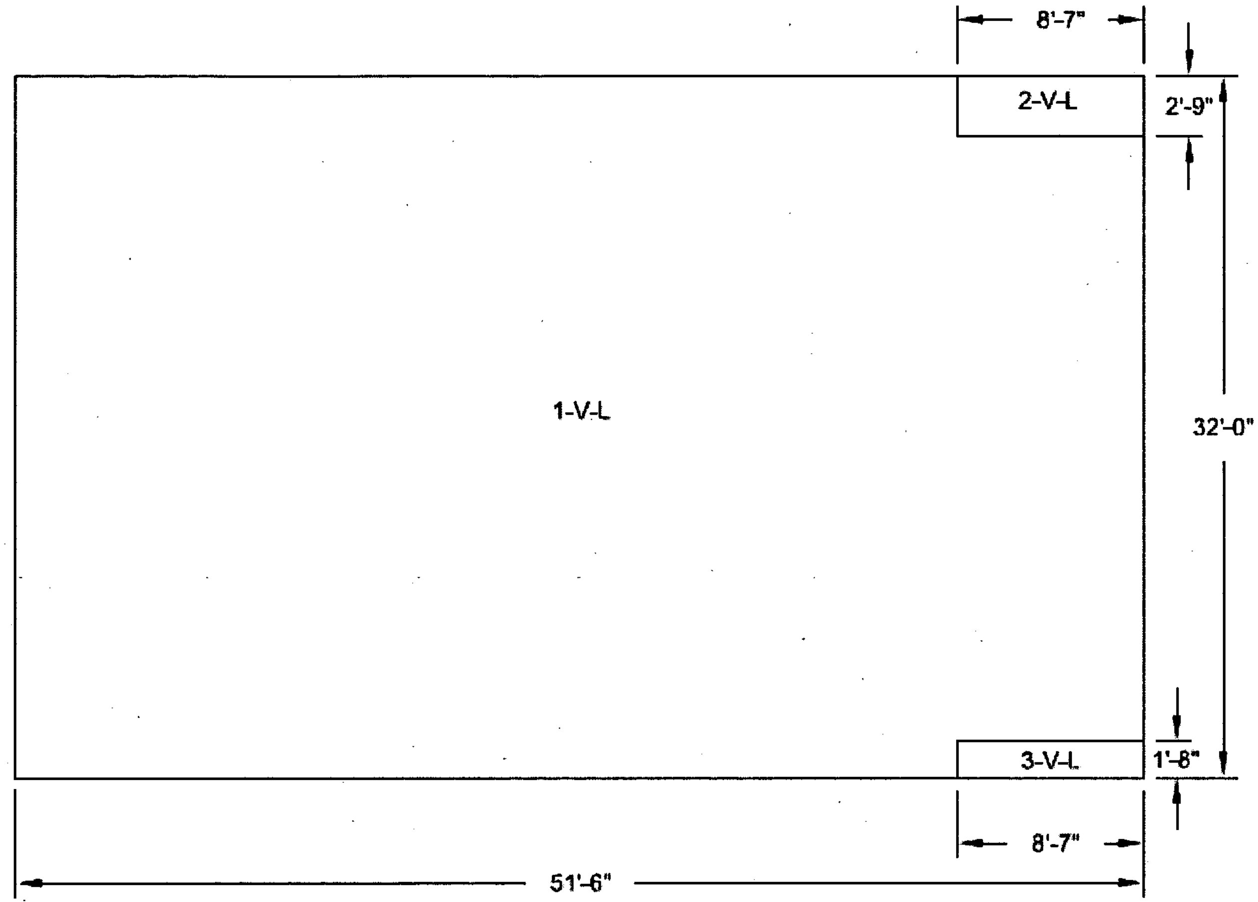


Figure 3H.6-148 Roof 2 Looking Down
Vertical Reinforcement Zones
Near Side Face

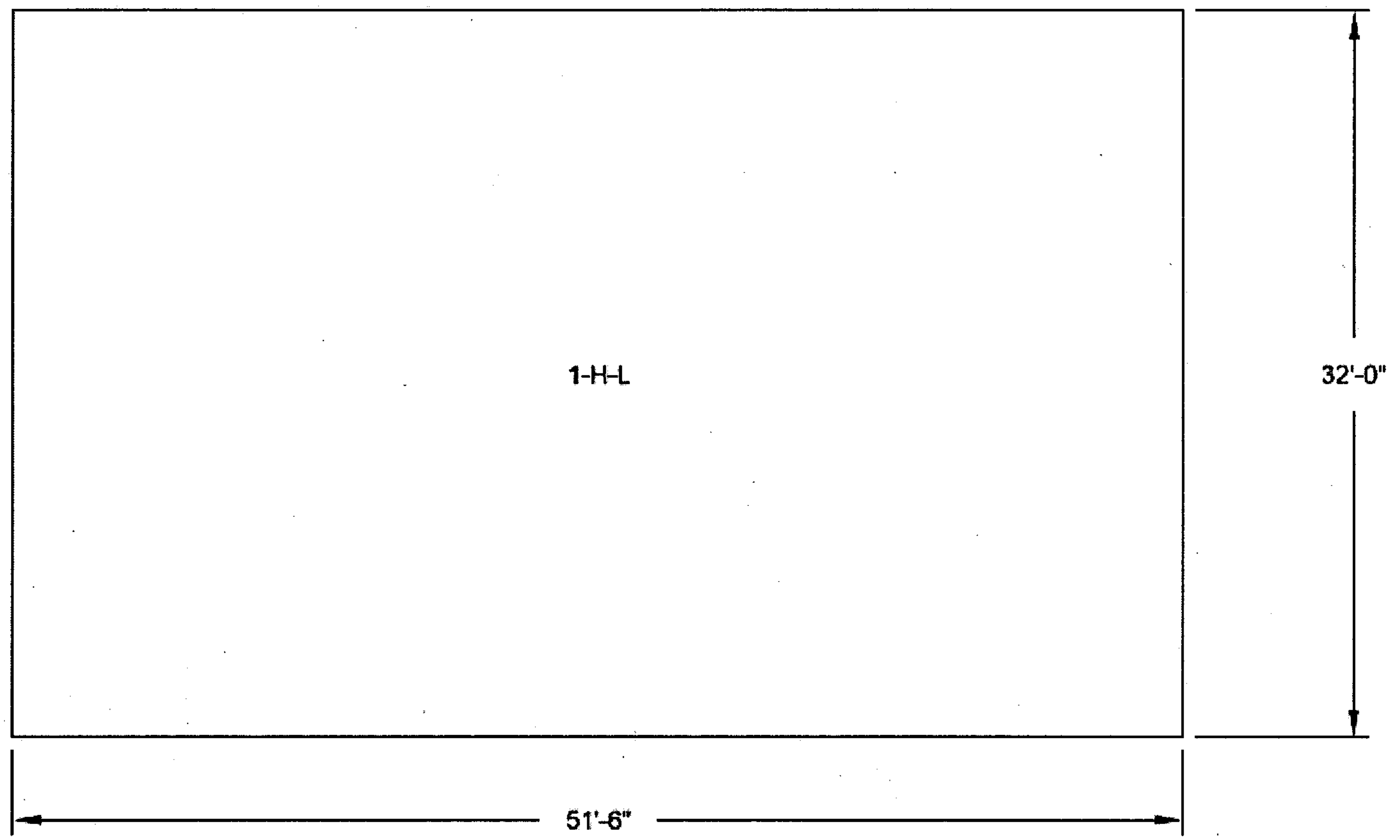


Figure 3H.6-149 Roof 2 Looking Down

Horizontal Reinforcement Zones

Far Side Face

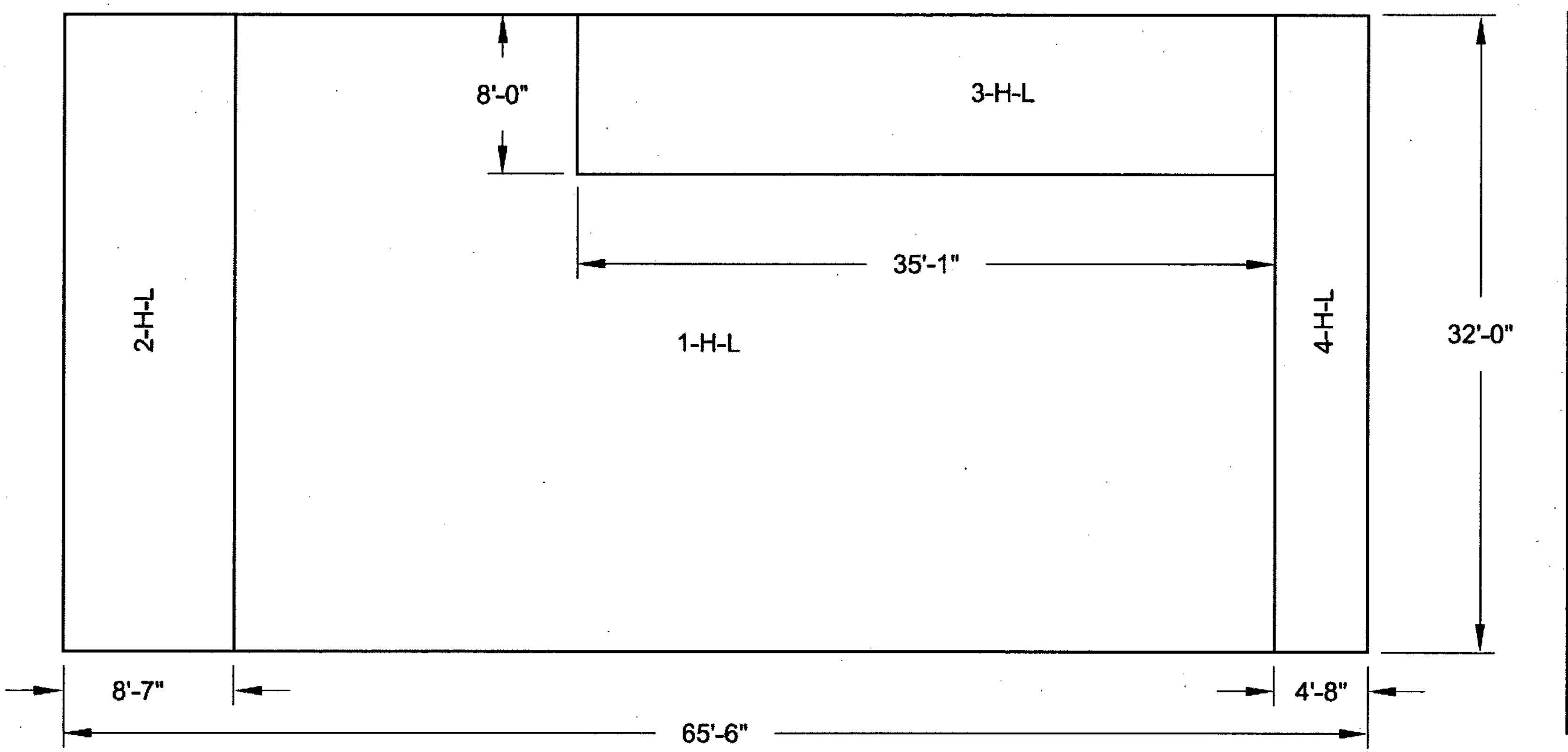


Figure 3H.6-163 Wall 7 Looking From Outside

Horizontal Reinforcement Zones

Near Side Face

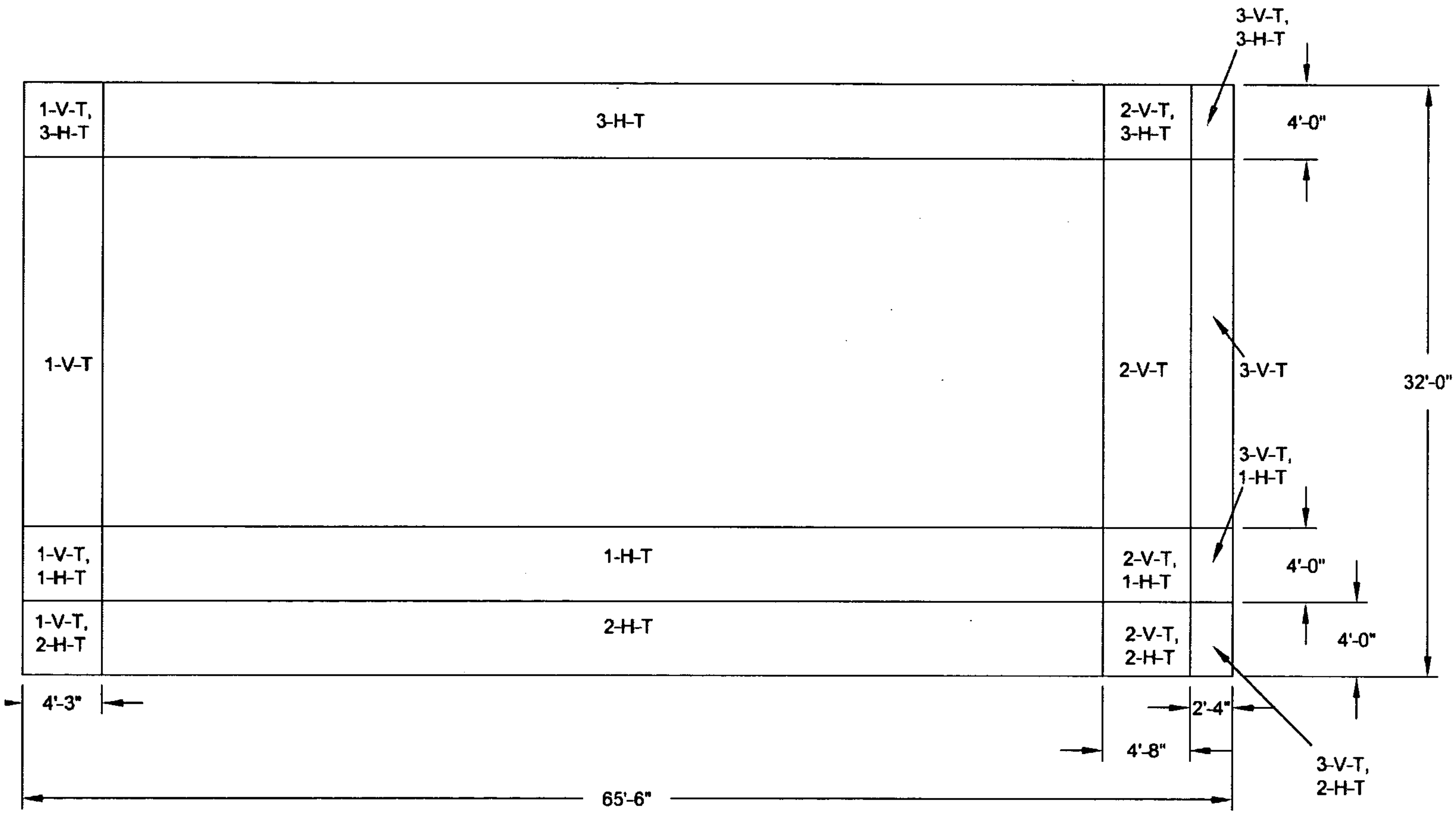


Figure 3H.6-167 Wall 7 Looking From Outside Transverse Reinforcement Zones

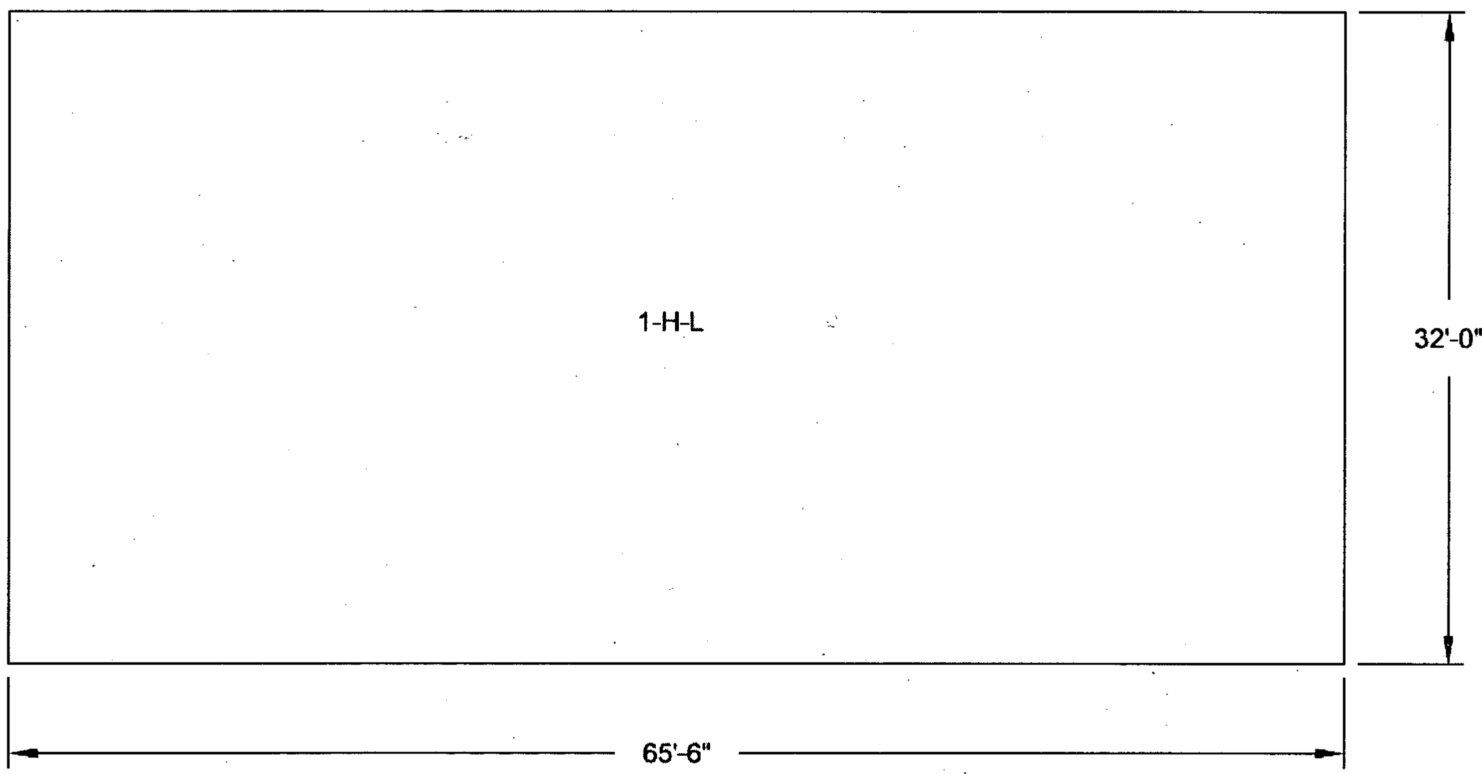


Figure 3H.6-170 Wall 8 Looking From Outside
Horizontal Reinforcement Zones
Far Side Face

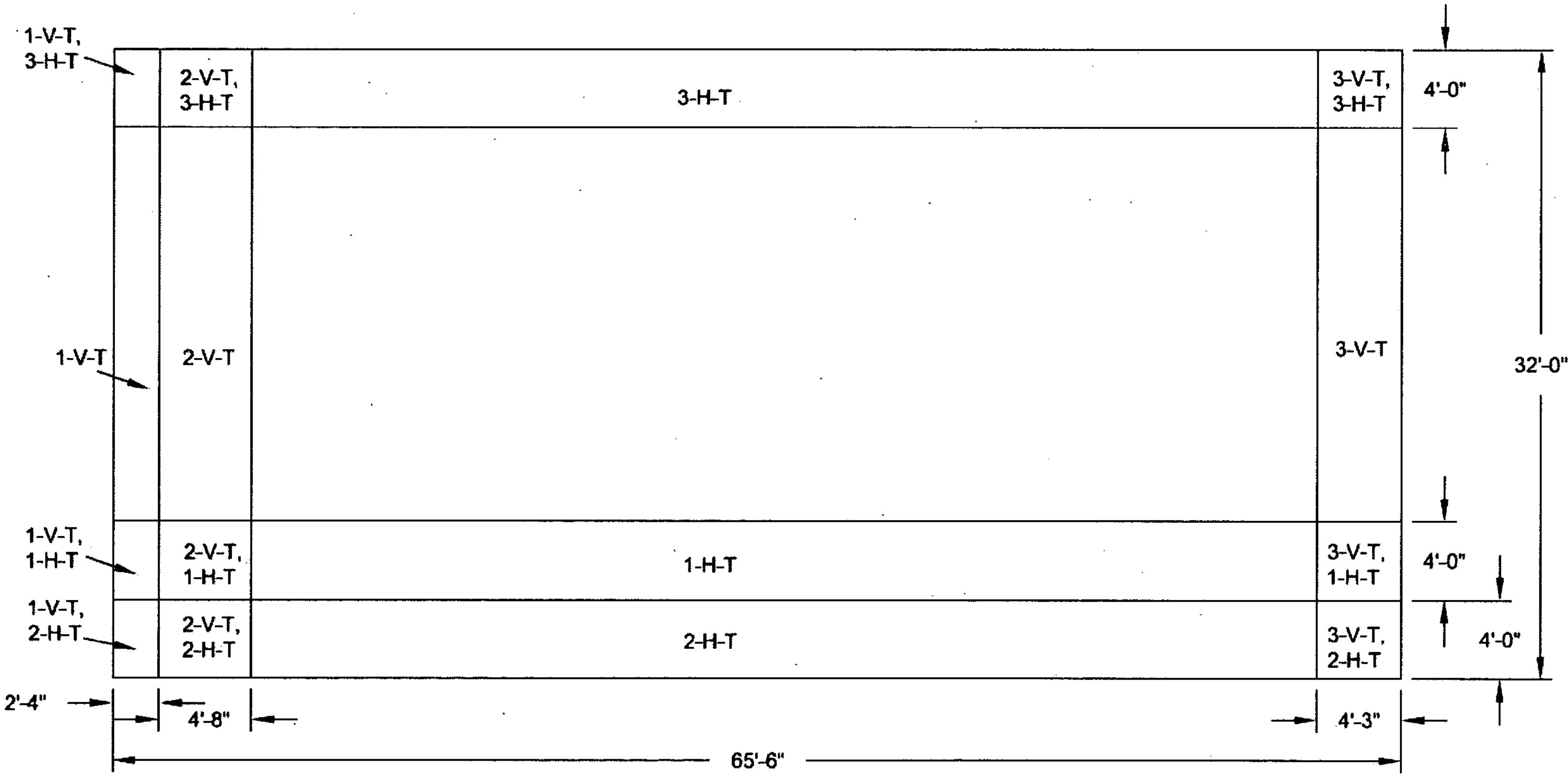


Figure 3H.6-172 Wall 8 Looking From Outside

Transverse Reinforcement Zones

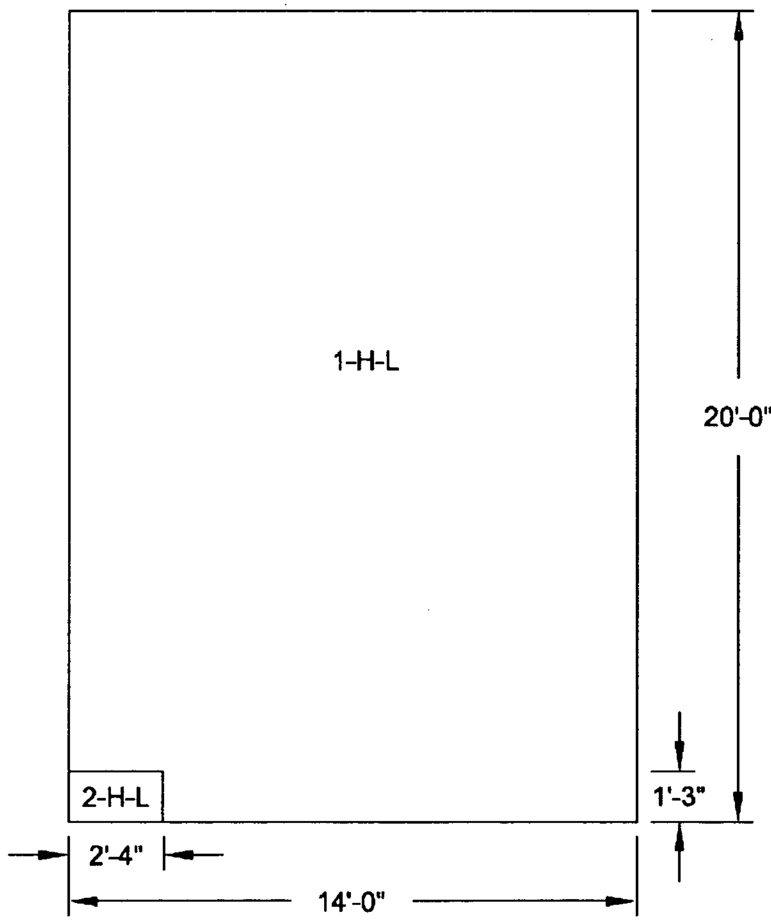


Figure 3H.6-173 Wall 9 Looking From Outside
Horizontal Reinforcement Zones
Near Side Face

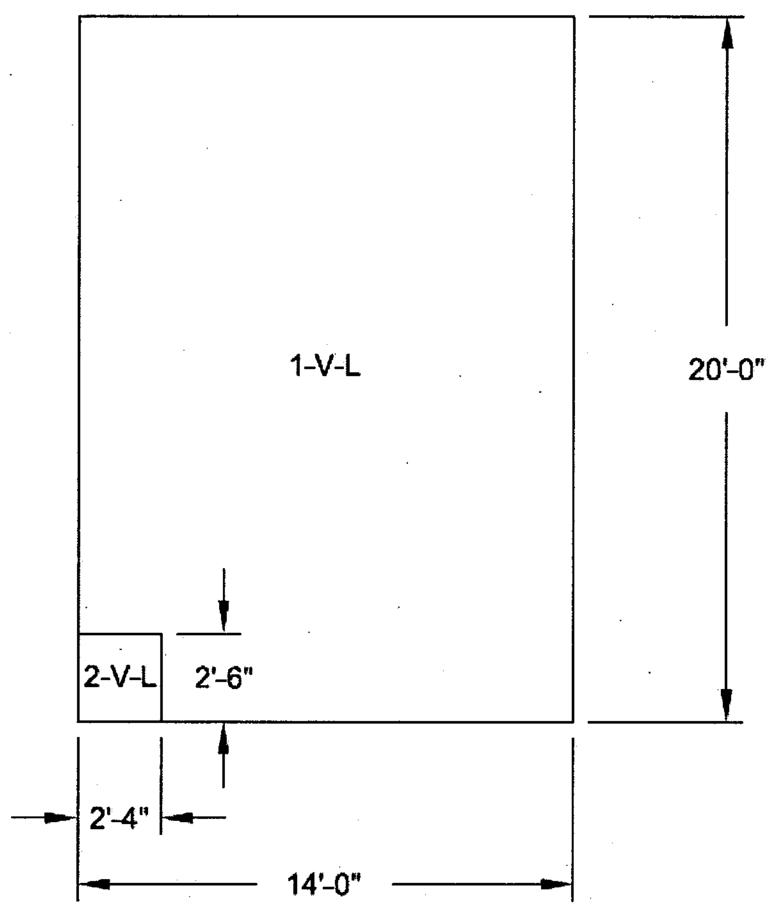


Figure 3H.6-174 Wall 9 Looking From Outside
Vertical Reinforcement Zones
Near Side Face

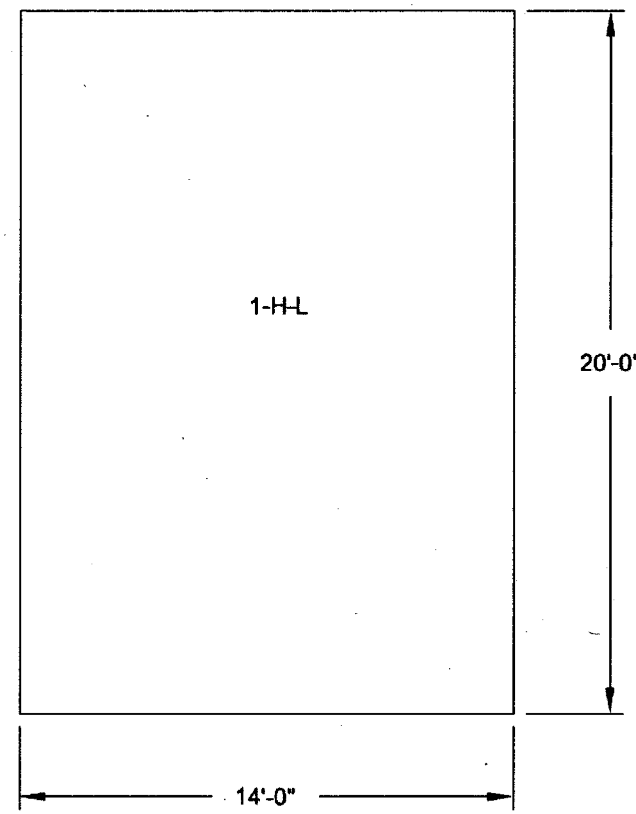


Figure 3H.6-175 Wall 9 Looking From Outside
Horizontal Reinforcement Zones
Far Side Face

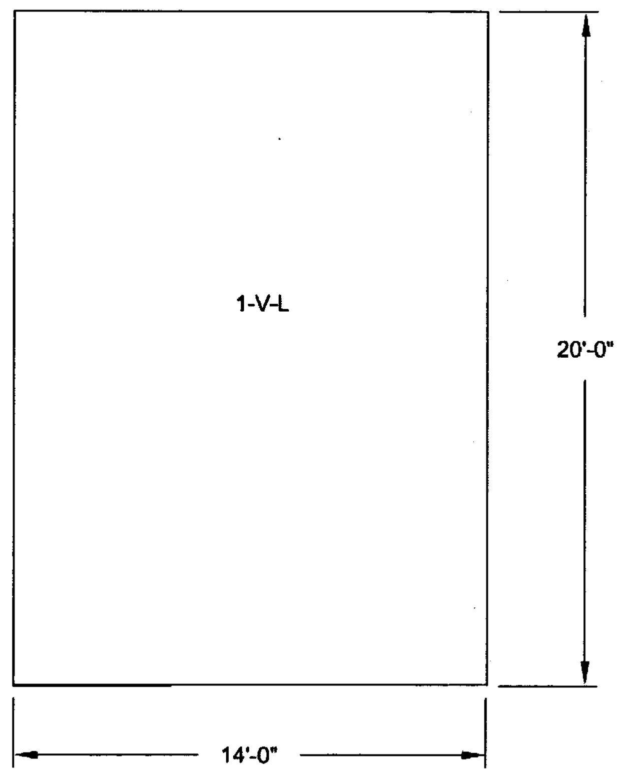


Figure 3H.6-176 Wall 9 Looking From Outside
Vertical Reinforcement Zones
Far Side Face

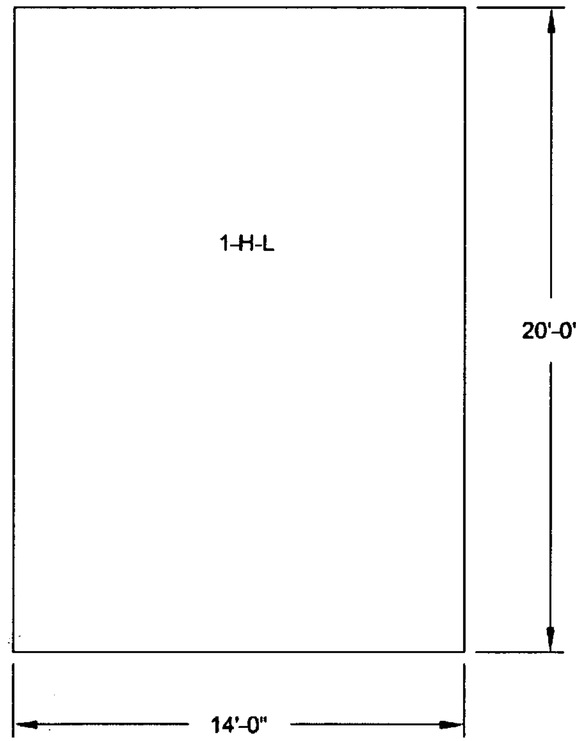


Figure 3H.6-177 Wall 10 Looking From Outside

Horizontal Reinforcement Zones

Near Side Face

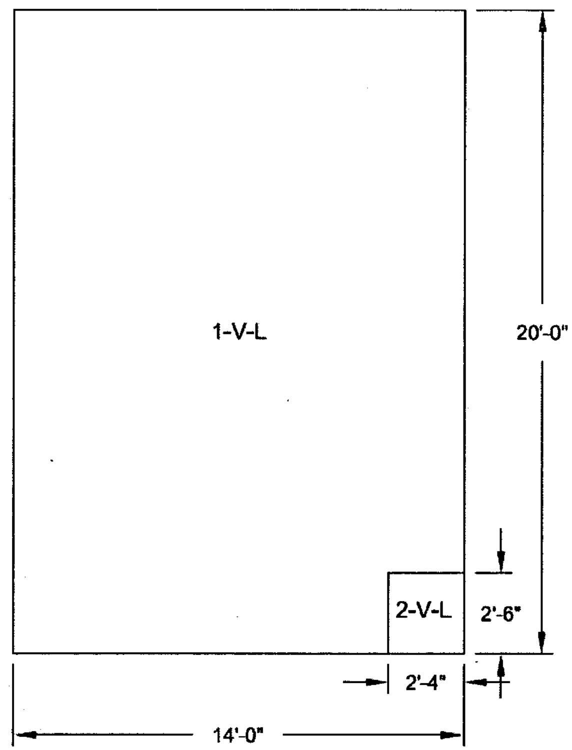


Figure 3H.6-178 Wall 10 Looking From Outside
Vertical Reinforcement Zones
Near Side Face

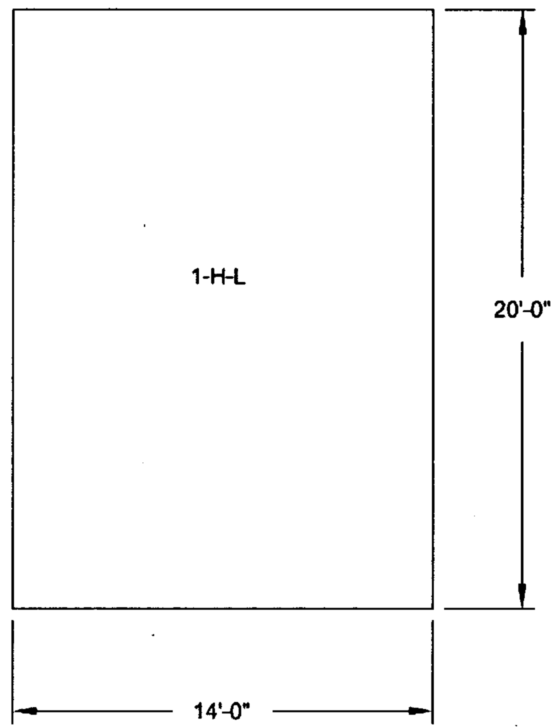


Figure 3H.6-179 Wall 10 Looking From Outside
Horizontal Reinforcement Zones
Far Side Face

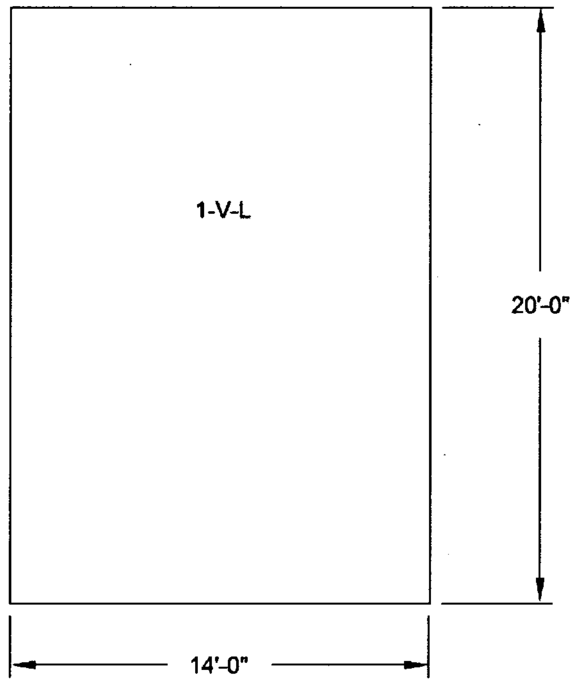


Figure 3H.6-180 Wall 10 Looking From Outside
Vertical Reinforcement Zones
Far Side Face

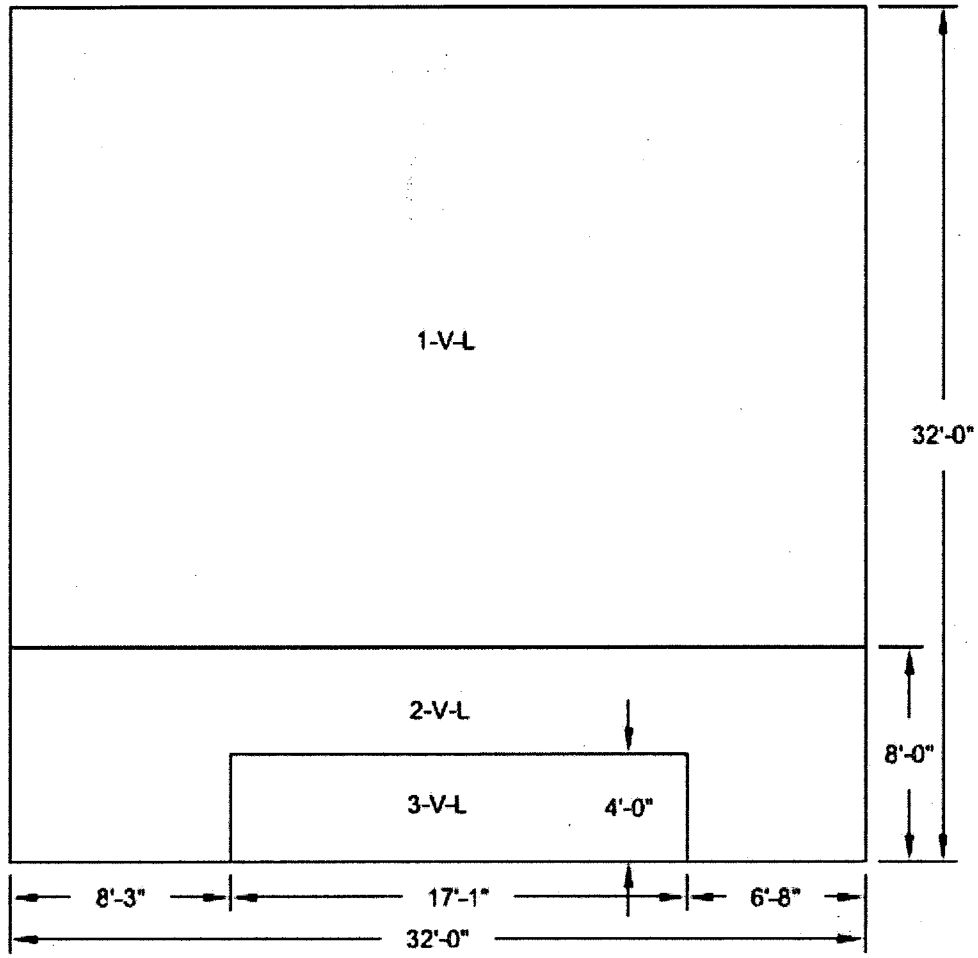


Figure 3H.6-186 Wall 12 Looking From Outside
Vertical Reinforcement Zones
Near Side Face

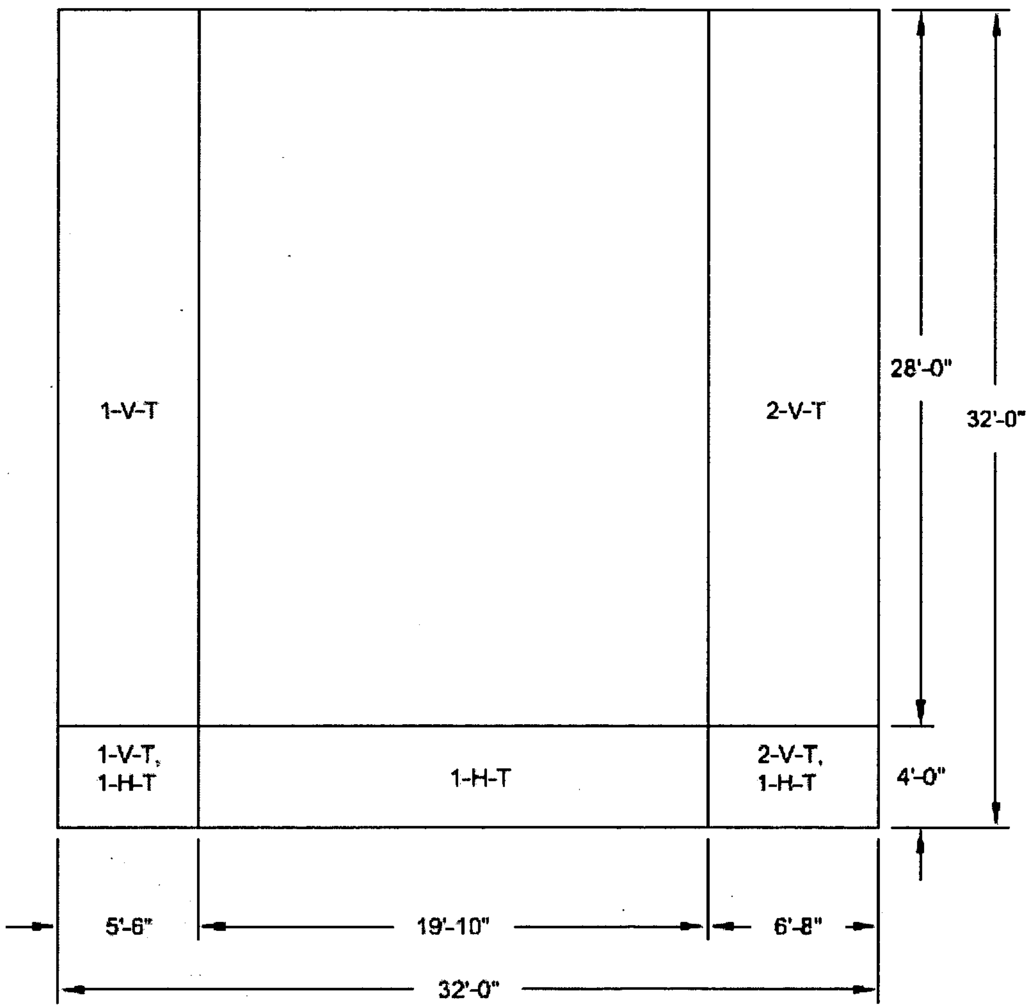


Figure 3H.6-189 Wall 12 Looking From Outside Transverse Reinforcement Zones

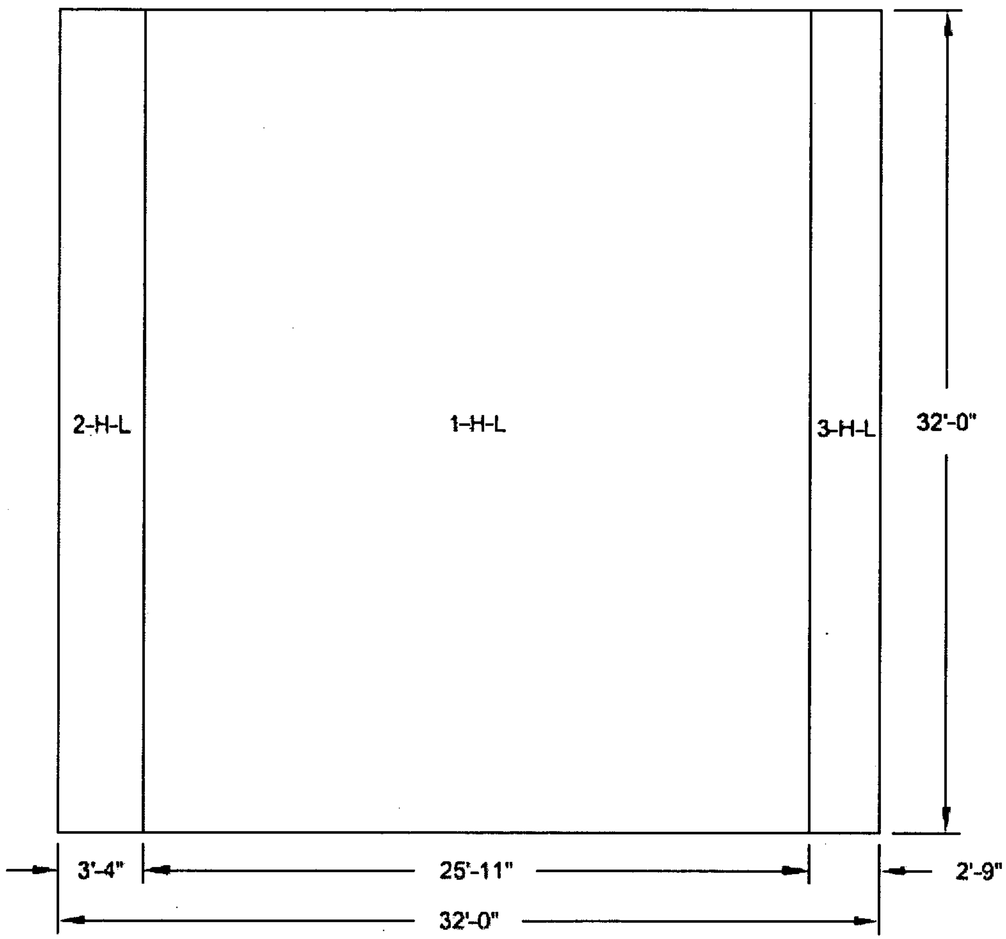


Figure 3H.6-190 Wall 13 Looking From Outside
Horizontal Reinforcement Zones
Near Side Face

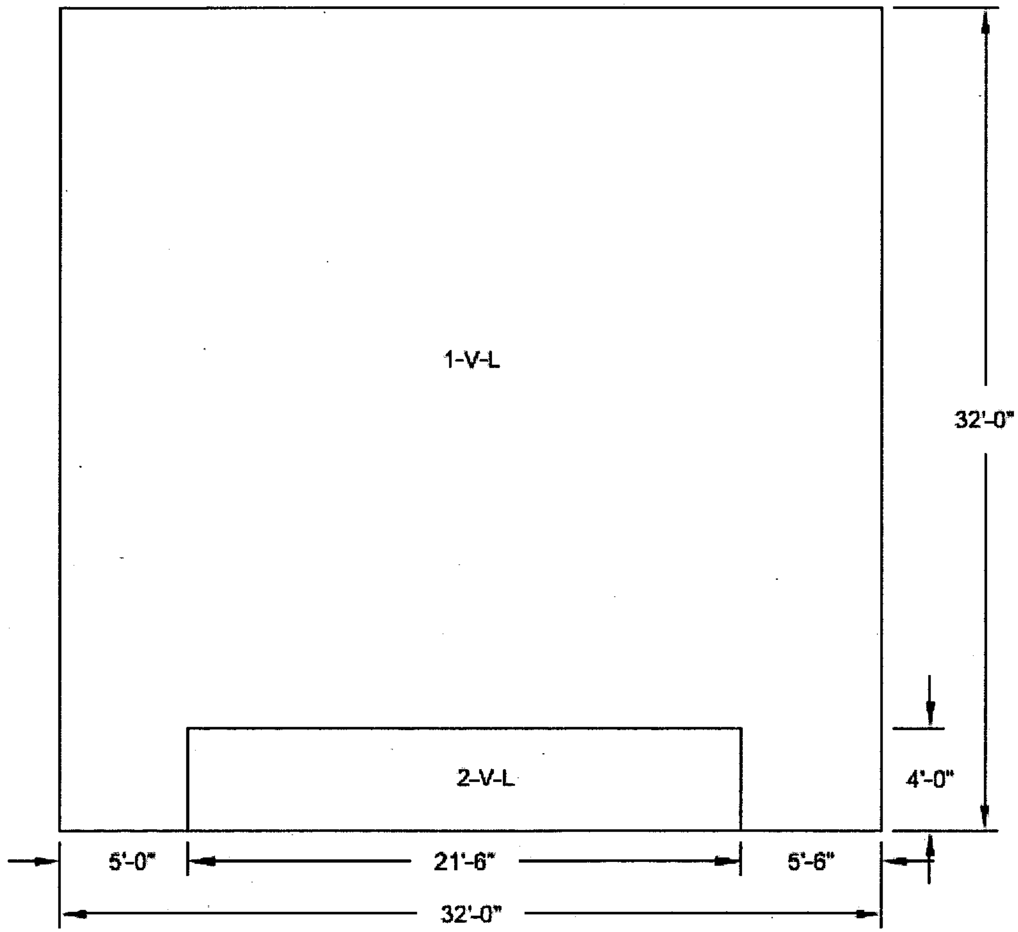


Figure 3H.6-191 Wall 13 Looking From Outside

Vertical Reinforcement Zones

Near Side Face

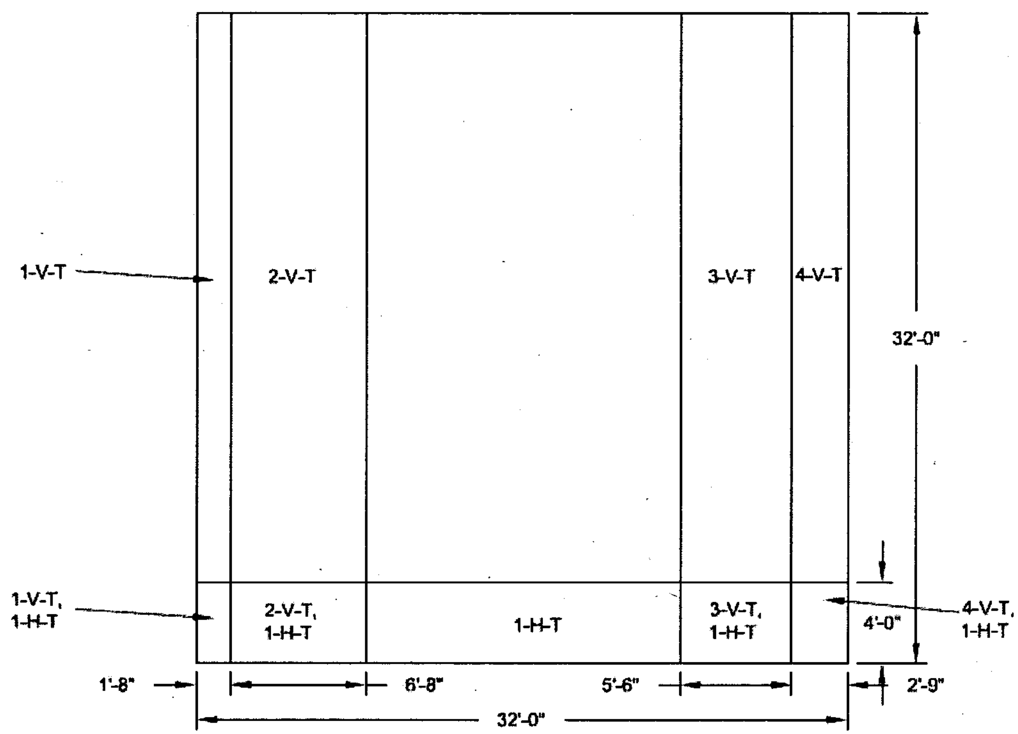


Figure 3H.6-194 Wall 13 Looking From Outside Transverse Reinforcement Zones

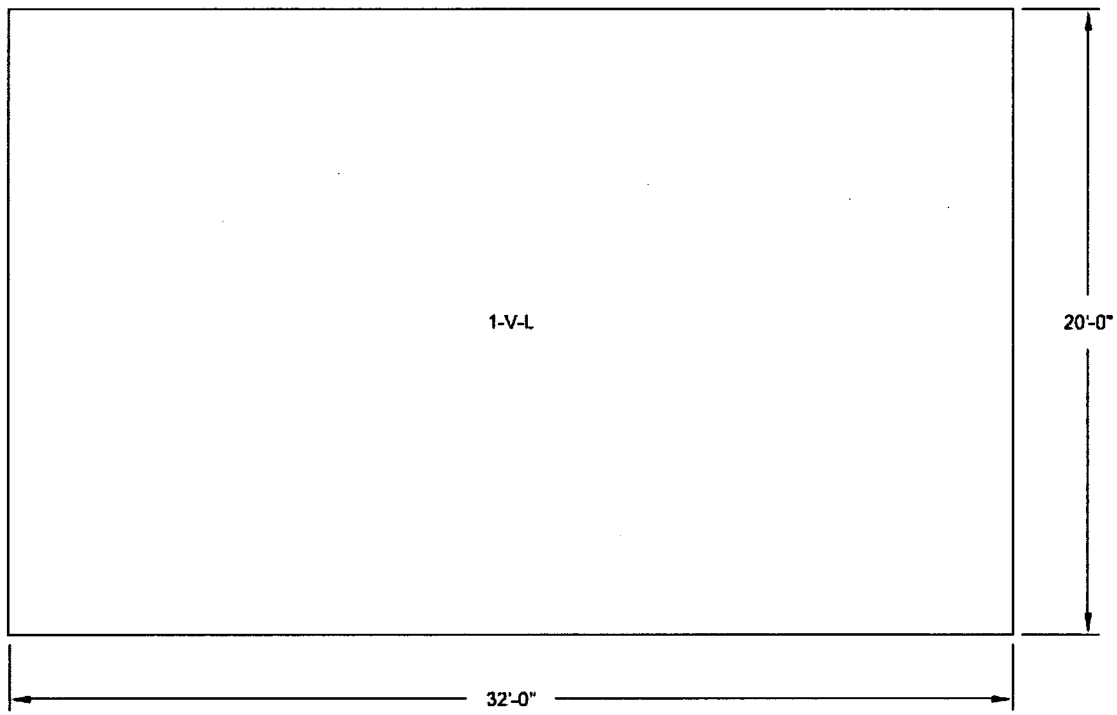


Figure 3H.6-196 Wall 14 Looking From Outside

Vertical Reinforcement Zones

Near Side Face

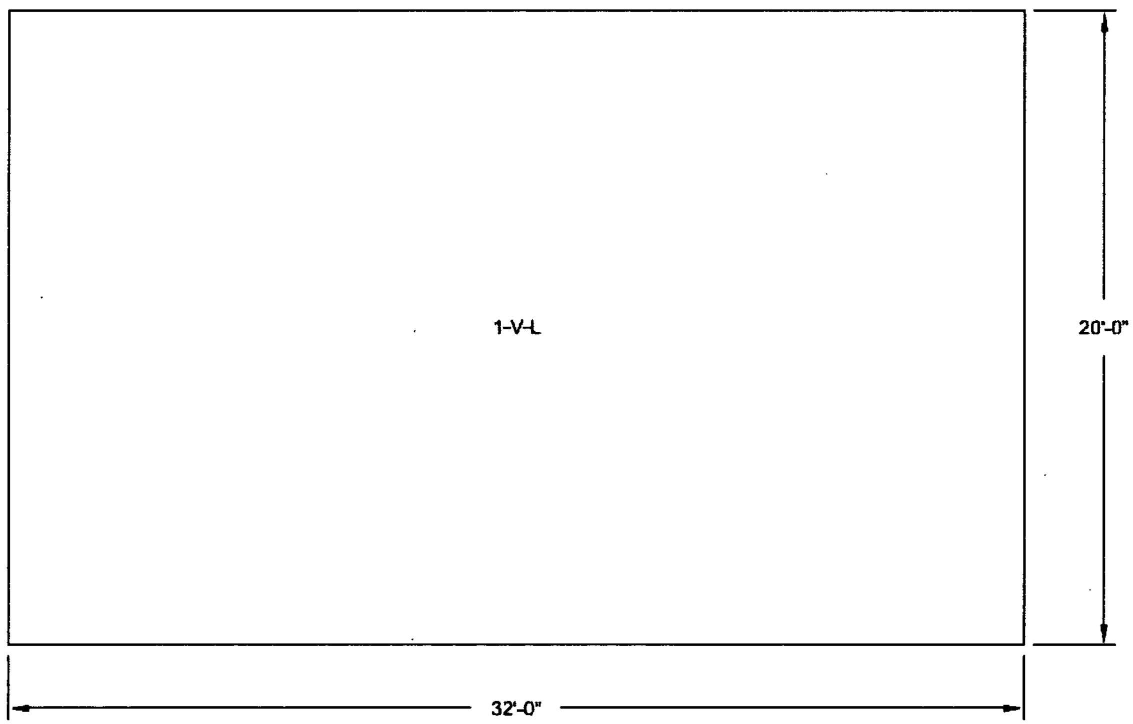


Figure 3H.6-198 Wall 14 Looking From Outside

Vertical Reinforcement Zones

Far Side Face

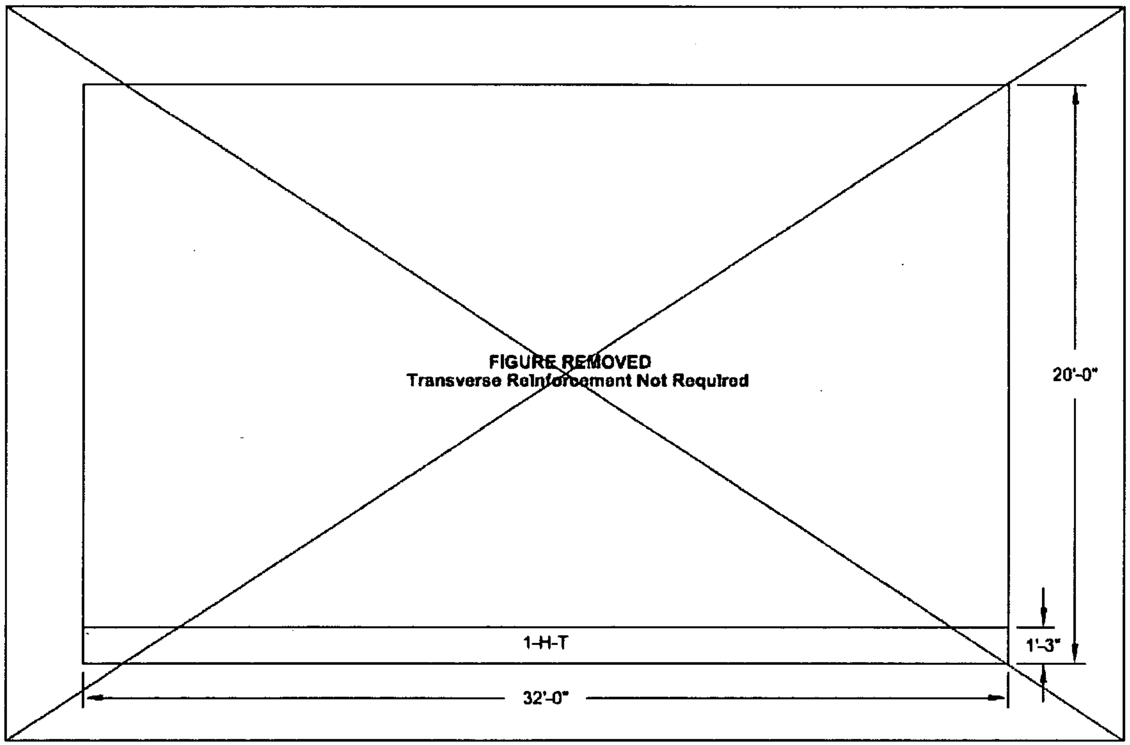


Figure 3H.6-199 ~~Wall 14 Looking From Outside Transverse Reinforcement Zones~~ Not Used

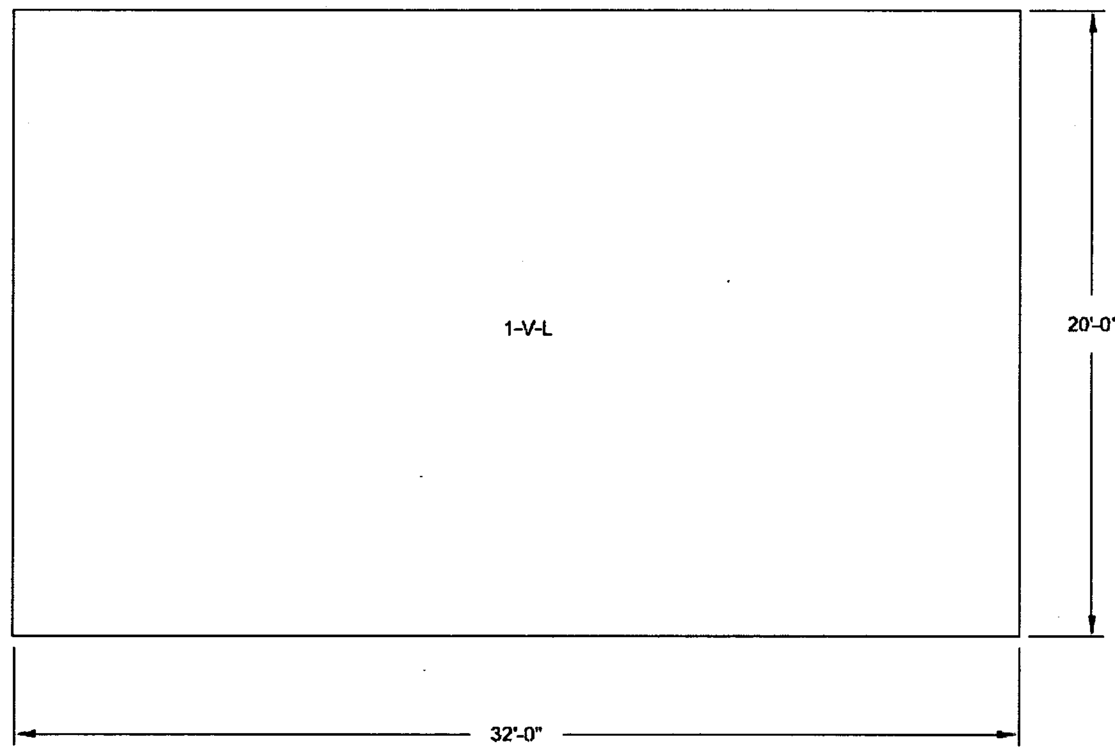


Figure 3H.6-201 Wall 15 Looking From Outside
Vertical Reinforcement Zones
Near Side Face

Enclosure 4

Revisions to COLA Section 3H.7

3H.7 Diesel Generator Fuel Oil Tunnel

3H.7.1 Objective and Scope

The scope of this section is to document the structural design and analysis of the Diesel Generator Fuel Oil Tunnels (DGFOTs) for STP Units 3 & 4.

3H.7.2 Summary

The following are the major summary conclusions on the design and analysis of the DGFOT:

- The provided concrete reinforcement listed in Table 3H.7-1 meets the requirements of the design codes and standards listed in Section 3H.7.4.1.
- The factors of safety against flotation, sliding and overturning of the structure under various loading combinations as shown in Table 3H.7-2 are higher than the required minimum factors of safety.
- The thickness of the exterior walls and roof slabs are more than the minimum required to preclude penetration, perforation, or spalling due to impact of design basis tornado missiles.

3H.7.3 Structural Description

The layout of the Diesel Generator Fuel Oil Tunnels (DGFOTs) is as shown in Figure 3H.6-221. There are three (3) reinforced concrete DGFOTs approximately 50 ft, 200 ft, and 220 ft long for each unit. Each DGFOT is connected at one end to the Reactor Building (RB) and at the other end to a Diesel Generator Fuel Oil Storage Vault (DGFOSV). There is a seismic gap between each of the DGFOT and the adjoining RB and DGFOSV. Table 3H.6-15 provides the magnitude of the required and provided seismic gaps at interface of DGFOTs and the adjoining RB and DGFOSVs.

Each DGFOT has two access regions which extend above grade; one access region is located where the tunnel interfaces with the DGFOSV and another where the tunnel interfaces with the RB. The access regions provide access to the below grade portions of the DGFOTs during maintenance and inspection. The overall above grade dimensions of the access regions are approximately 7.5 ft wide by 7.5 ft long and 15 ft high.

The top of the DGFOT is located approximately at grade. The DGFOT No. 1B, which is the shortest tunnel, running approximately 50 ft between the RB and DGFOSV No. 1B, has a wall thickness of 2'-0" on both sides. The interior below grade dimensions of this tunnel are approximately 7 ft high by 3.5 ft wide. The other two longer DGFOTs (approximately 200 ft and 220 ft long) have a wall thickness of 2'-0" on one side and 2'-6" on the other side to allow for placement of embedded conduits. The interior below grade dimensions of these tunnels are approximately 7 ft high by 3 ft wide. Any fuel leak from the fuel oil lines or water infiltration within the tunnels will be collected in a sump and removed by pumps. The tunnels slope away from the DGFOSV and the RB towards the sump located at the center of the tunnel runs.

3H.7.4 Structural Design Criteria

3H.7.4.1 Design Codes and Standards

The DGFOTs are designed to meet the design requirements of standard plant structures. The following codes, standards, and regulatory documents are applicable for the design of the DGFOT.

- ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary"
- ACI 349-97, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary"
- ASCE 7-88, "Minimum Design Loads for Buildings and Other Structures"
- NUREG-0800 SRP 3.3.2, "Tornado Loadings," Rev. 2, July 1981
- NRC RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments)," Rev 2, November 2001
- NRC RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," Rev 0, April 1974
- NUREG 0800 SRP 3.5.3 "Barrier Design Procedure", Revision 1, July 1981
- NUREG 0800 SRP 3.5.1.4 "Missiles Generated by Natural Pheonemena", Rev. 2, July 1981

3H.7.4.2 Site Design Parameters

3H.7.4.2.1 Soil Parameters

- Poisson's ratio (above groundwater)..0.42
- Poisson's ratio (below groundwater)..0.47
- Unit Weight (moist)..120 pcf
- Unit Weight (saturated)..140 pcf
- Liquefaction potential ....None

3H.7.4.2.2 Design Ground Water Level

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

3H.7.4.2.3 Design Flood Level

Design flood level is 33 feet MSL, as shown in DCD, Tier 1, Table 5.0. The external flood level due to MCR breach is shown in 3H.7.4.3.3.3.

3H.7.4.2.4 Maximum Snow Load

Roof snow load is 50 psf as shown in DCD Tier 1 Table 5.0. This snow load is above the value derived from ASCE 7-88 for the STP 3&4 site. This load is not combined with normal roof live load.

3H.7.4.2.5 Maximum Rainfall

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

3H.7.4.3 Design Load and Load Combinations

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

3H.7.4.3.1 Normal Loads

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

3H.7.4.3.1.1 Dead Loads (D)

Dead loads include the weight of the structure and other permanent static loads. An additional 50 psf uniform load is considered to account for dead loads due to piping on the DGFOT and access region walls.

3H.7.4.3.1.2 Live Loads (L)

Live loads include floor and roof area live loads and movable loads. A minimum normal floor live load of 200 psf is considered for the floor of the DGFOT. A normal live load of 50 psf is considered for the roof.

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

A surcharge load of 500 psf is applied to the top of the DGFOT at grade.

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.

3H.7.4.3.1.4 Thermal Load

The DGFOT is primarily below grade. The temperature of the DGFOT will remain essentially constant with the temperature of the surrounding soil. Therefore the thermal load condition is not applicable to the DGFOT.

3H.7.4.3.1.5 Internal Flood Load

The DGFOT contains sump pumps to keep the structure from flooding. The internal flooding condition is not applicable for the structural design of the DGFOT.

3H.7.4.3.2 Severe Environmental Load

Severe environmental loads consist of loads generated by wind.

3H.7.4.3.2.1 Wind Load (W)

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

- Basic wind speed (50 year recurrence interval, fastest mile)......110 mph (177 km/h), as shown in Table 2.0-2 as the ABWR Standard Plant Site Parameter.
- Exposure:...D
- Importance factor I:...1.11
- Velocity pressure exposure: ....0.00256Kz $(IV)^2$

Wind loads are calculated in accordance with the provisions of Chapter 6 of ASCE 7-88.

3H.7.4.3.3 Extreme Environmental Load

Extreme environmental loads consist of loads generated by tornado, SSE earthquake, extreme snow and flooding.

3H.7.4.3.3.1 Tornado Loads (W_t)

The following tornado load effects are considered in the design:

- Wind pressure: ....W_w
- Differential pressure: ....W_p
- Missile Impact: ....W_m

The tornado parameters used in the calculations of tornado loads are as follows:

- Maximum wind speed: ....300 mph
- Pressure differential: ....2 psi
- Radius of maximum rotational speed: ....150 feet
- Pressure differential rate: ....1.2 psi/sec
- Missile spectrum (per DCD Tier 2 Table 2.0-1) : ...

A : 4000 lbs automobile (16.4ft x 6.6ft x 4.3ft)

B: 276 lbs, 8" diameter armor piercing artillery shell

C: 1" diameter solid steel sphere..............

Notes:

(1) Tornado wind pressure (W_w)

(a). Wind velocity and wind pressure are constant with height.

(b). Wind velocity and wind pressure vary with horizontal distance from the center of the tornado.

(2) Tornado differential pressure (W_p)

The differential pressure is applied to the top of the tunnel slab and access region. The differential pressure causes suction on the exterior walls.

(3). Tornado missile impact (W_m)

Tornado missile impact effects on the structure are assessed as noted below:

(a). Local damage in terms of penetration, perforation, and spalling.

(b). Structural response in terms of deformation limits, strain energy capacity, structural integrity and structural stability.

(c) All missiles are considered to impact at 35% of the maximum horizontal tornado wind speed horizontally and 70% of horizontal impact velocity vertically.

(d) Barrier design is evaluated assuming a normal impact at the surface for the schedule 40 pipe and automobile missiles.

(e) The automobile missile is considered to impact at all attitudes less than 30 feet above grade level.

(4) Table 3H.7-3 contains the results of the tornado missile impact evaluation.

- Tornado load combinations

Tornado load effects are combined per USNRC Standard Review Plan, NUREG-800 Section 3.3.2 as follows:

$W_t = W_w$

$W_t = W_p$

$W_t = W_m$

$W_t = W_w + 0.5\ W_p$

$W_t = W_w + W_m$

$W_t = W_w + 0.5\ W_p + W_m$

3H.7.4.3.3.2 Earthquake (E')

The Safe Shutdown Earthquake (E') loads are applied in three mutually orthogonal directions – two horizontal directions and the vertical direction. The total structural response is predicted by combining the applicable maximum co-directional responses by the SRSS method.

3H.7.4.3.3.3 Extreme Environmental Flood (FL)

The design basis flood level is 40 feet, in accordance with Subsection 2.4S.2.2. The flood water unit weight, considering maximum sediment concentration, is 63.85 pcf per Section 2.4S.4.2.2.4.3. The design requirements for this flood, including hydrostatic, hydrodynamic, and floating debris loading, are included in Section 3.4.2.

3H.7.4.3.3.4 Lateral Soil Pressures Including the Effects of SSE (H')

The calculated lateral soil pressures including the effects of SSE are presented in Figures 3H.7-5 through 3H.7-8.

3H.7.4.3.4 Load Combinations

3H.7.4.3.4.1 Notations

U = Required strength for strength design method

D = Dead load

F' = Hydrostatic and hydrodynamic load due to flood

L = Live load

H = Lateral soil pressure and groundwater effects

H' = Lateral soil pressure and groundwater effects, including dynamic effects

W = Wind load

W_t = Total tornado load, including missile effects

E' = SSE seismic load

FL = Extreme environmental flood

3H.7.4.3.4.2 Reinforced Concrete Load Combinations

U = 1.4D + 1.7L + 1.7H

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

$U = D + L + H + FL$

$U = D + L + H + W_t$

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

$U = 1.05D + 1.3L + 1.3H$

$U = 1.05D + 1.3L + 1.3H + 1.3 W$

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

3H.7.4.4 Materials

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

3H.7.4.4.1 Reinforced Concrete

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

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

3H.7.4.4.2 Reinforcement

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

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

3H.7.4.4.3 Structural Steel

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

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

3H.7.4.5 Stability Requirements

The following minimum factors of safety are required against overturning, sliding, and flotation:

Load Combination	Overturning	Sliding	Flotation
$D + F_b$	-	-	1.1
$D + H + W$	1.5	1.5	-
$D + H + W_t$	1.1	1.1	-
$D + H' + E'$	1.1	1.1	-

Loads D, H, H', W, W_t, and E' are defined in Subsection 3H.7.4.3.4.1. F_b is the buoyant force corresponding to the flood water level.

3H.7.5 Structural Analysis and Design Summary

3H.7.5.1 Analytical Model Analysis and Design

The DGFOTs are Seismic Category I structures. The structural analysis and design of the DGFOT is performed using a three-dimensional (3D) SAP 2000 finite element analysis (FEA) with shell elements representing the walls, slabs and mat. The foundation soil is represented by vertical and horizontal springs. The FEA finite element model (FEM) is shown in Figure 3H.7-1.

The DGFOT No. 1B, which is the shortest tunnel, running approximately 50 ft between the RB and the DGFOSV No. 1B, has a wall thickness of 2'-0" on both sides. The interior below grade dimensions of this tunnel are approximately 7 ft high by 3.5 ft wide. The other two longer DGFOTs (approximately 200 ft and 220 ft long) have a wall thickness of 2'-0" on one side and 2'-6" on the other side to allow for placement of embedded conduits. The interior below grade dimensions of these tunnels are approximately 7 ft high by 3 ft wide. The DGFOT No. 1B, with a wall thickness of 2'-0" on both sides and shorter tunnel length for resisting torsion effects, is selected as the critical tunnel for the FEA.

The Safe Shutdown Earthquake (SSE) design forces (E') are conservatively determined using equivalent static seismic loads. The mass of the structure, equipment weights, and seismic live loads are excited in the X, Y, and Z directions using the enveloping maximum nodal accelerations in the X, Y, and Z directions from the soil-structure interaction (SSI) analysis. A comparison between the maximum accelerations from the SSI analysis and the design accelerations for the DGFOT shows the design accelerations envelope the SSI analysis accelerations. The resulting element forces and moments due to X, Y, and Z excitations are combined using the SRSS method.

Figures 3H.7-5 through 3H.7-8 show a comparison of the SSI soil pressures, the SSSI soil pressures, the ASCE 4-98 soil pressures and the total enveloping soil pressure used in design on the walls of the DGFOT.

The forces at tunnel bends due to SSE wave propagation are determined per Section 3H.7.5.2.4 and are included as additional loads in the SAP2000 models.

Multiple SAP2000 FEA models were created to represent different conditions and load combinations for the DGFOTs. The following is a breakdown of the different FEA models:

1. Normal (Operating Condition, Heavy Load Condition, and Flood Load Condition):

   The purpose of these models is to consider the effects of operating load conditions (i.e. dead loads, minimum live loads, etc.), the heavy load condition (when heavy vehicles and cargo are moved across the top of the tunnel), and the flood load condition (the extreme flood loads due to a MCR breach).

2. SSE (SSE loads without SSE Wave Propagation):

   The purpose of these models is to consider the effects of SSE loads without the effects of the SSE wave propagation, which are considered in a separate model. The dead loads, live loads, soil loads, and accidental eccentricity loads are applied to the static (non-seismic) model. The SSE loads are combined using the SRSS method in the dynamic (seismic) model.

3. SSE (SSE loads with SSE Wave Propagation per ASCE 4-98):

   The purpose of these models is to consider the effects of SSE loads with the effects of the SSE wave propagation and additional forces and moments due to bends in the tunnel per ASCE 4-98. The dead loads, live loads, soil loads, accidental eccentricity loads, SSE wave propagation loads and additional forces and moments due to bends in the tunnel are applied to the static (non-seismic) model. The SSE loads are combined using the SRSS method in the dynamic (seismic) model.

4. Tornado Missile:

   The purpose of these models is to consider the effects of vertical tornado missiles. The full tornado load combinations, outlined in Section 3H.7.4.3.4.2, are applied to the model considering a vertical tornado missile. The results of this SAP2000 model are combined with those from a manual calculation which considers the full tornado load combination and a horizontal tornado missile.

5. Effect of Uplift:

   The purpose of this model is to consider the effects of uplift on the basemat during a seismic event. All loads are simultaneously applied to a single static model.

The models described above are developed to determine the reinforcement required for their specific loading conditions. The results are post-processed as described in Section 3H.7.5.3.1.

The required reinforcement (longitudinal, in-plane shear and transverse) reported in Table 3H.7-1 is based on the envelop of the required reinforcement determined from all the SAP2000 FEA analyses and the required reinforcement determined via the manual calculation for the full tornado load combination.

3H.7.5.2 Analysis

3H.7.5.2.1 Seismic Analysis

The DGFOTs are long reinforced concrete tunnels with above grade access regions at the two ends of each tunnel. The widened envelop spectra of the resulting in-structure response spectra from the following two seismic analyses are used as the final in-structure response spectra for these tunnels and their access regions.

- Two-dimensional (2D) soil-structure-interaction (SSI) analysis of a typical cross section of the DGFOT
- Three-dimensional (3D) fixed base seismic analysis of the DGFOT No. 1B (approximately 50 ft long) including its access regions at the two ends of the tunnel.

The details of the above two seismic analyses are provided below.

A. 2D SSI Analysis of a Typical Cross section of DGFOT

SASSI2000 computer code is used for the SSI analysis, using the direct method. Figure 3H.7-20 shows the structural part of the 2D plane-strain model of the DGFOT with 2 ft thick mud mat under the base mat. The top of the tunnel is at the grade elevation. The specifics of the 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 out-of-plane direction) of the tunnel.
- Layered soil is modeled up to 74 ft depth (more than two times the horizontal cross section dimension of the tunnel plus its embedment depth) with halfspace below it.
- Sixteen cases of strain dependent soil properties representing the in-situ lower bound, mean and upper bound; lower bound backfill over in-situ lower bound, mean backfill over in-situ mean and upper bound backfill over in-situ upper bound; cracked concrete wall with in-situ upper bound soil, soil separation with in-situ upper bound soil; ABWR DCD/Tier 2 generic soil profiles UB1D, VP3D, VP4D, VP5D, VP7D, R , R with soil separation and R with cracked wall.

- Concrete and mud mat damping are assigned 4% for all cases (conservatively 4% damping is also used for cracked concrete cases).

- Groundwater is considered at 8 ft depth for site-specific soil and backfill cases and 2 ft depth for DCD cases. In site-specific and backfill cases, the groundwater effect is included by using minimum P-wave velocity of 5000 ft/sec with Poisson's ratio caped at 0.495. In DCD cases, the groundwater effect is included by using minimum P-wave velocity of 4800 ft/sec with Poisson's ratio caped at 0.495 (per Section 3A.3.3 of DCD, the compression wave velocity of water is 1463 m/sec, i.e. 4800 ft/sec).

- The models are capable of passing frequencies up to at least 33 Hz, in both the vertical and horizontal directions.

- For all SSI cases analyzed, a cut-off frequency of 35 Hz is used for transfer function calculations.

- Acceleration time histories consistent with Regulatory Guide 1.60 response spectra anchored at 0.3g peak ground acceleration are used as input at the grade elevation.

  The foundation input response spectra (FIRS) for the DGFOT were calculated and were compared to the outcrop spectra at the foundation level of the DGFOT. The outcrop spectra were calculated from a deconvolution analysis performed in the SHAKE program with the site-specific SSE motion applied at the free field ground surface. Figures 3H.7-22 through 3H.7-30 show the comparison of the outcrop response spectra and the FIRS, in the two horizontal directions and the vertical direction for the lower bound, mean and upper bound in-situ soil properties. These figures show that the FIRS are enveloped by the foundation outcrop spectra in all cases. The figures also show that the response spectra at the SHAKE outcrop of DGFOT foundation level also envelop a broad band spectrum anchored at 0.1g. This is the minimum requirement as stated in SRP 3.7.1 and Appendix S to 10 CFR 50. The broadband spectrum used in this comparison is conservatively defined as the Regulatory Guide 1.60 spectrum anchored at 0.1g.

- Since the tunnels run along both East-West and North-South directions, the horizontal input motions from both East-West and North-South time histories are considered. East-West input motion is applied to the tunnel sections running North-South and North-South input motion is applied to the tunnel sections running East-West. The input motions consistent with RG 1.60 response spectra anchored at 0.3g peak ground acceleration envelop both the site-specific input motions and the amplified site-specific motions considering the impact of nearby heavy RB and Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) Pump House.

- In-structure response spectra are generated at the top of floor slab (middle of span), at the top of the roof slab (middle of span) and at the mid-height of two walls of the tunnel cross-section.

- The responses from the horizontal and vertical directions are combined using the square-root-of-sum-of-square (SRSS) method.

- The responses from all SSI analyses cases are enveloped.

- The in-structure response spectra at the top of the floor slab (middle of span), at the roof of slab (middle of span) and at the mid-height of two walls of the tunnel cross-section are enveloped to conservatively provide the in-structure response spectra for the entire 2D cross-section of the tunnel.

B. 3D Fixed Base Analysis of DGFOT No. 1B Including its Two Access Regions

A 3D fixed base seismic (basemat fixed) analysis of the DGFOT No. 1B running between the RB and DGFOSV No. 1B is performed. The following provides the details of this fixed base analysis:

- SAP2000 computer code is used to perform the seismic analysis.

- Modal time history method of analysis is used.

- Shell elements are used for modeling the reinforced concrete tunnel section and the access regions at the two end of the tunnel.

- 4% damping is used for the shell elements.

- Acceleration time histories (two horizontal directions and a vertical direction) consistent with Regulatory Guide 1.60 response spectra anchored at 0.3g peak ground acceleration are used as input motions.

- Nodal acceleration time history responses obtained from the SAP2000 analysis are processed using the RSG computer code to calculate in-structure response spectra at selected nodes. The nodes selected for the in-structure response spectra generation are; four nodes on top of each access regions (middle of four walls) and three nodes at the top of tunnel (middle of the tunnel).

- The maximum co-directional responses from each of the three directions of excitations are combined using the SRSS method.

- The in-structure response spectra at the selected nodes are enveloped to conservatively provide the in-structure response spectra from fixed base analysis, for the entire tunnel and the access regions.

The corresponding in-structure response spectra obtained from the 2D SSI analysis and in-structure response spectra obtained from the 3D fixed base analysis described in parts A and B above are enveloped and peak widened by ± 30%. The 30% peak widening is used to cover any frequency shift due to the foundation soil flexibility, which is not included in the fixed base seismic analysis. The final widened in-structure response spectra for the horizontal and vertical directions of the DGFOTs and their access regions are provided in Figures 3H.7-31 and 3H.7-32, respectively. The spectra in Figures 3H.7-31 and 3H.7-32 provide the in-structure response spectra for the entire SGFOTs and their access towers at the two ends.

3H.7.5.2.2 Structure-Soil-Structure Interaction (SSSI) Analysis for Seismic Soil Pressures

Two 2D section cuts are taken for site-specific SSSI analyses; one East-West section cut through DGFOT No. 1C, DGFOSV No. 1A and the Crane Foundation Retaining Wall (CFRW) and one East-West section cut through the RB, DGFOT No. 1A and the CFRW. These SSSI analyses are used to obtain seismic soil pressures on the walls of DGFOT considering the effect of nearby structures.

The SSSI model and analyses details for the section cut through DGFOT No. 1C, DGFOSV No. 1A and the CFRW are provided in Section 3H.6.7.

The structural part of SSSI model for the section cut through the RB, DGFOT No. 1A and the CFRW is shown in Figure 3H.7-21. The methodology for the SSSI model including strain dependent soil properties; soil cases analyzed; and method of analyses are same as those for the section cut through DGFOT No. 1C, DGFOSV No. 1A and the CFRW described in Section 3H.6.7. This SSSI model is capable of passing frequencies up to at least 33 Hz in both the vertical and horizontal directions and the analysis uses a cut-off frequency 33 Hz for calculation of transfer functions.

Figures 3H.7-5 through 3H.7-8 show a comparison of the SSI, SSSI, ASCE 4-98 seismic soil pressures and the enveloping seismic soil pressures used for the design of the DGFOT walls.

The design of the DGFOTs also accounts for the axial tensile strain and the seismic induced forces at the tunnel bends due to SSE wave propagation as described in section 3H.7.5.2.4.

3H.7.5.2.3 Torsional Effects

The accidental torsion is computed in accordance with ASCE 4-98 considering an additional eccentricity of +/- 5% of the maximum building dimension for both horizontal directions. The induced member forces due to this accidental torsion are obtained from static analysis of the structure and are added to the induced forces to other applicable loads whether the analysis predicts positive or negative results (ie: absolute sum).

3H.7.5.2.4 SSE Wave Propagation Effects

The axial strain on the DGFOT due to SSE wave propagation is determined based on the equations and commentary outlined in Section 3.5.2.1 of ASCE 4-98. The maximum curvature is computed based on Equation 3.5-3 in Section 3.5.2.1.3 of ASCE 4-98.

The forces at bends due to SSE wave propagation are determined based on Section 3.5.2.2 of ASCE 4-98.

3H.7.5.3 Structural Design

3H.7.5.3.1 Reinforced Concrete Elements

The strength design criteria defined in ACI 349, as supplemented by RG 1.142, was used to design the reinforced concrete elements making up the DGFOT. Concrete with a compressive strength of 4.0 ksi and reinforcing steel with a yield strength of 60 ksi are considered in the design. All loads and load combinations listed in Section 3H.7.4 are considered in the design.

The design forces and provided longitudinal and transverse reinforcement for the DGFOT and access region walls and slabs are shown in Table 3H.7-1.

The shell forces from every element for every load combination in the finite element analysis were evaluated to determine the required reinforcement. The following out-of-plane moment and axial force coupled with the corresponding load combination are reported in Table 3H.7-1 when the governing forces, moments and reinforcement is from the SAP2000 models:

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

For each surface, the following in-plane and transverse shears with the corresponding load combination are reported in Table 3H.7-1 when the governing forces, moments and reinforcement is from the SAP2000 models:

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

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

The provided transverse shear reinforcing (as required) is determined based on the transverse shears and axial forces perpendicular to the shear plane occurring simultaneously for every load combination.

3H.7.5.3.2 Foundation Design

The foundation for the DGFOT consists of a reinforced concrete mat and a lean concrete mud mat. The basemat deflections due to the flexibility of the basemat and supporting soil were accounted for through the use of foundation soil springs in the SAP2000 finite element analysis models. Both the Winkler and the Pseudo-Coupled Methods were used to model the foundation soil springs. The results of the two analyses were enveloped for design purposes.

Two different subgrade reactions (soil spring constants) are used, one for seismic loads and one for non-seismic loads. The following soil spring constants were used in the FEA models of the DGFOTs:

Vertical springs (with static loads)..260 kips/ft/ft^2

Vertical springs (with seismic loads)..531 kips/ft/ft^2

North-south springs (with static and seismic loads)...............................318 kips/ft/ft^2

East-west springs (with static and seismic loads)..................................318 kips/ft/ft^2

3H.7.5.3.3 Uplift Analysis

The effect of uplift on the basemat during a seismic event was considered through the use of a SAP2000 design model which simulated the uplift condition. The seismic design accelerations applied to the SAP2000 design uplift model are adjusted by a scale factor which scales the seismic forces to the maximum level possible during an uplift condition of the DGFOT. The scaled seismic accelerations along with applicable loads described in Section 3H.7.4 are then combined. The results of the uplift model and the design models were enveloped for design purposes.

3H.7.5.3.4 Stability Evaluation

The DGFOT stability evaluations are performed for the various load combination listed in Section 3H.7.4.5. The DGFOT factors of safety against sliding, overturning, and flotation are provided in Table 3H.7-2. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for combination of the X, Y, and Z seismic excitations.

Restraints are provided around the Access Regions to limit movement and rotation due to a tornado missile.

Table 3H.7-1: Results of DGFOT Concrete Wall and Slab Design

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1,8)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads					Longitudinal Reinforcement Provided (in²/ft)	Transverse Shear Design Loads		Transverse Shear (7) Reinforcement Provided (in²/ft²)	Remarks
								Axial and Flexure Loads			In-Plane Shear Loads						
								Loads (11) Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	Loads (11) Combination	In-plane (5) Shear (kips / ft)		Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)		
Tunnel Walls	2	Near Side	Horizontal	3H 7-11	1-H-L	Max Tension w/ corresponding moment	951	D + L + H' +E' (WP)	130	-28	D + L + H' +E' (WP)	26	4.68	-	-	-	
						Max Compression w/ corresponding moment	932	D + L + H' +E' (WP)	-66	-1							
						Max Moment with axial tension	952	D + L + H' +E' (WP)	48	-32							
						Max Moment with axial compression	953	D + L + H' +E' (WP)	-1	-28							
					2-H-L	Max Tension w/ corresponding moment	153	D + L + H' +E' (WP)	89	-11	D + L + H' +E' (WP)	21	3.12	-	-	-	
						Max Compression w/ corresponding moment	854	D + L + H' +E' (WP)	-77	-1							
						Max Moment with axial tension	265	D + L + H' +E' (WP)	62	-17							
						Max Moment with axial compression	706	D + L + H' +E' (WP)	-8	-16							
					3-H-L	Max Tension w/ corresponding moment	149	D + L + H' +E' (WP)	108	-28	D + L + H' +E' (WP)	26	4.68	-	-	-	
						Max Compression w/ corresponding moment	149	D + L + H' +E' (WP)	-123	-6							
						Max Moment with axial tension	149	D + L + H' +E' (WP)	104	-28							
						Max Moment with axial compression	141	D + L + H' +E' (WP)	-9	-28							
		Far Side	Horizontal	3H 7-12	1-H-L	Max Tension w/ corresponding moment	284	D + L + H +Wt	109	0	D + L + H' +E' (WP)	26	3.12	-	-	-	
						Max Compression w/ corresponding moment	149	D + L + H' +E' (WP)	-129	25							
						Max Moment with axial tension	634	D + L + H' +E' (WP)	4	28							
						Max Moment with axial compression	277	D + L + H' +E' (WP)	-72	30							
		Near Side	Vertical	3H 7-13	1-V-L	Max Tension w/ corresponding moment	953	D + L + H' +E'	35	-6	D + L + H + Wt	59	3.12	-	-	-	
						Max Compression w/ corresponding moment	918	D + L + H +Wt	-96	-16							
						Max Moment with axial tension	902	D + L + H' +E' (WP)	14	-86							
						Max Moment with axial compression	902	D + L + H' +E' (WP)	-10	-86							
		Far Side	Vertical	3H 7-14	1-V-L	-	-	D + L + H +Wt	-	17	D + L + H + Wt	59	3.12	-	-	-	

Table 3H.7-1: Results of DGFOT Concrete Wall and Slab Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1,8)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads: Axial and Flexure Loads: Loads (11) Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	In-Plane Shear Loads: Loads (11) Combination	In-plane (5) Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads: Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)	Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
Access Region Walls	2	Near Side	Horizontal	3H.7-9	1-H-L	-	-	D + L + H + Wt	See Note (9)		D + L + H' +E' (WP)	34	3.12	-	-	-	
		Far Side	Horizontal	3H.7-9	1-H-L	-	-	D + L + H + Wt			D + L + H' +E' (WP)	34	3.12	-	-	-	
		Near Side	Vertical	3H.7-10	1-V-L	-	-	D + L + H + Wt			D + L + H + Wt	182	3.12	-	-	-	
		Far Side	Vertical	3H.7-10	1-V-L	-	-	D + L + H + Wt			D + L + H + Wt	182	3.12	-	-	-	

Table 3H.7-1: Results of DGFOT Concrete Wall and Slab Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number (1,8)	Reinforcement Zone Number (2)	Maximum Forces (3)	Element	Longitudinal Reinforcement Design Loads					Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads		Transverse Shear (7) Reinforcement Provided (in^2/ft^2)	Remarks
								Axial and Flexure Loads			In-Plane Shear Loads						
								Loads (11) Combination	Axial (4) (kips / ft)	Flexure (4) (ft-kips / ft)	Loads (11) Combination	In-plane (5) Shear (kips / ft)		Load Combination	Transverse Shear (6) Reinforcement Design Loads (kips / ft)		
Basemat	2	Near Side	Horizontal	3H.7-15	1-H-L	-	-	D + L + H + Wt	See Note (10)		D + L + H' +E' (WP)	27	3.12	-	-	-	
		Far Side	Horizontal	3H.7-15	1-H-L	Max Tension w/ corresponding moment	2584	D + L + H' +E' (WP)	95	8	D + L + H' +E' (WP)	27	3.12	-	-	-	
						Max Compression w/ corresponding moment	309	D + L + H' +E' (WP)	-117	12							
						Max Moment with axial tension	2351	D + L + H' +E' (WP)	12	21							
						Max Moment with axial compression	2316	D + L + H +Wt	-13	32							
		Near Side	Vertical	3H.7-16	1-V-L	Max Tension w/ corresponding moment	2425	D + L + H' +E' (WP)	20	-60	D + L + H' +E' (WP)	47	3.12	-	-	-	
						Max Compression w/ corresponding moment	301	D + L + H' +E' (WP)	-23	0							
						Max Moment with axial tension	2433	D + L + H' +E' (WP)	16	-74							
						Max Moment with axial compression	2554	D + L + H' +E' (WP)	-2	-72							
		Far Side	Vertical	3H.7-16	1-V-L	Max Tension w/ corresponding moment	2315	D + L + H +Wt	13	2	D + L + H' +E' (WP)	47	3.12	-	-	-	
						Max Compression w/ corresponding moment	309	D + L + H' +E' (WP)	-35	79							
						Max Moment with axial tension	2438	D + L + H' +E' (WP)	1	56							
						Max Moment with axial compression	2496	D + L + H' +E' (WP)	-29	87							
				3H.7-17	1-H-T	-	-	-	-	-	-	-	-	D + L + H + Wt	47	0.31	

Table 3H.7-1: Results of DGFOT Concrete Wall and Slab Design (Continued)

Location	Thickness (ft)	Face	Direction	Reinforcement Layout Drawing Number [1,8]	Reinforcement Zone Number [2]	Maximum Forces [3]	Element	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Loads [11] Combination	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Axial [4] (kips / ft)	Longitudinal Reinforcement Design Loads – Axial and Flexure Loads – Flexure [4] (ft-kips / ft)	Longitudinal Reinforcement Design Loads – In-Plane Shear Loads – Loads [11] Combination	Longitudinal Reinforcement Design Loads – In-Plane Shear Loads – In-plane [5] Shear (kips / ft)	Longitudinal Reinforcement Provided (in^2/ ft)	Transverse Shear Design Loads – Load Combination	Transverse Shear Design Loads – Transverse Shear [6] Reinforcement Design Loads (kips / ft)	Transverse Shear [7] Reinforcement Provided (in^2/ft^2)	Remarks
Roof of Tunnel	2	Near Side	Horizontal	3H.7-18	1-H-L	Max Tension w/ corresponding moment	174	D + L + H + Wt	124	-11	D + L + H' +E' (WP)	34	3.12	-	-	-	
						Max Compression w/ corresponding moment	1703	D + L + H' +E' (WP)	-117	-4							
						Max Moment with axial tension	1688	D + L + H + Wt	53	-38							
						Max Moment with axial compression	1694	D + L + H' +E' (WP)	-9	-30							
		Far Side	Horizontal	3H.7-18	1-H-L	Max Tension w/ corresponding moment	1710	D + L + H' +E' (WP)	118	7	D + L + H' +E' (WP)	34	3.12	-	-	-	
						Max Compression w/ corresponding moment	1703	D + L + H' +E' (WP)	-117	12							
						Max Moment with corresponding axial tension	1695	D + L + H + Wt	10	41							
						Max Moment with corresponding axial compression	1840	D + L + H + Wt	-10	53							
		Near Side	Vertical	3H.7-19	1-V-L	Max Tension w/ corresponding moment	1694	D + L + H + Wt	17	-20	D + L + H' +E' (WP)	53	3.12	-	-	-	
						Max Compression w/ corresponding moment	1694	D + L + H' +E' (WP)	-28	-45							
						Max Moment with corresponding axial tension	1710	D + L + H' +E' (WP)	0	-65							
						Max Moment with corresponding axial compression	174	D + L + H' +E' (WP)	-14	-70							
		Far Side	Vertical	3H.7-19	1-V-L	Max Tension w/ corresponding moment	1694	D + L + H + Wt	16	9	D + L + H' +E' (WP)	53	3.12	-	-	-	
						Max Compression w/ corresponding moment	1839	D + L + H' +E' (WP)	-23	6							
						Max Moment with corresponding axial tension	209	D + L + H' +E' (WP)	2	54							
						Max Moment with corresponding axial compression	209	D + L + H' +E' (WP)	-5	54							

Table 3H.7-1: Results of DGFOT Concrete Wall and Slab Design (Continued)

Notes:

(1) The reinforcement layout drawings show the various zones used to define the minimum reinforcement that will be provided based on finite element analysis results. Actual provided reinforcement based on final rebar layout and including development length may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported boundaries. The dimensions in the reinforcement drawings are based on the dimensions of the 2D SAP2000 shell elements, which are modeled at the centerline of the walls and slabs.

(2) Each reinforcement layout drawing is divided into reinforcement zones. The reinforcement zone naming convention is as follows: "H" = horizontal, "V" = vertical, "L" = longitudinal reinforcement, "T" = transverse reinforcement. For slabs, vertical corresponds to Y-axis and horizontal corresponds to X-axis as shown on Figure 3H.7-1.

(3) The maximum tension and compression axial forces are provided with the corresponding moment from the same load combination. The maximum moment that has a corresponding tension in the same load combination and the maximum moment that has a corresponding compression in the same load combination are also provided.

(4) Negative axial load is compression and positive axial load is tension. Negative moment applies tension to the top face of the shell element and positive moment applies tension to the bottom face of the shell element. For walls or slabs where the same reinforcement is provided on both faces, the moment is shown as absolute value.

(5) The reported in-plane shear is the maximum average in-plane shear along a plane that crosses the longitudinal reinforcement zone.

(6) The reported transverse shear is the maximum average transverse shear along a plane in that transverse reinforcement zone.

(7) In areas where horizontal and vertical transverse shear zones overlap, the total transverse shear reinforcement to be supplied in the overlapping area is the sum of the transverse reinforcement required from the horizontal and vertical zones.

(8) Openings in the Access Regions have not been included in the Reinforcement Layout Drawings.

(9) The Access Region is governed by the tornado load combination. The outside layer of transverse torsional reinforcement (all 4 near sides horizontal) in conjunction with the near side vertical longitudinal reinforcement are utilized to resist a torsional moment of 1438 kip*ft due to an eccentric tornado missile load. The far side horizontal reinforcement is utilized to resist an axial force of 805 kip due to a concentric tornado missile load as well as a tornado wind pressure of 294 psf. The remaining capacity of the near side vertical longitudinal reinforcement in conjunction with the far side vertical longitudinal reinforcement are utilized to resist a moment of 10076 kip*ft due to the tornado load combination.

(10) The basemat near side horizontal reinforcement is governed by the tornado load combination. The outside layer of transverse torsional reinforcement is composed of near side vertical reinforcement (tunnel walls in Z-direction and roof and basemat in Y-direction) in conjunction with the near side horizontal reinforcement (2 tunnel walls, roof, and basemat in X-direction) are utilized to resist a torsional moment of 8085 kip*ft due to tornado load combination.

(11) The "E ' (WP)" designation in the load combination column indicates seismic SSE loading including wave propagation effects.

Table 3H.7-2: Factors of Safety against Sliding, Overturning and Flotation for DGFOT

Load Combination	Calculated Safety Factor			Notes
	Overturning	Sliding	Flotation	
D + F_b	--	--	1.70	
D + H + W	1.58	3.47	--	2, 3 (Sliding Only)
D + H + Wt	1.10	1.10	--	2, 4
D + H' + E'	1.30	1.28	--	2, 3

Notes:

1) Loads D, H, H', W, Wt, and E' are defined in Section 3H.7.4.3.4. F_b is the buoyant force corresponding to the design basis flood.
2) Coefficients of friction for sliding resistance are 0.58 for static conditions and 0.39 for dynamic conditions for the Diesel Generator Fuel Oil Tunnel.
3) The calculated safety factors consider the full passive pressure.
4) The minimum calculated safety factor against sliding and overturning for tornado wind is 2.32. For tornado wind in conjunction with tornado missile, subsequent detailed design of the restraints for the Access Regions will provide sliding and overturning safety factors greater than 1.10.

Table 3H.7-3 Tornado Missile Impact Evaluation for Diesel Generator Fuel Oil Tunnel

Local Check	**DGFOT and Access Regions**	Minimum required thickness to prevent penetration, perforation, and scabbing = 15.14" Minimum provided thickness = 24"
Overall Check of Impacted Element	**Walls and Slabs of DGFOT and Access Regions**	Flexure controls. Maximum impact load including Dynamic Load Factor (DLF) = 899 kips for Access Regions and 862 kips for DGFOT Ductility demand = 1.4 for shell missile and 1.0 for automobile missile < Ductility limit = 10
Global Check		Equivalent static impact forces due to missile impact are considered in the local and global design of the DGFOT. The analysis results presented in Table 3H.7-1 provide a summary of the results for all load combinations including those affected by the tornado missile impact.

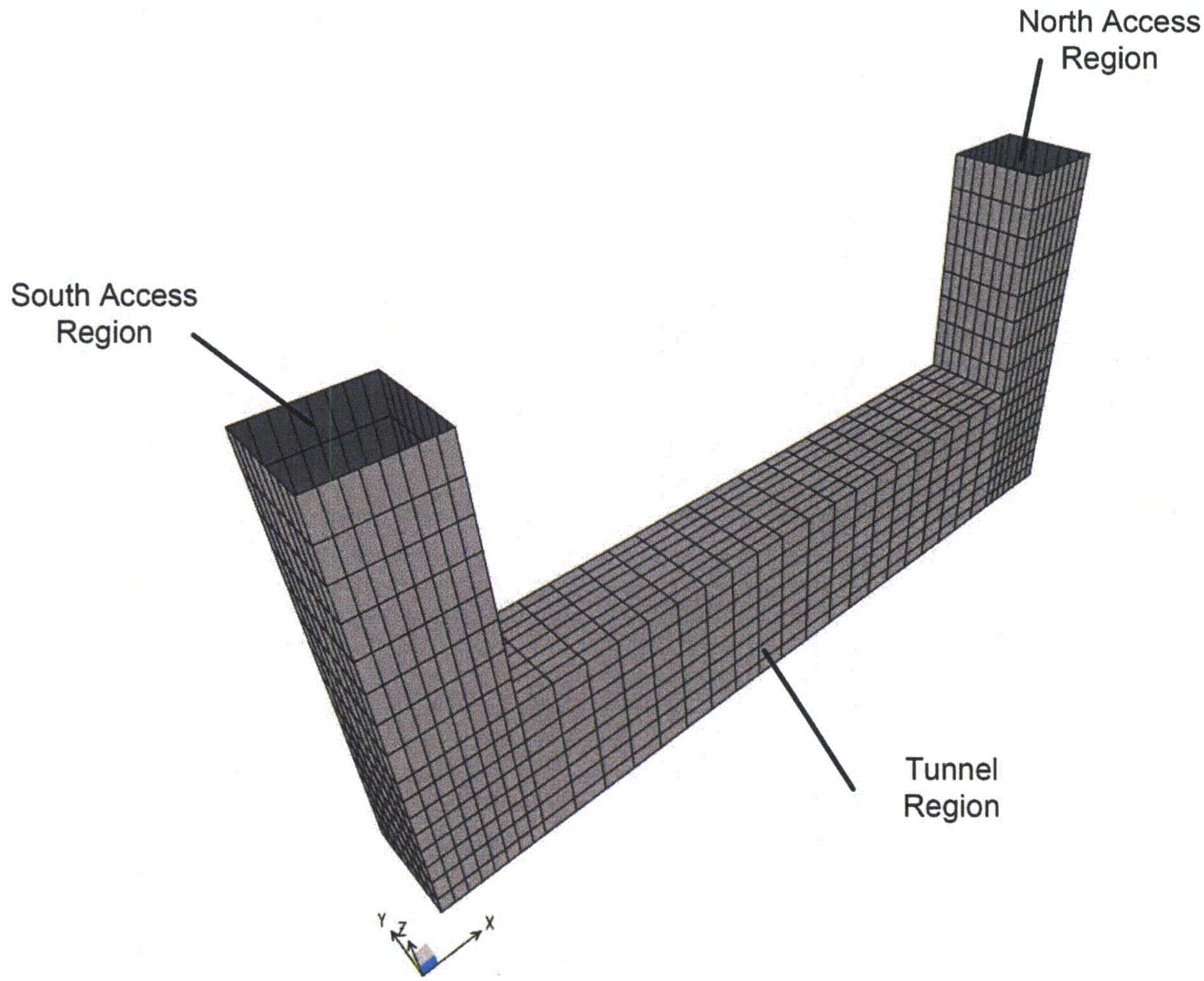


Figure 3H.7-1: SAP2000 Finite Element Analysis Model for DGFOT

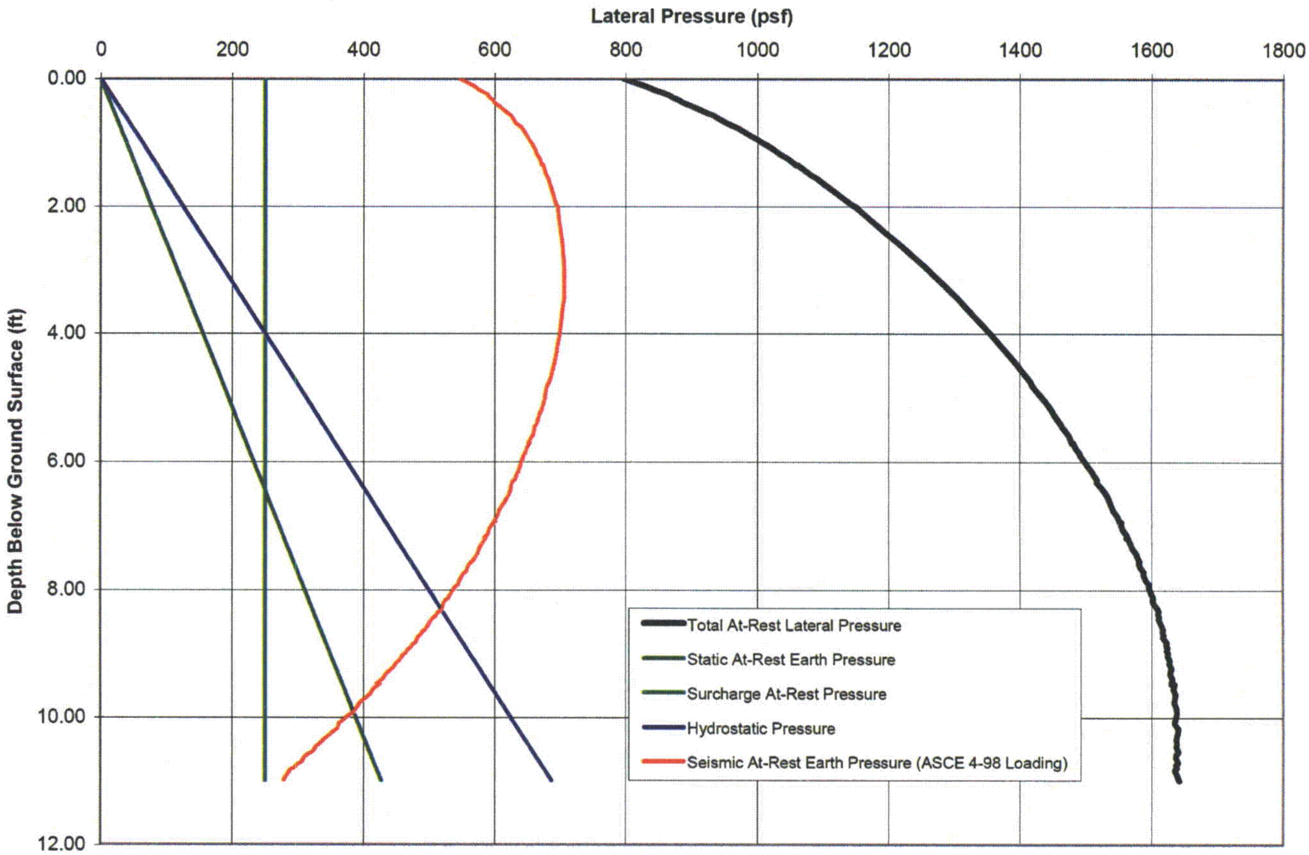


Figure 3H.7-2: At-Rest Lateral Earth Pressure (psf) on the Walls of the Fuel Oil Tunnel

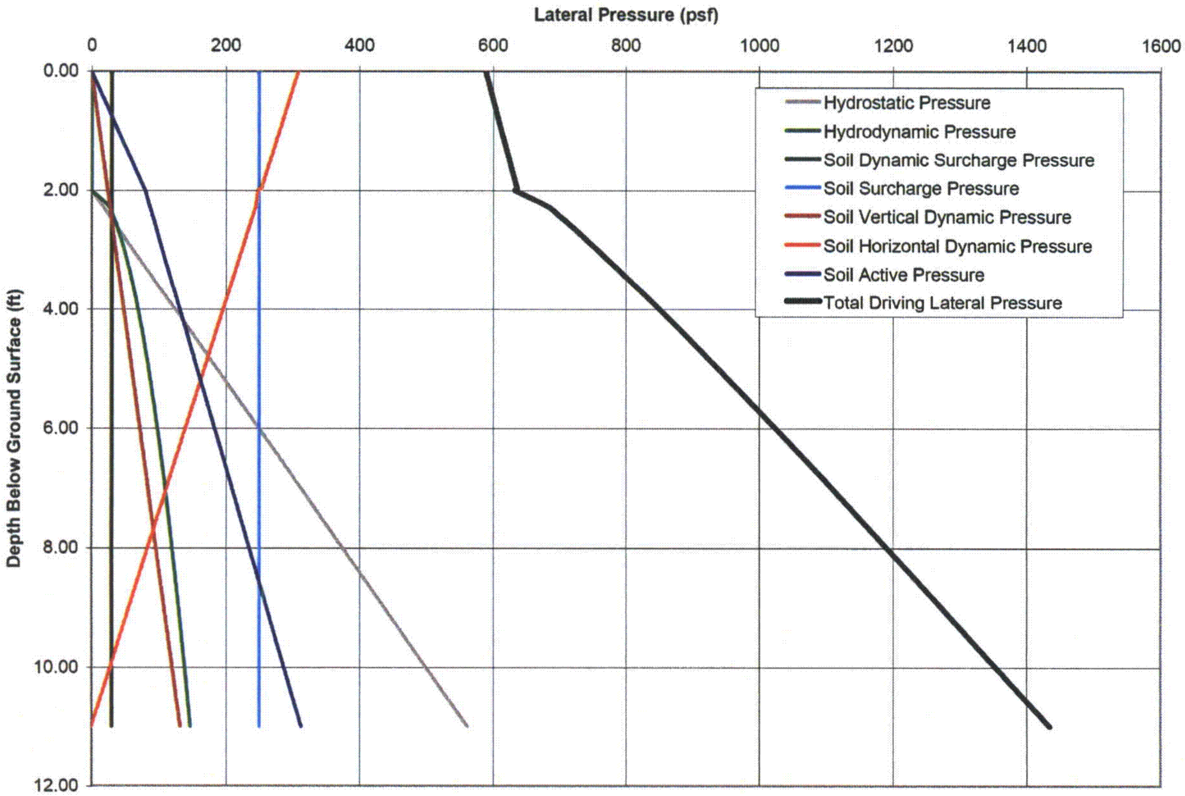


Figure 3H.7-3: Driving Lateral Earth Pressure (psf) on the Walls of the Fuel Oil Tunnel

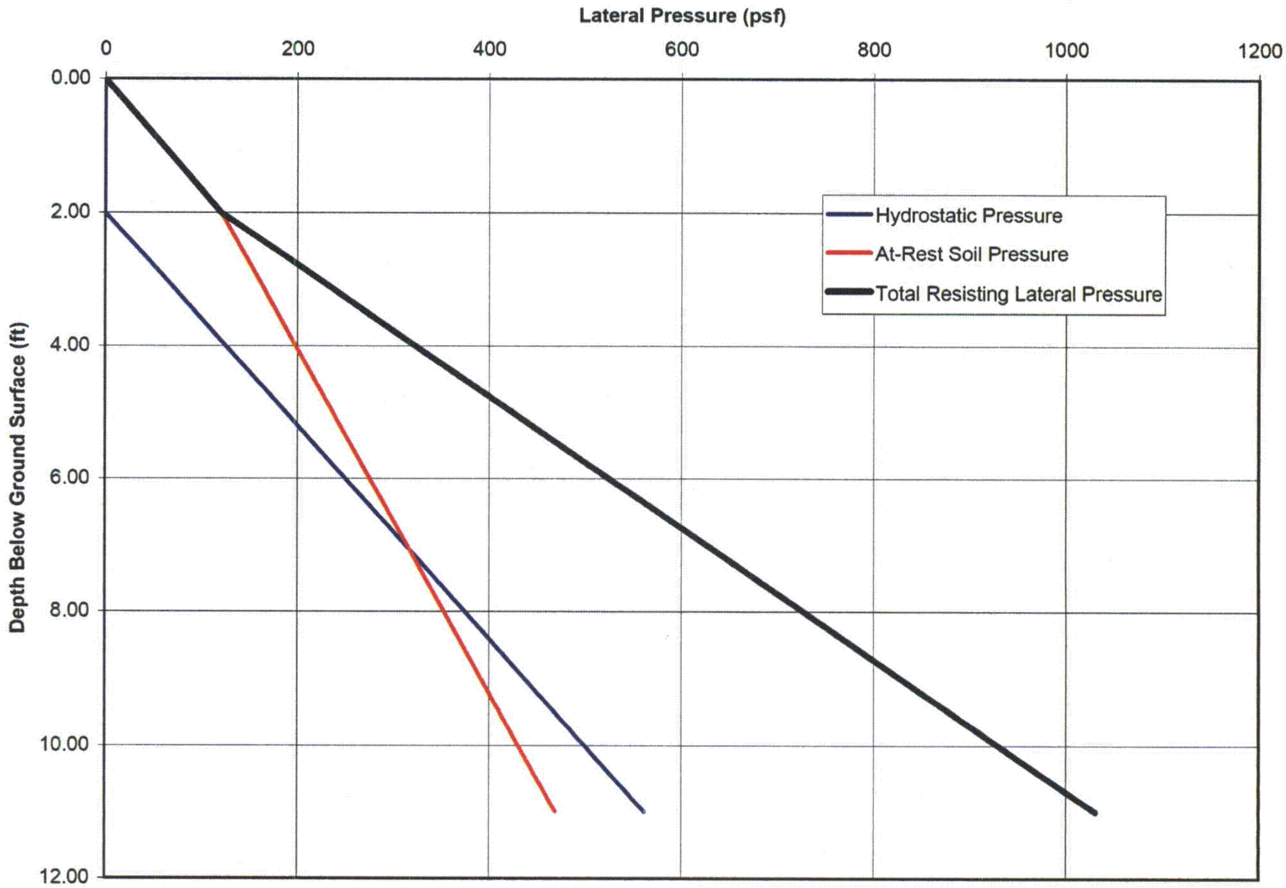


Figure 3H.7-4: Resisting Lateral Earth Pressure (psf) on the Walls of the Fuel Oil Tunnel

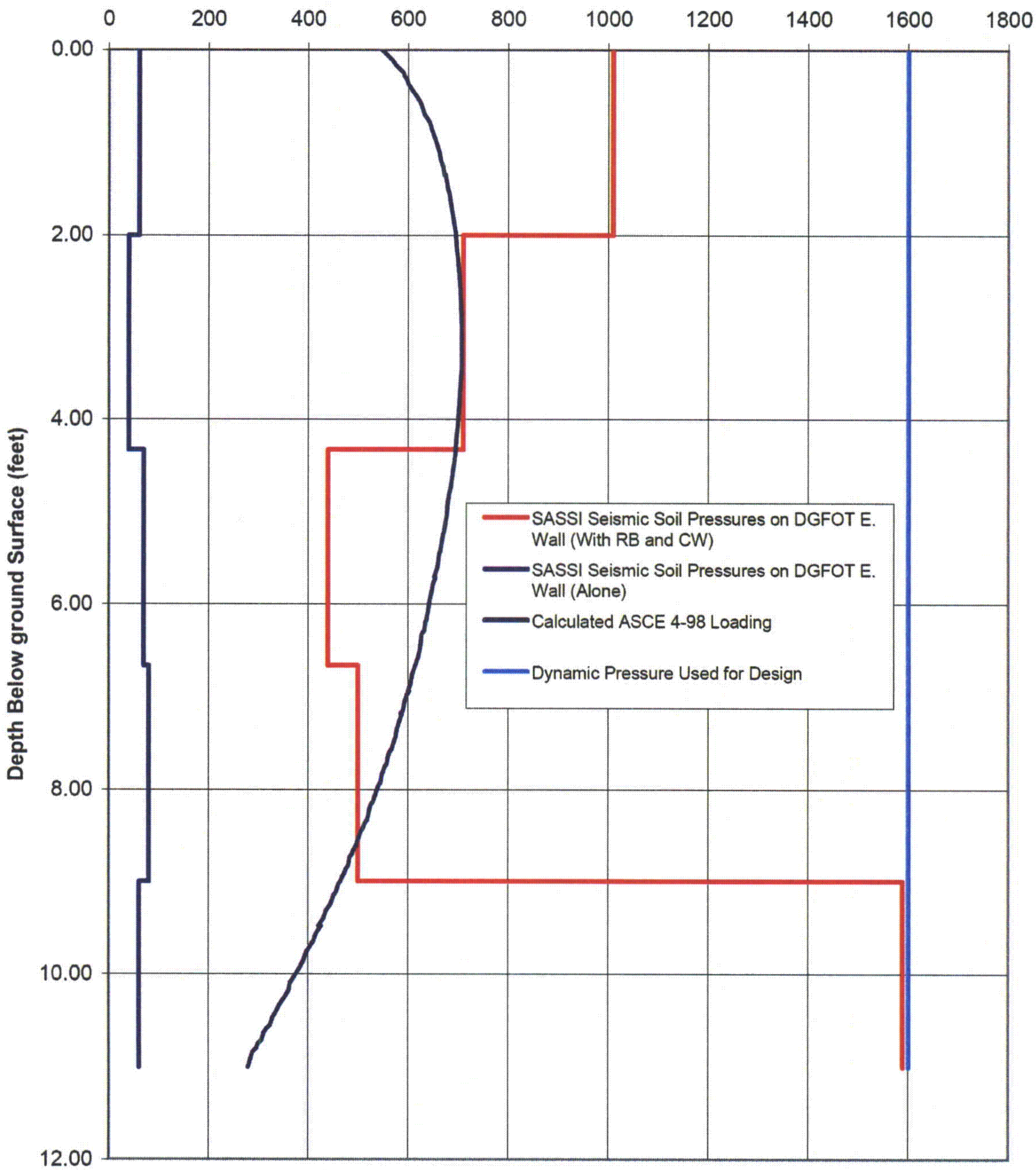


Figure 3H.7-5: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel East Wall with Reactor Building and Crane Wall

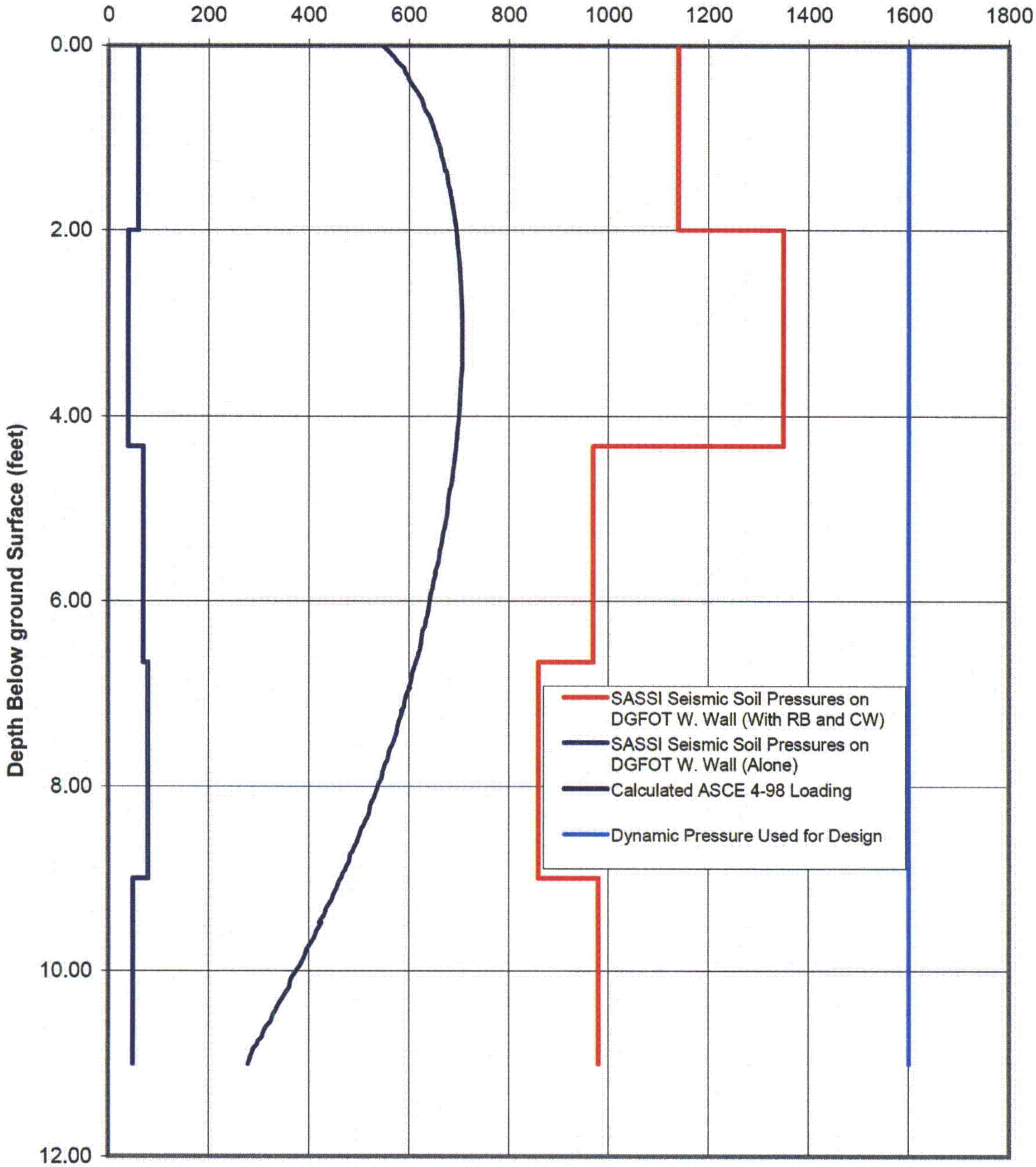


Figure 3H.7-6: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel West Wall with Reactor Building and Crane Wall

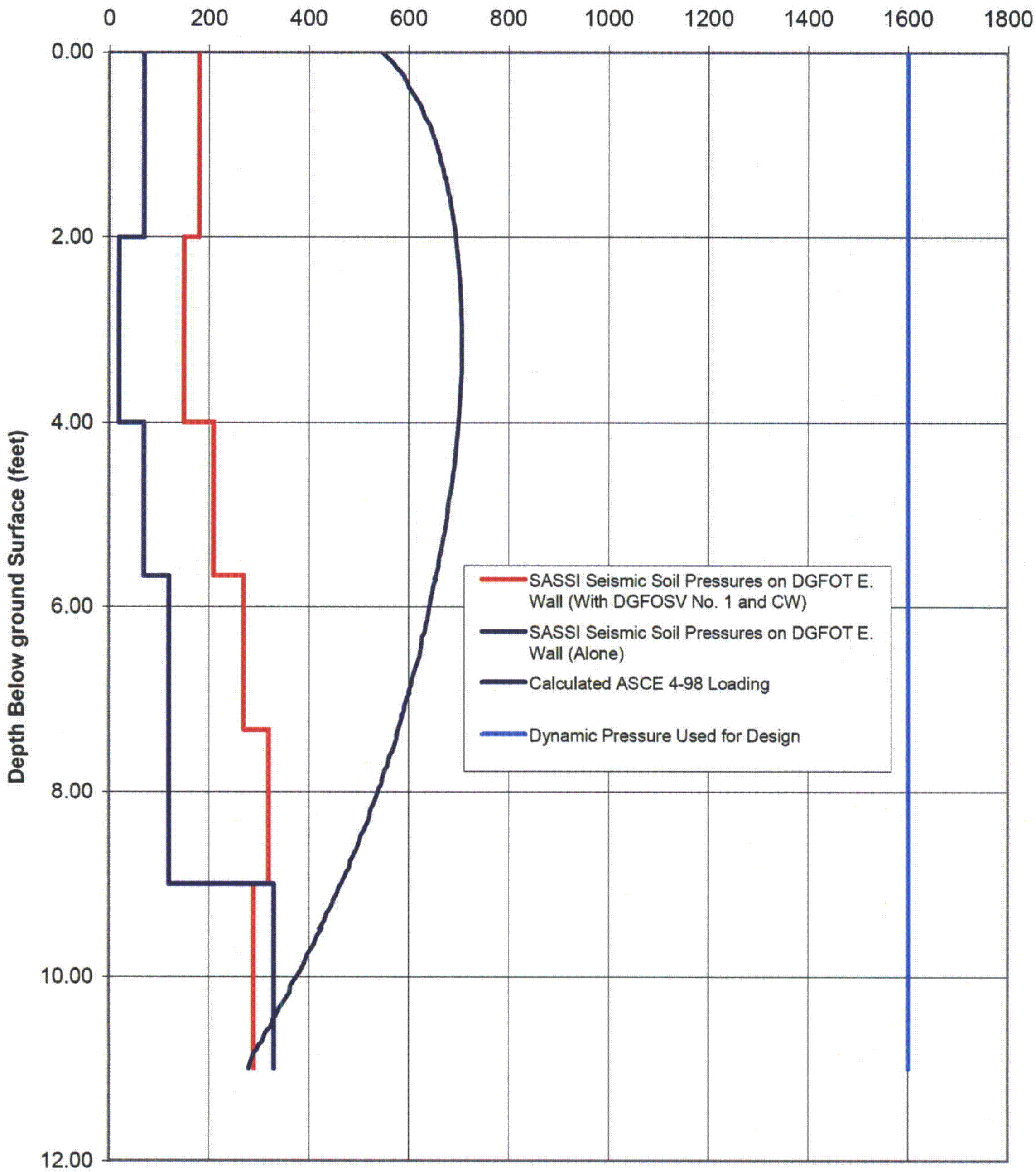


Figure 3H.7-7: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel East Wall with Diesel Generator Fuel Oil Storage Vault and Crane Wall

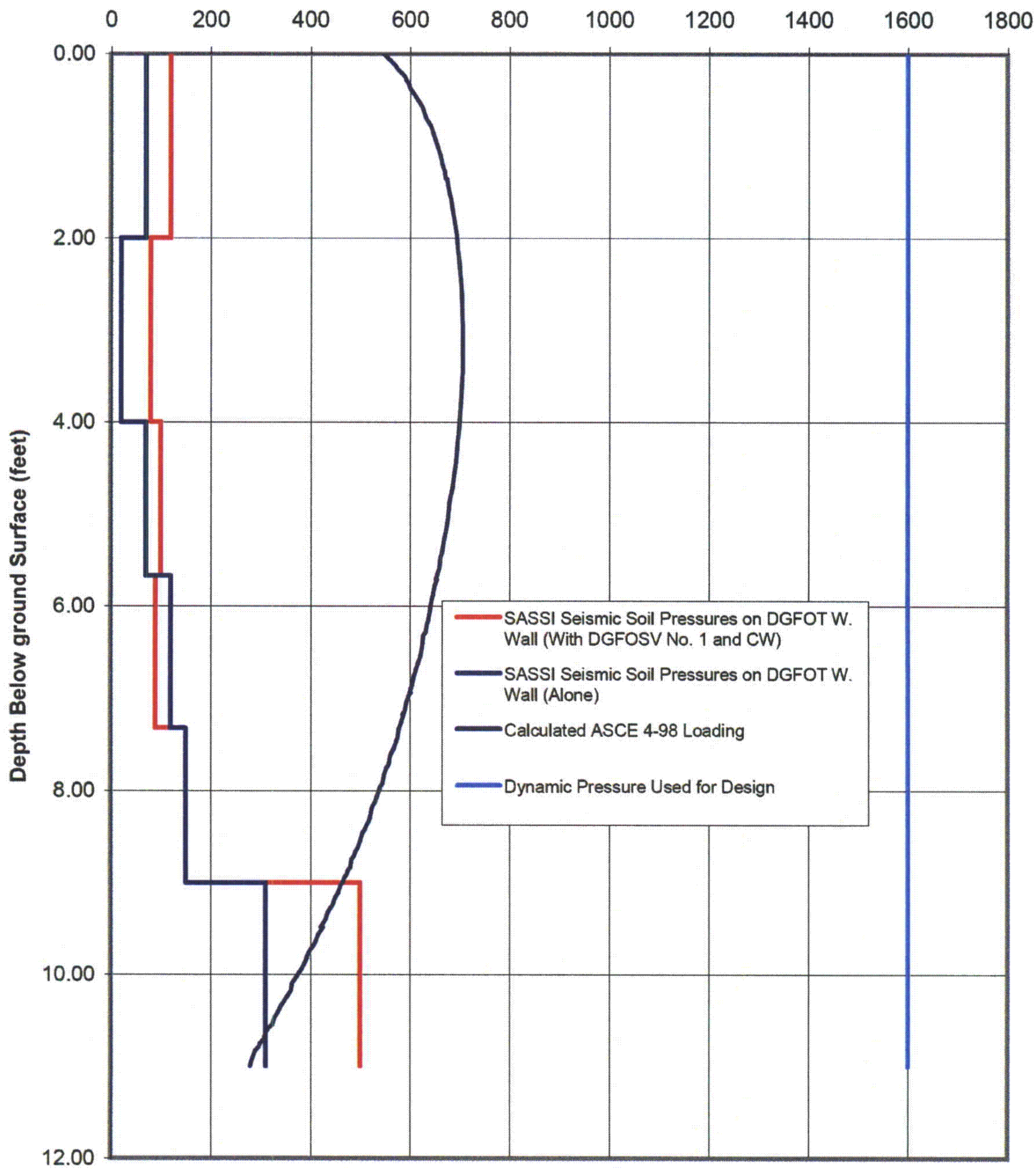


Figure 3H.7-8: SSI, SSSI, ASCE 4-98 and Design Lateral Seismic Soil Pressures (psf) on Fuel Oil Tunnel West Wall with Diesel Generator Fuel Oil Storage Vault and Crane Wall

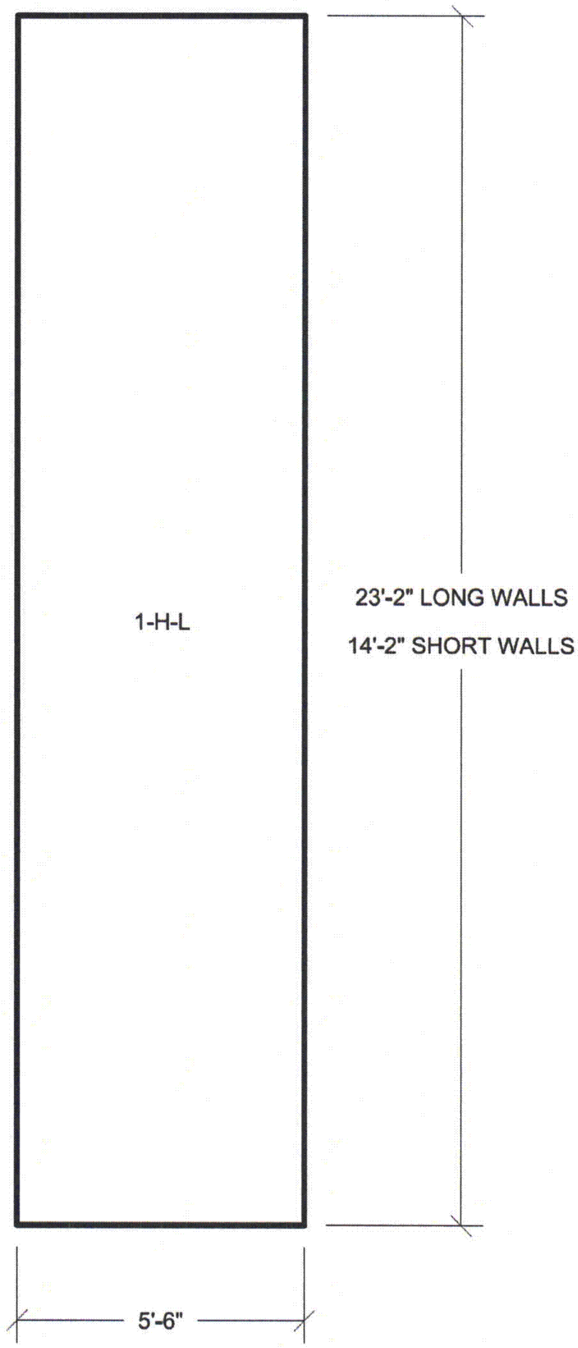


**Figure 3H.7-9: Access Region Walls Looking From Outside
Horizontal Reinforcement Zones
Near Side and Far Side Faces**

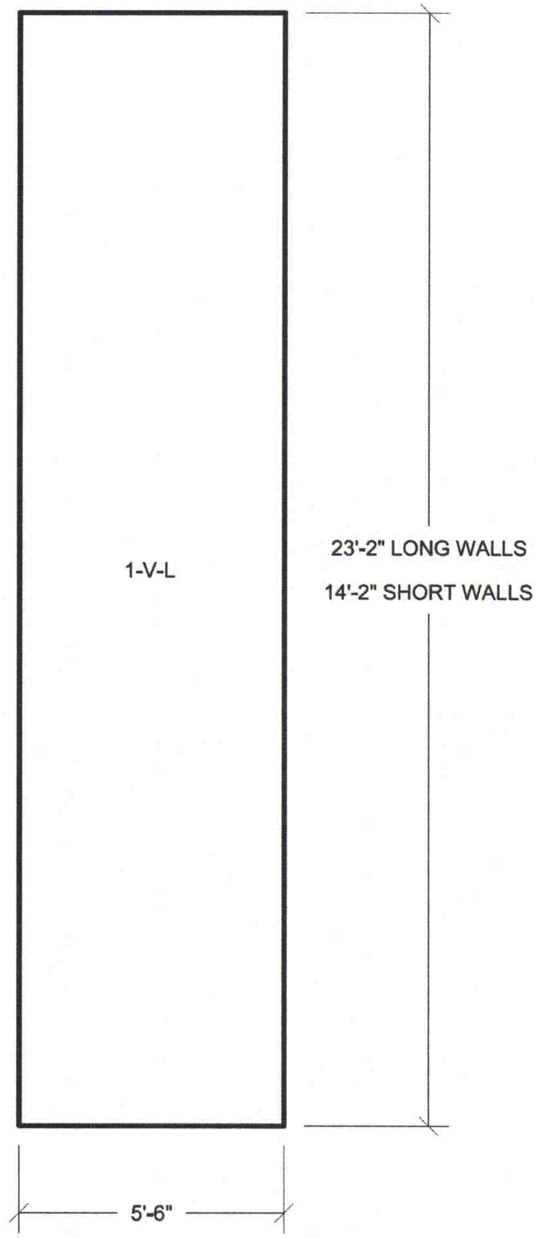


**Figure 3H.7-10: Access Region Walls Looking From Outside
Vertical Reinforcement Zones
Near Side and Far Side Faces**

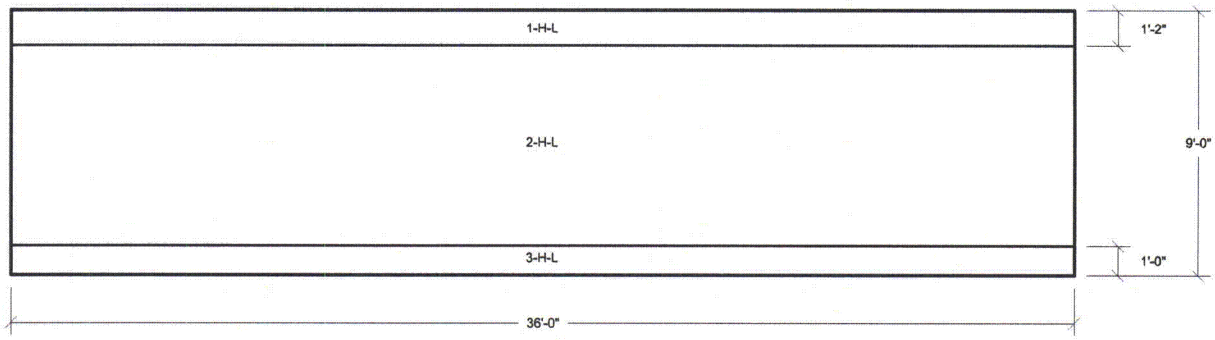


**Figure 3H.7-11: Tunnel Walls Looking From Outside
Horizontal Reinforcement Zones
Near Side Face**

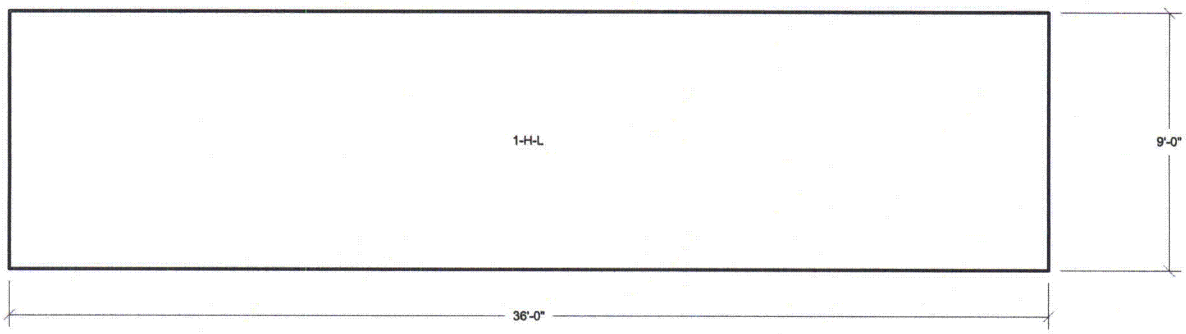


Figure 3H.7-12: Tunnel Walls Looking From Outside
Horizontal Reinforcement Zones
Far Side Face

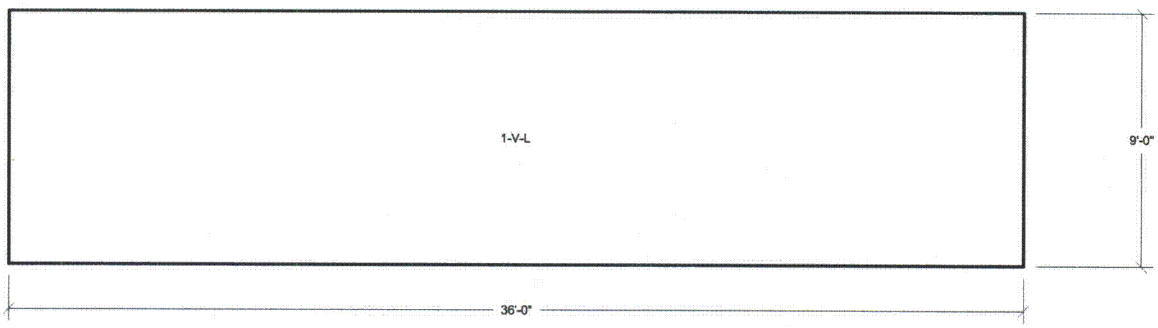


Figure 3H.7-13: Tunnel Walls Looking From Outside
Vertical Reinforcement Zones
Near Side Face

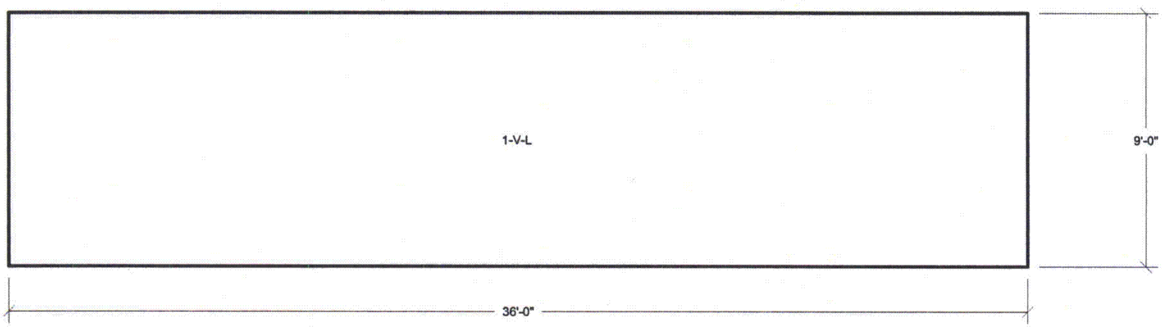


Figure 3H.7-14: Tunnel Walls Looking From Outside
Vertical Reinforcement Zones
Far Side Face

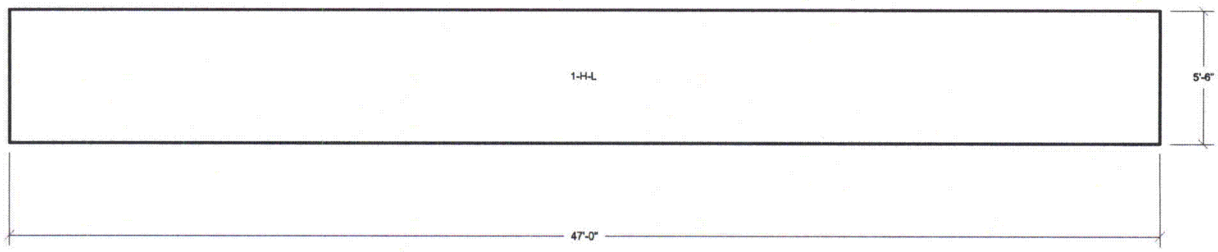


**Figure 3H.7-15: Tunnel and Access Region Basemat Looking Down
Horizontal Reinforcement Zones
Near Side and Far Side Faces**

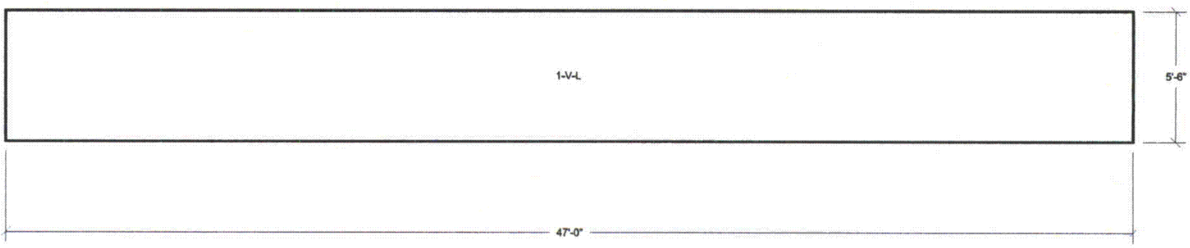


**Figure 3H.7-16: Tunnel and Access Region Basemat Looking Down
Vertical Reinforcement Zones
Near Side and Far Side Faces**

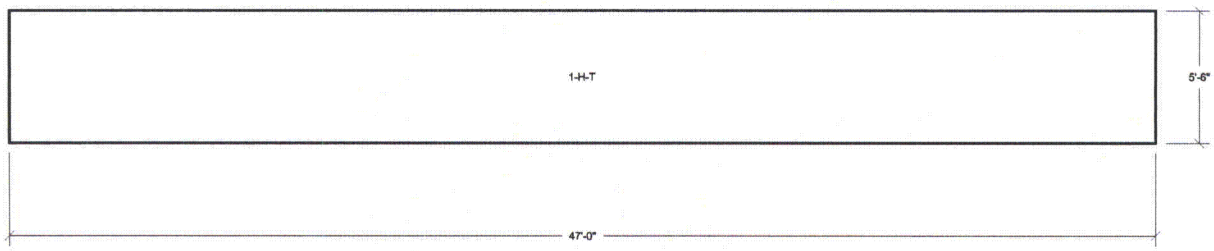


Figure 3H.7-17: Tunnel and Access Region Basemat Looking Down Transverse Reinforcement Zones

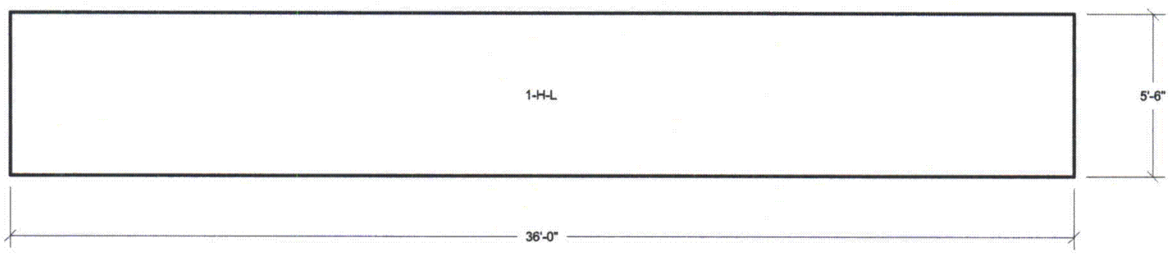


Figure 3H.7-18: Roof of Tunnel Looking Down
Horizontal Reinforcement Zones
Near Side and Far Side Faces

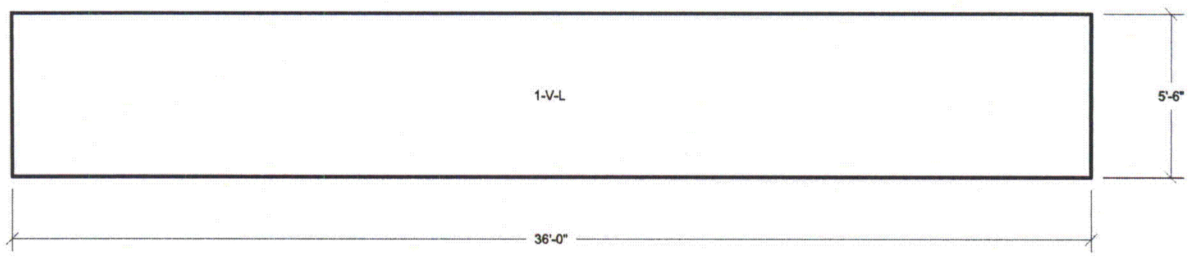


Figure 3H.7-19: Roof of Tunnel Looking Down
Vertical Reinforcement Zones
Near Side and Far Side Faces

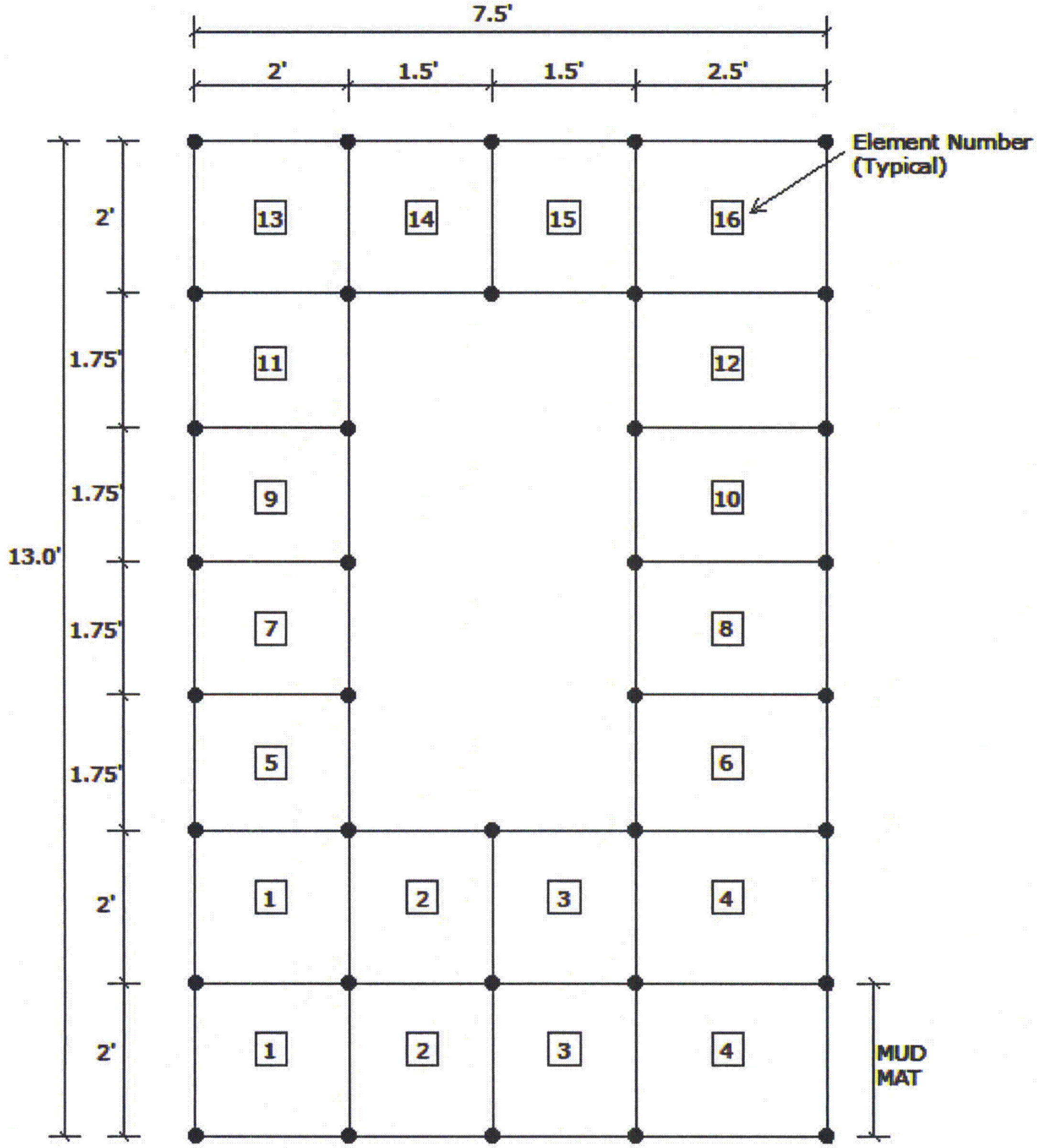


Figure 3H.7-20: 2D Model for SSI Analysis of a Typical Cross section of DGFOT

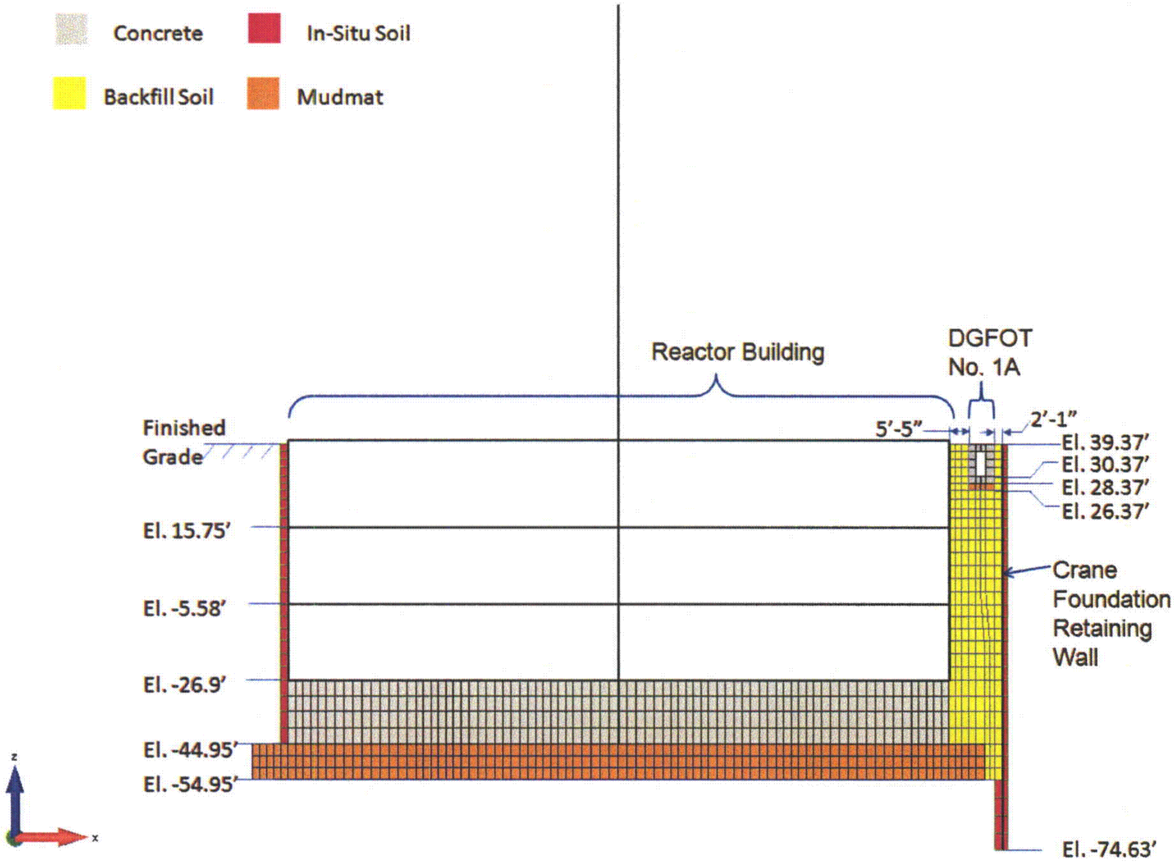


Figure 3H.7-21: 2D SSSI Model of RB, DGFOT and Crane Foundation Retaining Wall

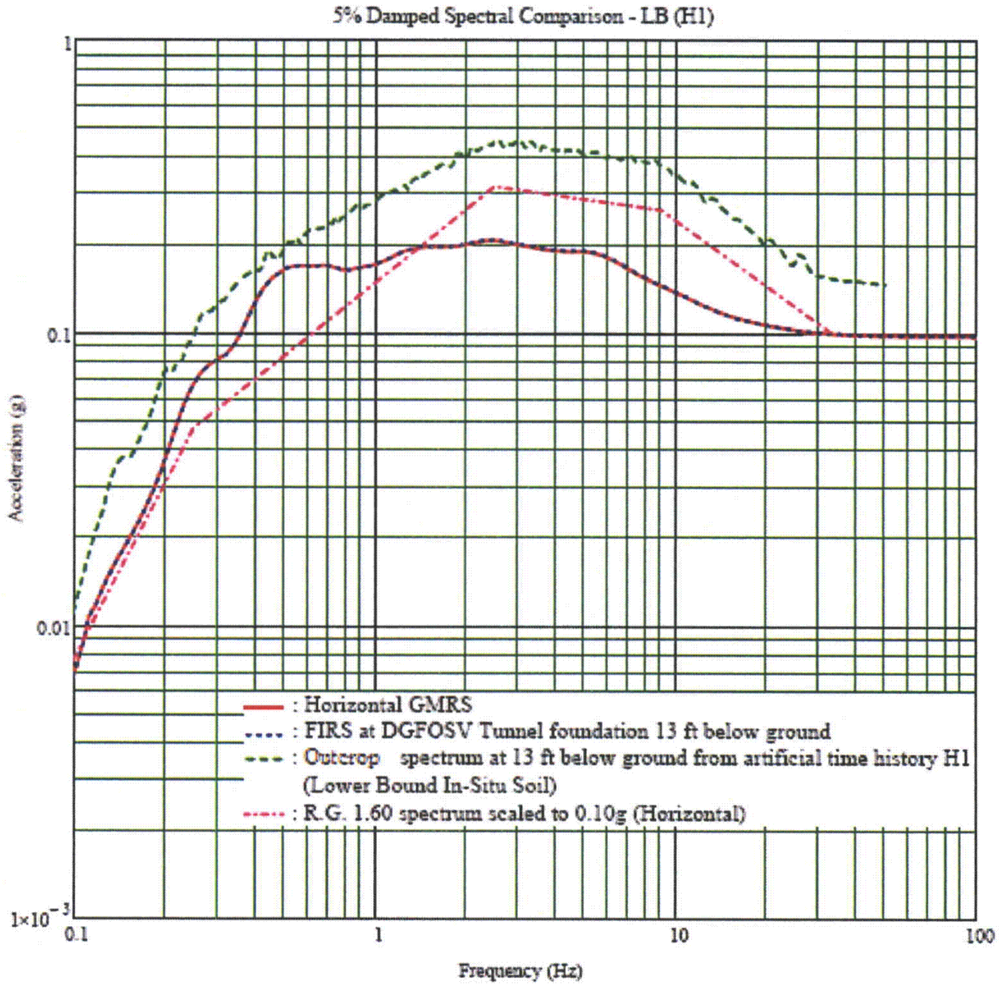


Figure 3H.7-22: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Horizontal X Direction

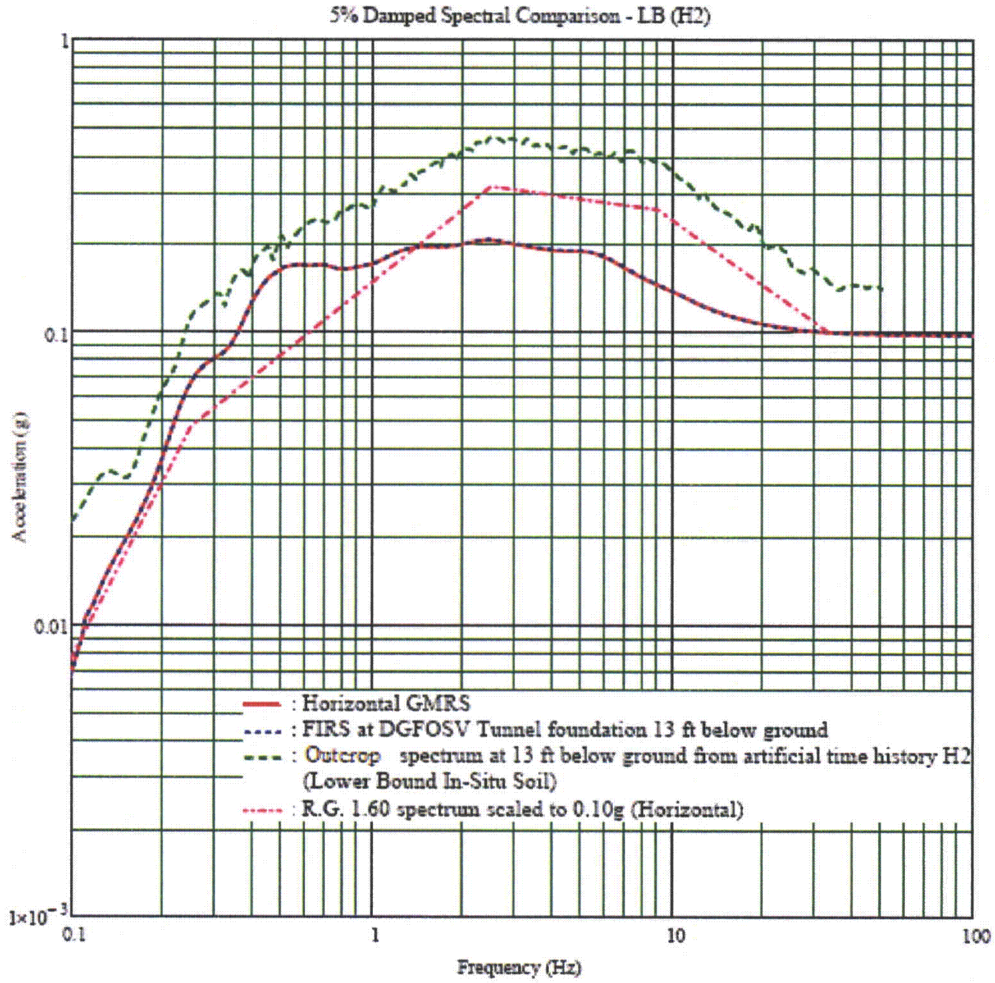


Figure 3H.7-23: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Horizontal Y Direction

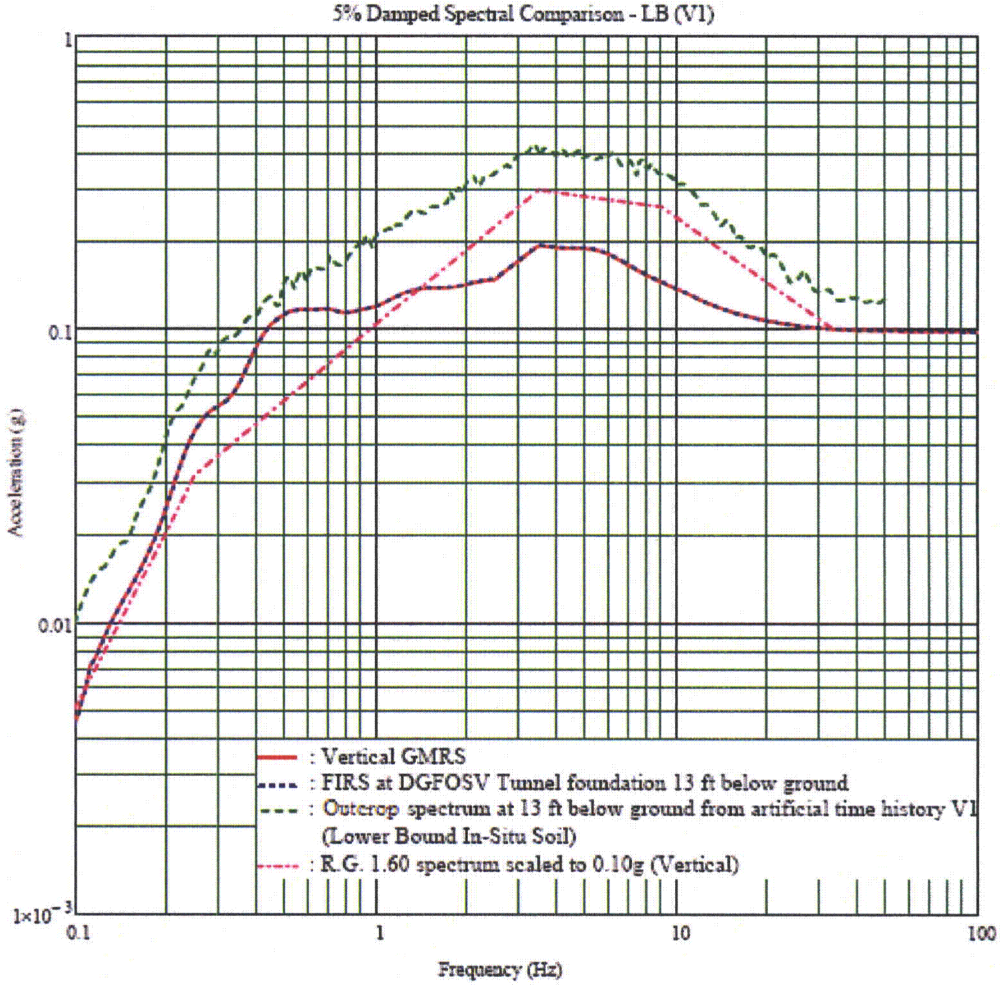


Figure 3H.7-24: Comparison of Spectra at Foundation of DGFOT – Lower Bound Soil Properties, Vertical Direction

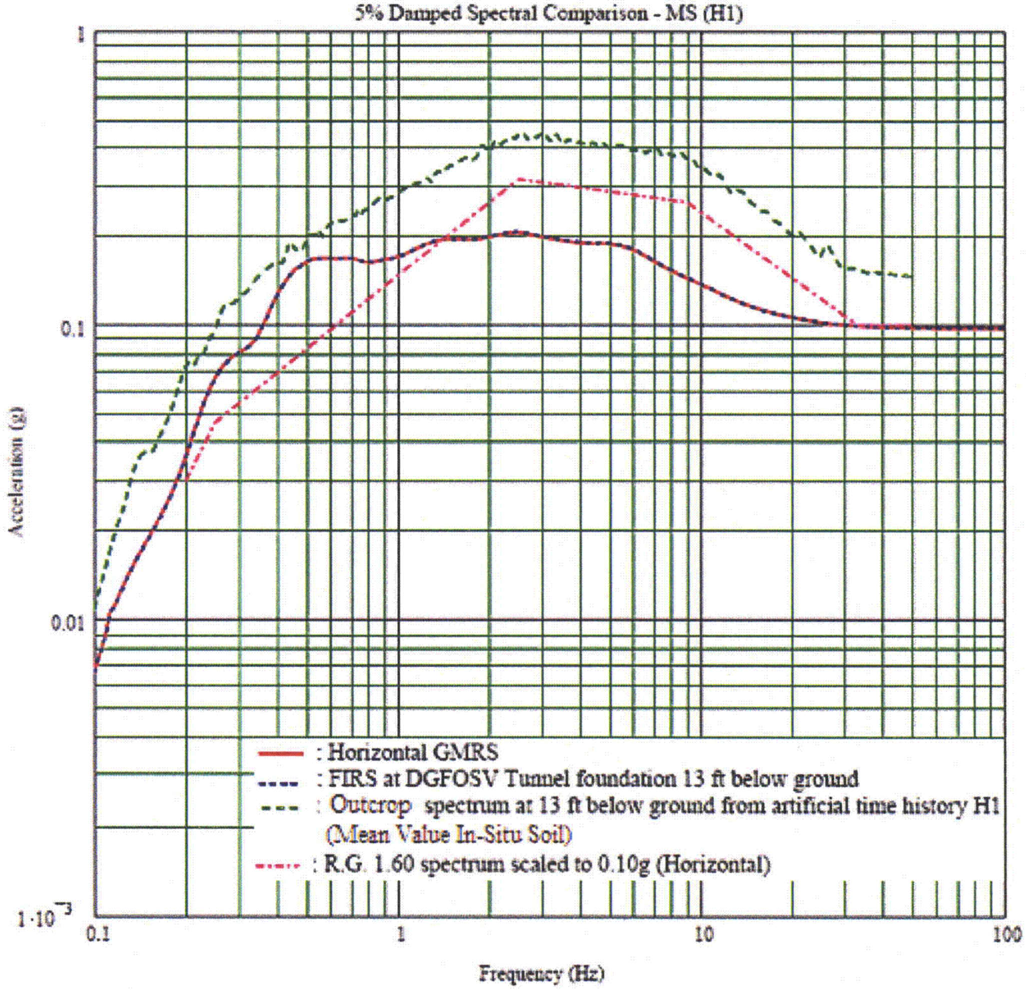


Figure 3H.7-25: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Horizontal X Direction

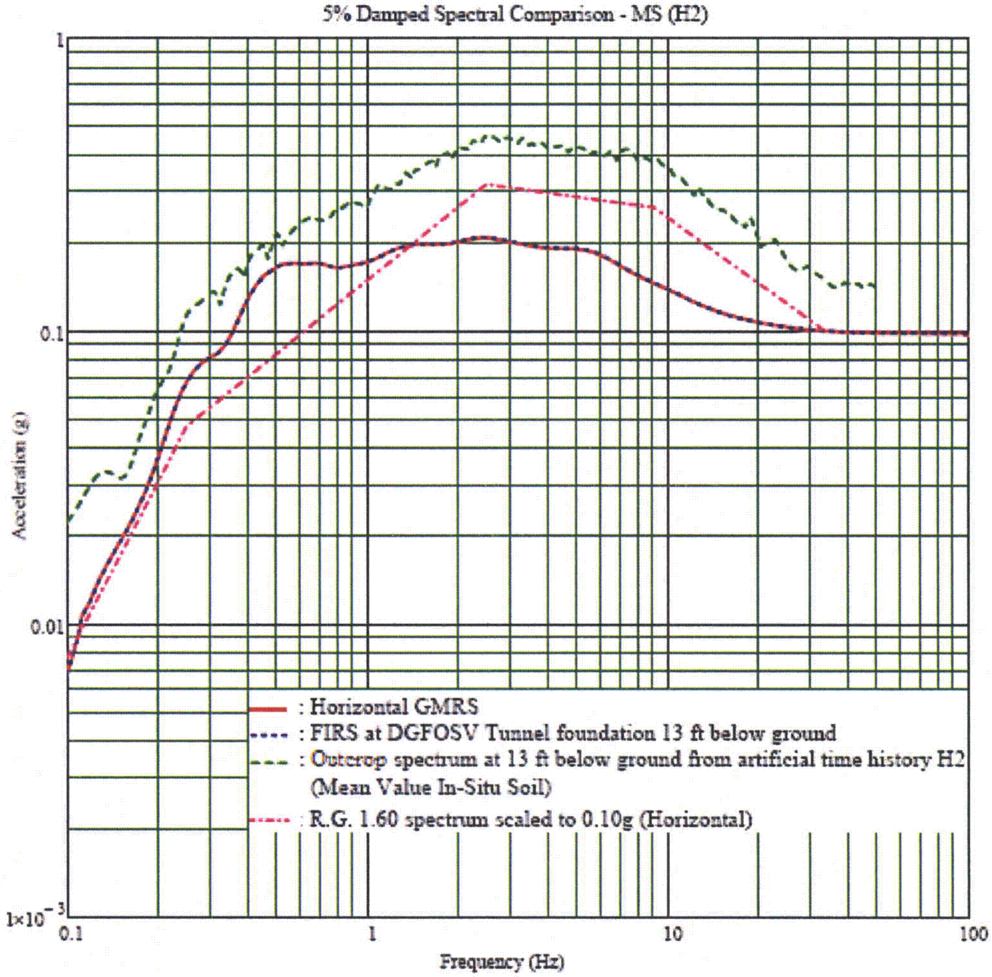


Figure 3H.7-26: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Horizontal Y Direction

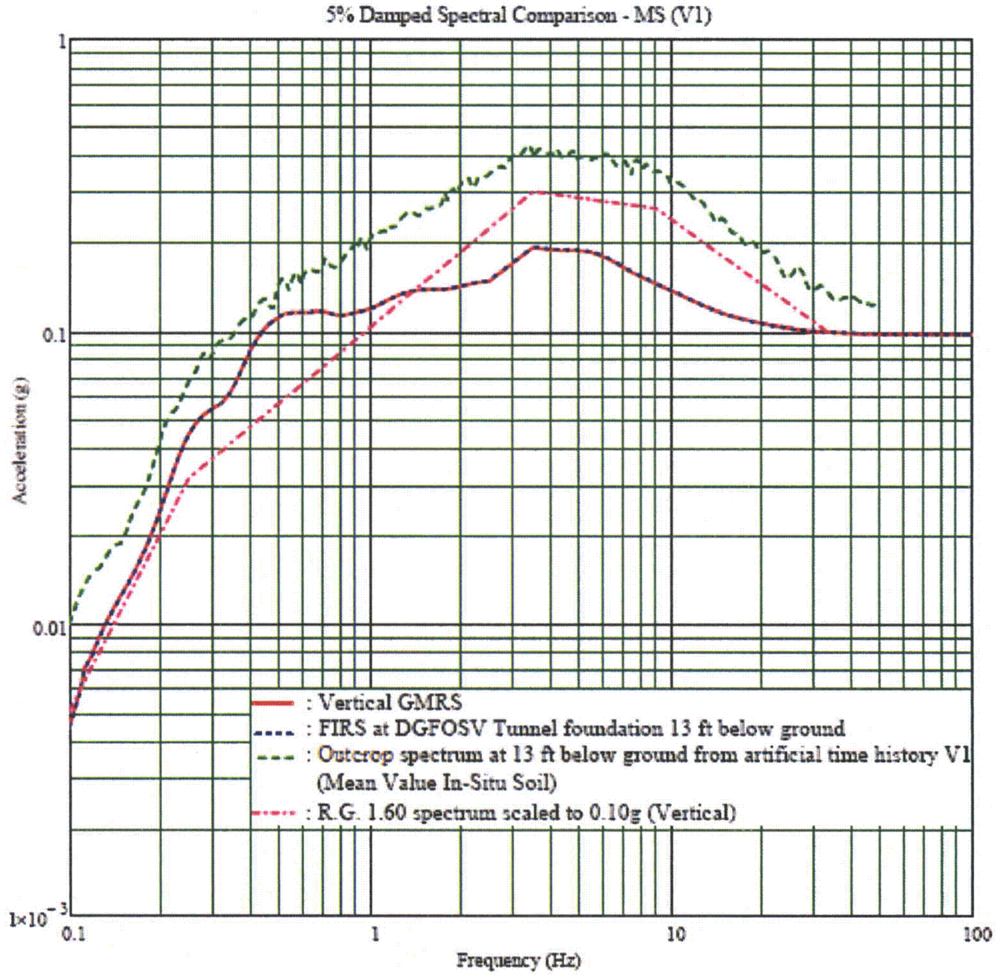


Figure 3H.7-27: Comparison of Spectra at Foundation of DGFOT – Mean Soil Properties, Vertical Direction

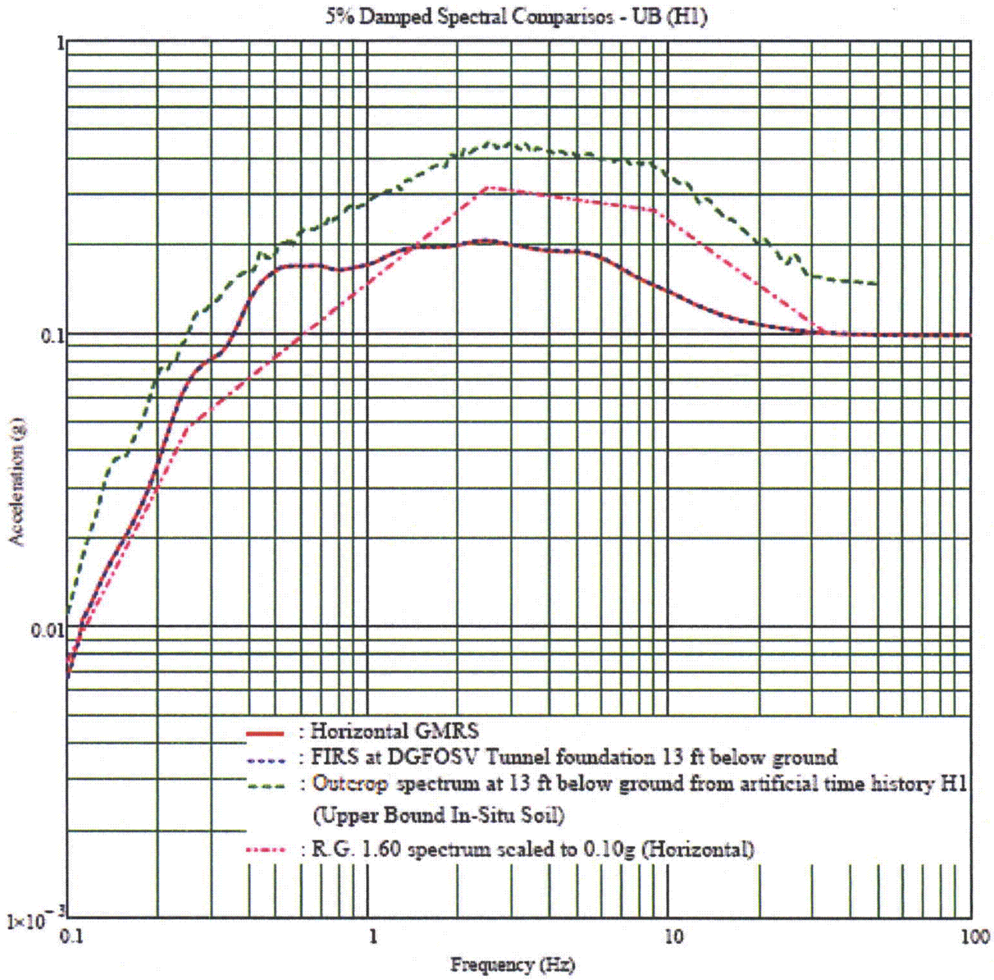


Figure 3H.7-28: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Horizontal X Direction

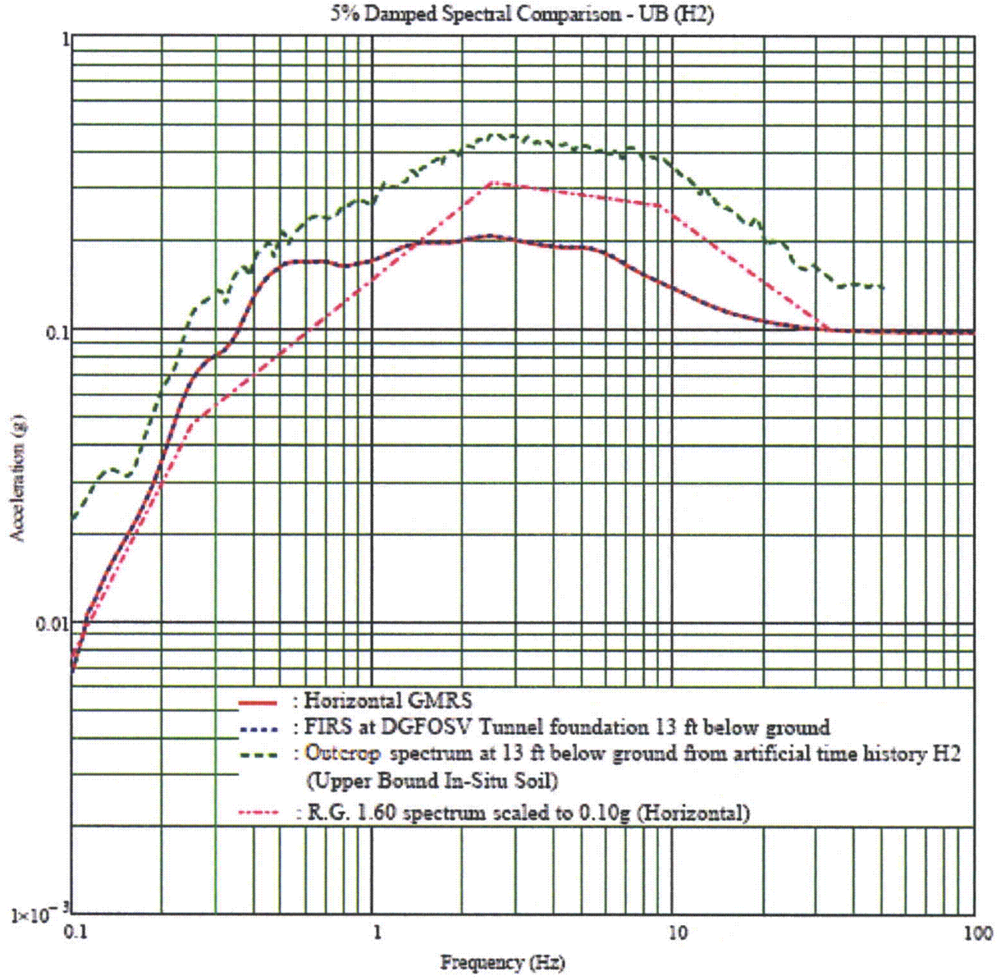


Figure 3H.7-29: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Horizontal Y Direction

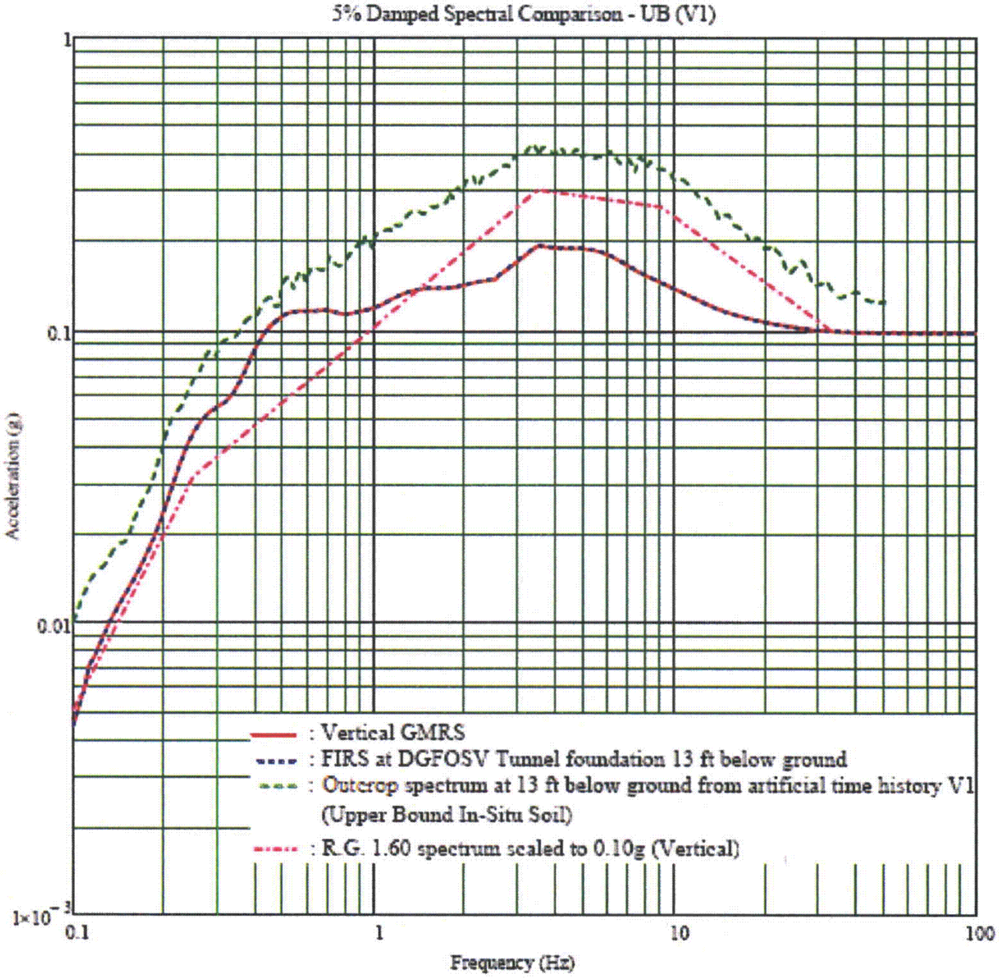


Figure 3H.7-30: Comparison of Spectra at Foundation of DGFOT – Upper Bound Soil Properties, Vertical Direction

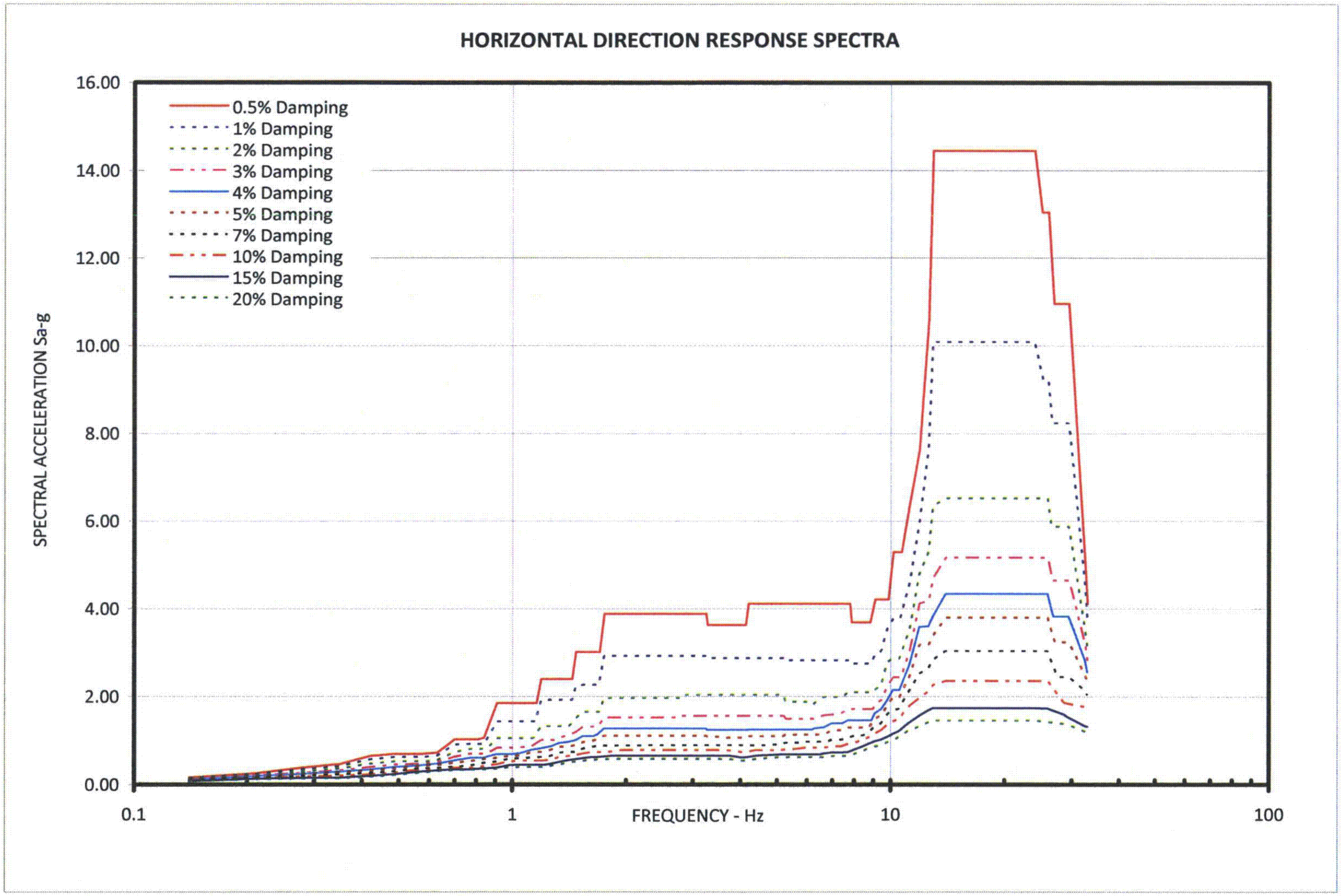


Figure 3H.7-31: Enveloped, Broadened Horizontal Response Spectra for DGFOTs

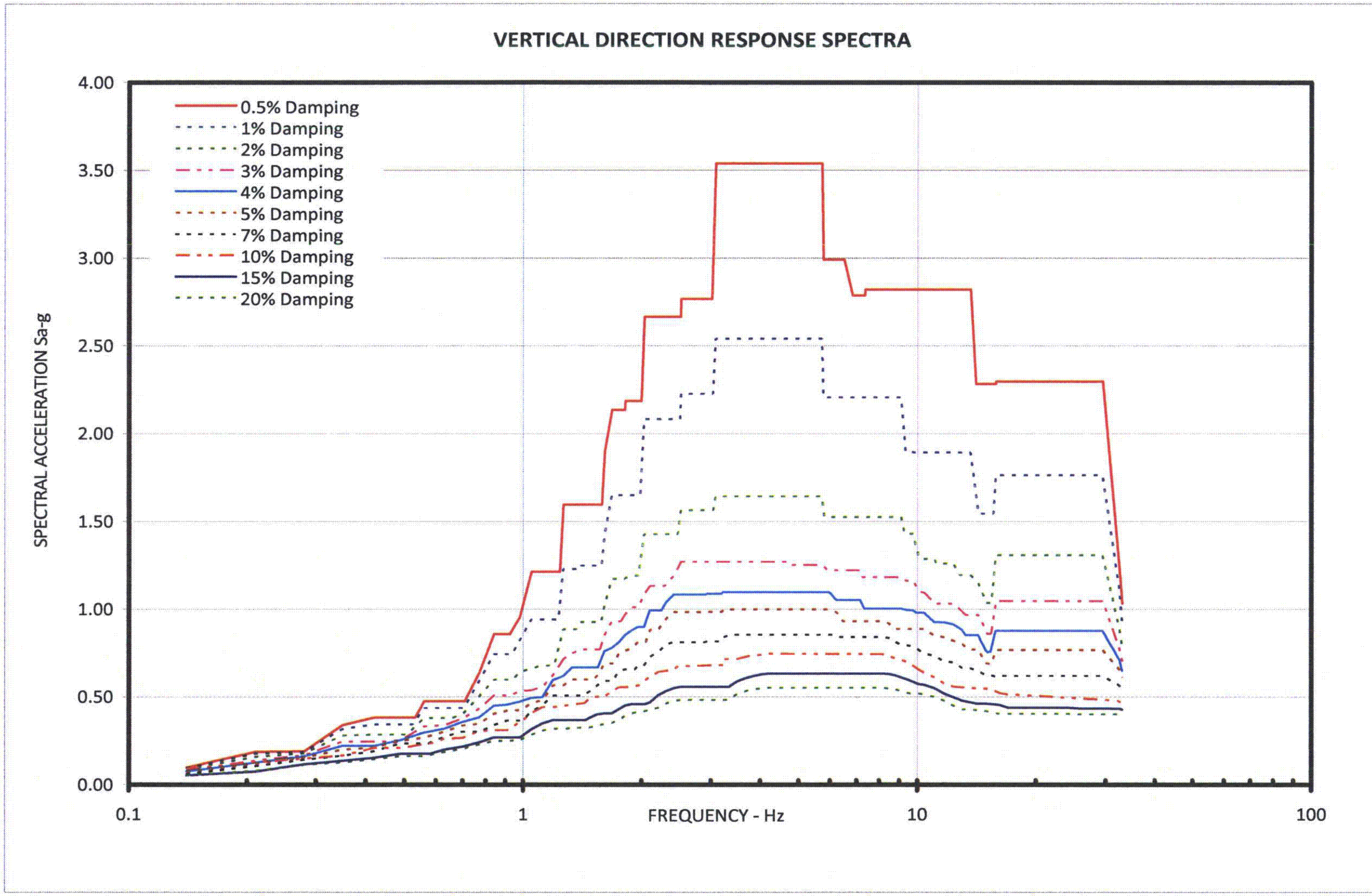


Figure 3H.7-32: Enveloped, Broadened Vertical Response Spectra for DGFOTs

RAI 03.08.04-31, Supplement 1

QUESTION:

Follow-up to Question 03.08.04-25

The staff reviewed the applicant's response to Question 03.08.04-25 (letter U7-C-STP-NRC-100108, dated May 13, 2010). In order for the staff to conclude that the interface between seismic category I buildings and tunnels will not result in any unacceptable interaction, the applicant is requested to provide the following additional information:

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

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

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

SUPPLEMENTAL RESPONSE:

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

2. The layout of the Diesel Generator Fuel Oil Tunnels (DGFOTs) is as shown in COLA Part 2, Tier 2 Figure 3H.6-221 provided in the Supplement 1 response to RAI 03.07.01-27 which was submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010. There are three (3) DGFOTs for each unit and each DGFOT is connected at one end to the Reactor Building (RB) and at the other end to a Diesel Generator Fuel Oil Storage Vault (DGFOSV). There is a seismic gap

between each of the DGFOT and the adjoining RB and DGFOSV. COLA Part 2, Tier 2, Table 3H.6-15 will be revised (see Enclosure 1) to include the required and provided gaps for the DGFOTs. This revised table also incorporates changes due to:

- Revised soil-structure-interaction (SSI) analysis for the Reactor Service Water (RSW) Piping Tunnels described in the Supplement 1 response to RAI 03.07.02-24 which was submitted with STPNOC letter U7-C-STP-NRC-100253 dated November 29, 2010.
- Revised SSI analysis for the DGFOSVs described in the Supplement 1 response to RAI 03.07.01-27 which was submitted with STPNOC letter U7-C-STP-NRC-100274 dated December 21, 2010.

Revision 4 of COLA Part 2, Tier 2 will be revised as shown in Enclosure 1.

RAI 03.08.04-31, Supplement 1
Enclosure 1

Revised Table 3H.6-15

Table 3H.6-15: Required and Provided Gaps at the Interface of Site-Specific Seismic Category I Structures and Diesel Generator Fuel Oil Tunnels with ~~and the~~ Adjoining Structures

Interfacing Structures	Required and Provided Gaps (inches)	
	Required Gap	Provided Gap
RSW Piping Tunnels and Control Building	~~4.41~~4.54	~~4.5~~5.0
RSW Pump House and RSW Piping Tunnel A	~~3.51~~3.99	~~4.5~~5.0
RSW Pump House and RSW Piping Tunnel B	~~4.44~~4.92	~~4.5~~5.0
RSW Pump House and RSW Piping Tunnel C	~~2.59~~3.07	~~4.5~~5.0
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. 1A and its Diesel Generator Fuel Oil Tunnel	~~1.44~~2.37	~~2.0~~3.0
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. ~~2~~1B and its Diesel Generator Fuel Oil Tunnel	~~1.62~~2.60	~~2.0~~3.0
Diesel Generator Fuel Oil Storage Vault (DGFOSV) No. ~~3~~1C and its Diesel Generator Fuel Oil Tunnel	~~1.38~~2.42	~~2.0~~3.0
Reactor Building and Diesel Generator Fuel Oil Tunnel (DGFOT) No. 1A	2.42	3.5
Reactor Building and Diesel Generator Fuel Oil Tunnel (DGFOT) No. 1B	3.07	3.5
Reactor Building and Diesel Generator Fuel Oil Tunnel (DGFOT) No. 1C	2.90	3.5

Note: See Figure 3H.6-221 for layout of the above structures