**Eugene S. Grecheck**
Vice President
Nuclear Development

**Dominion Energy, Inc. • Dominion Generation**
Innsbrook Technical Center
5000 Dominion Boulevard, Glen Allen, VA 23060
Phone: 804-273-2442, Fax: 804-273-3903
E-mail: Eugene.Grecheck@dom.com

**Dominion**®

April 4, 2011

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D. C. 20555

Serial No. NA3-11-009RA
Docket No. 52-017
COL/DWL

**DOMINION VIRGINIA POWER**
**NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION**
**SRP 02.05.04: RESPONSE TO RAI LETTER 59**

On February 9, 2011, the NRC requested additional information to support the review of certain portions of the North Anna Unit 3 Combined License Application (COLA). The response to one of three RAI questions was previously submitted by Dominion letter NA3-11-009R on March 7, 2011. The responses to the two remaining Request for Additional Information (RAI) Questions are provided in Enclosures 1 and 2:

- RAI 5386 Question 02.05.04-22 Concrete Fill Thermal Cracking Prevention
- RAI 5386 Question 02.05.04-24 Total Lateral Earth Pressure Comparison

This information will be incorporated into a future submission of the North Anna Unit 3 COLA, as described in the enclosures.

Please contact Regina Borsh at (804) 273-2247 (regina.borsh@dom.com) if you have questions.

Very truly yours,

Eugene S. Grecheck

D089
NRO

Enclosures:

1. Response to RAI Letter Number 59, RAI 5386, Question 02.05.04-22
2. Response to RAI Letter Number 59, RAI 5386, Question 02.05.04-24

Commitments made by this letter:

1. Incorporate proposed changes in a future COLA submission.

COMMONWEALTH OF VIRGINIA

COUNTY OF HENRICO

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Eugene S. Grecheck, who is Vice President-Nuclear Development of Virginia Electric and Power Company (Dominion Virginia Power). He has affirmed before me that he is duly authorized to execute and file the foregoing document on behalf of the Company, and that the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 4th day of April, 2011
My registration number is 7173057 and my
Commission expires: August 31, 2012

Wanda K. Marshall
Notary Public

WANDA K. MARSHALL
Notary Public
Commonwealth of Virginia
7173057
My Commission Expires Aug 31, 2012

cc: U. S. Nuclear Regulatory Commission, Region II
C. P. Patel, NRC
T. S. Dozier, NRC
J. T. Reece, NRC

# ENCLOSURE 1

## Response to NRC RAI Letter 59

## RAI 5386 Question 02.05.04-22

# RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

**North Anna Unit 3**

**Dominion**

**Docket No. 52-017**

**RAI NO.: 5386 (RAI Letter 59)**

**SRP SECTION: 02.05.04 – STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS**

**QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS1)**

**DATE OF RAI ISSUE: 02/09/2011**

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**QUESTION NO.: 02.05.04-22**

FSAR Section 2.5.4.2.5, Engineering Properties, states that concrete fill will be placed with an average thickness of 15 ft (maximum thickness of 38 ft) below the base of the R/B foundation. Concrete fill will also be placed with an average thickness of 23 ft (maximum of 33 ft) below the base of the West PS/B foundation. Since thermal cracking can be an issue for a large concrete mass, describe how the concrete fill will be placed to reduce thermal cracking distress. Also describe methods to ensure the long term strength and stability of the concrete. This RAI is asked in accordance with 10 CFR 100.23 and to ensure the stability of the building foundations.

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**Dominion Response**

Thermal cracking distress for large concrete masses is a well-recognized issue in the construction industry. The general objective is to limit volume changes and the temperature differential across the concrete as much as practical by properly controlling and/or limiting the heat generated by hydration of the mass concrete. To ensure long term strength and stability of the concrete, as well as to reduce thermal cracking distress, applicable provisions and guidelines of American Concrete Institute standard ACI 349, *Code Requirements for Nuclear Safety-Related Structures,* regarding materials (Chapter 3), durability requirements (Chapter 4), and concrete quality, mixing, and placing (Chapter 5) are used. This includes related ASTM standards and publications of ACI Committees 201 and 207 referenced within ACI 349.

The construction specifications governing the mix design and means and methods of placement for the concrete fill have not yet been developed for the project. However, FSAR Section 2.5.4.2.5 will be revised to include a statement referencing ACI 349 with regard to concrete fill durability, design, construction, and quality assurance. The construction specifications will provide controls on the construction process (including placement techniques), material properties (including mix design and concrete properties during placement; for example slump, air content, and mix temperature) and other variables.

**Proposed COLA Revision**

FSAR Section 2.5.4.2.5 will be revised as indicated on the attached markup.

## Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

Since the Unit 3 power block area is approximately 1500 ft northeast of the center of the Unit 2 Containment Building, the tests focused on verifying that the properties of the soil and rock beneath the Unit 3 power block area were similar to those beneath Units 1 and 2 as determined during previous studies. In addition, chemical tests (for corrosiveness toward buried steel and aggressiveness toward buried concrete) and RCTS tests (for shear modulus and damping ratio variation with cyclic strain) were run on selected saprolite samples.

The details and results of the laboratory testing are included in Appendices 2.5.4AA and 2.5.4CC, except for the RCTS test results which are included in Appendix 2.5.4AAS1. Appendices 2.5.4AA and 2.5.4CC include references to the industry standards used for each specific laboratory test. The results of the tests on soil samples (excluding strength and RCTS tests) are summarized in Table 2.5-210. Table 2.5-211 gives the results of the unconfined compression tests on the rock cores. The results of the RCTS tests are shown in Figure 2.5-223.

The results of the laboratory tests as they relate to the engineering properties of the soil and rock are described in Section 2.5.4.2.5.

### 2.5.4.2.5 Engineering Properties

The engineering properties for Zones IIA, IIB, III, III-IV, and IV derived from the Unit 3 field exploration and laboratory testing programs are provided in Table 2.5-212 and described in the following paragraphs. These engineering properties are similar to those obtained from the previous field and laboratory testing programs (as shown in SSAR Table 2.5-45), with some differences. Where there are differences, the impact from an engineering standpoint is usually either the same or more favorable.

The following paragraphs discuss selected properties shown in Table 2.5-212 under the subheadings: a) rock properties, including concrete fill; b) soil properties, including structural backfill; c) RCTS results; and d) chemical properties.

#### a. Rock and Concrete Fill Properties

In general, the rock strength and stiffness values, derived from the field and laboratory testing of the Unit 3 rock, are higher than given in the SSAR. This could reflect less fractured or weathered rock beneath the

Unit 3 area, and/or better rock coring equipment and techniques that produced better quality cores.

The Recovery and RQD are based on the results presented for each core in the boring logs in Appendices 2.5.4AA, 2.5.4BB, and 2.5.4CC. The RQDs from the borings for Strata III, III-IV and IV are plotted versus elevation in Figure 2.5-224. For Stratum III, RQD generally ranges from zero to around 50 percent, with some higher values. The average value is about 20 percent. For Stratum III-IV, RQD generally ranges from around 50 to 90 percent. The average value is about 65 percent (compared to 50 percent in the SSAR). For Stratum IV, RQD is generally above 80 percent and mostly above 90 percent. The average value is about 95 percent. The average recovery values for Zone III, III-IV and IV are about 50 percent, 90 percent, and 98 percent, respectively.

The unconfined compressive strengths and unit weights in Table 2.5-212 are based on the rock strength test results shown in Table 2.5-211. The elastic modulus values are also based on the values shown in Table 2.5-211. The shear modulus values are derived from the elastic modulus values using the Poisson's ratio values tabulated in Table 2.5-212. These higher strain shear modulus values agree well with the low strain values derived from the geophysical tests performed for the Unit 3 exploration program described in Section 2.5.4.4. These high and low strain shear modulus values are essentially the same for high strength rock, certainly for the Zone IV and Zone III-IV rock. Some strain softening has been allowed in the case of the Zone III rock, as described in Section 2.5.4.7. Low strain is defined here as $10^{-4}$ percent while high strain is taken as 0.25 to 0.5 percent, the amount of strain frequently associated with settlement of structures on soil.

The shear and compression wave velocities in Table 2.5-212 are based on suspension P-S velocity logging performed as part of the Unit 3 exploration program (Appendices 2.5.4AA and 2.5.4CC). These results are summarized in Section 2.5.4.4.4.

**Concrete Fill**

As stated in Section 2.5.4.10, if Zone II saprolitic soils and/or Zone III weathered rock is encountered at foundation subgrade level of the R/B, Power Source Buildings (PS/Bs), and Power Source Fuel Storage Vaults (PSFSVs), they will be removed and replaced with concrete fill. Concrete fill will also replace Zone II saprolitic soils beneath the remaining seismic

category I structures, i.e., Ultimate Heat Sink Related Structures (UHSRS), UHSRS pipe chase, and Essential Service Water Pipe Tunnel (ESWPT). The concrete fill will have a minimum strength of 2500 psi, with a unit weight and Poisson's ratio of 145 pcf and 0.15, respectively. The bearing capacity of concrete fill is addressed in Section 2.5.4.10.1.

Figures 2.5-229, 2.5-230, and 2.5-233 show weathered rock will be removed and replaced by concrete fill from up to about 38 ft depth below the base of the R/B foundation, with an average thickness of about 15 ft. Figure 2.5-233 shows weathered rock will be removed and replaced by concrete fill from up to 33 ft depth below the base of the West PS/B foundation, with an average thickness of about 23 ft. Figures 2.5-231 and 2.5-232 show as much as 60 ft of Zone II and Zone III materials being replaced by concrete fill beneath portions of the East and West PSFSVs.

Analysis indicates that if the top 25 ft of Zone III rock beneath the R/B foundation is replaced with concrete, the seismic response at foundation level decreases with increasing shear wave velocity ($V_S$) of the concrete. Based on the calculated log-mean $V_S$ values at and below the R/B foundation (shown in Figure 2.5-241a), the $V_S$ of the in-situ rock at 25 ft below the R/B foundation base is approximately 5000 ft/sec. Therefore, the $V_S$ of the concrete fill should be equal to or greater than 5000 ft/sec to ensure that the seismic response of the column that includes the concrete fill is equal to or less than the response from the original analysis of the in-situ rock. Further analysis indicates that concrete with strength of 2500 psi has a $V_S$ of at least 6295 ft/sec.

Construction, quality assurance, and engineering properties of the concrete fill,including strength and durability, are controlled through project specifications. The project specifications provide controls on the construction process (including placement techniques), material properties (including mix design and concrete properties during placement; for example slump, air content, and mix temperature) and other variables. The concrete fill is required by project specifications to conform to the pertinent provisions of ACI 349 (Reference 2.5-215) including provisions contained in ASTM standards and ACI Committee 201 and 207 publications referenced within ACI 349.

# ENCLOSURE 2

## Response to NRC RAI Letter 59

## RAI 5386 Question 02.05.04-24

# RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

**North Anna Unit 3**

**Dominion**

**Docket No. 52-017**

**RAI NO.: 5386 (RAI Letter 59)**

**SRP SECTION: 02.05.04 – STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS**

**QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS1)**

**DATE OF RAI ISSUE: 02/09/2011**

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**QUESTION NO.: 02.05.04-24**

FSAR Section 2.5.4.10.3, Earth Pressures, discusses static and seismic lateral earth pressures for plant below-ground walls and refers to details provided in Appendix 3NN. However, after review of this section and appendix, the staff did not find a comparison between the total lateral earth pressure (static and dynamic) and the standard plant design. Please provide this comparison of the site-specific values to the standard design. This RAI is in accordance with 10 CFR 100.23.

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**Dominion Response**

The current FSAR Appendix 3NN, Section 3NN.6 addresses the seismic/dynamic lateral earth pressure distribution for the Unit 3 structures. This section refers to Figure 3NN-22 for the comparison of the dynamic lateral earth pressures to the standard plant design. Figure 3NN-22 will be revised to show a comparison of the North Anna Unit 3 total lateral earth pressures (static and dynamic) versus the standard plant total lateral earth pressures (static and dynamic). Appropriate related portions of the FSAR will also be revised to describe the new comparison presented in Figure 3NN-22. The revised Figure 3NN-22 shows that the Unit 3 total lateral earth pressures (static and dynamic) are enveloped by the standard design total lateral earth pressures (static and dynamic). A detailed description of the comparison follows.

MHI Technical Report MUAP-10006, Revision 1, Table 4-12 and Figure 3-8 present the static and dynamic lateral earth pressure loading applicable to the standard plant design exterior below-grade walls of the Reactor Building Complex and Power Source Building. Section 3.7 in MUAP-10006 describes the methodology that is used to compute the

static and dynamic lateral earth pressure loading. MUAP-10006, Revision 1, includes increases in the standard plant dynamic lateral earth pressure loading that were implemented in response to DCD RAI 660-5134 Question 03.07.02-62. The proposed revision for Figure 3NN-22 includes those changes.

For computing static lateral earth pressures on below-grade building walls, the standard plant design considers structural fill with a total unit weight of 130 pcf, groundwater level which meets or exceeds the maximum groundwater level in DCD Tier 2 Table 2.0-1, and a Rankine at-rest pressure coefficient of 0.50. The standard plant design also considers lateral loading on below-grade walls due to adjacent surface surcharge loads. As stated in Section 3NN.6 of Appendix 3NN, for computing static lateral earth pressure for the Unit 3 analyses of the Reactor Building Complex and Power Source Buildings, the analysis considers structural fill with a total unit weight of 130 pcf, groundwater level at 7 ft below plant grade, and an at-rest pressure coefficient of 0.36, based on data presented in FSAR Table 2.5-212 and FSAR Figure 2.4-216. Further discussion of the Unit 3 characteristics is contained in FSAR Sections 2.5.4.10.3 and 2.4.12.4. The Unit 3 methodology for computing static lateral earth pressure loading on basement walls is described in FSAR Section 2.5.4.10.3, and is consistent with the standard plant methodology. Static lateral earth pressures are computed using classic formulas in which earth pressure, which includes hydrostatic pressure, increases linearly with depth. Lateral load on below-grade walls due to surcharge loading is also considered. Lateral load due to surcharge loading is computed by multiplying the Rankine at-rest coefficient times the surcharge area load. Figure 2.5-254 shows a general static earth pressure diagram for nonyielding walls which includes a lateral load due to surcharge. Because the key characteristics (unit weight of structural fill, at-rest pressure coefficient, and groundwater level) for the standard plant are the same as or envelop the corresponding Unit 3 characteristics, the standard plant static lateral earth pressure loading envelopes the Unit 3 static lateral earth pressure loading.

Therefore, as discussed in this response and as shown in the proposed revision for Figure 3NN-22, the standard plant total earth pressure loading (static and dynamic) envelopes the Unit 3 total lateral earth pressure loading (static and dynamic).

Figure 3NN-22 presents plots for combined static and dynamic loading in which the loading is added algebraically for purposes of comparison. However, for purposes of structural design and evaluation, the load factors and load combinations are in accordance with FSAR Section 3.8.

## Proposed COLA Revision

FSAR Sections 2.5.4.10.3, 3.7.2.4.1, 3.7.2.8, 3.7.5, FSAR Appendix 3NN,Section 3NN.6, and FSAR Figure 3NN-22 will be revised as indicated on the attached markups.

# Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

$h_i$ = thickness of layer i

$E_i$ = elastic modulus of layer i

The stress distribution below rectangular foundations is based on a Boussinesq-type distribution for flexible foundations (Reference 2.5-216). The computation extends to a depth where the increase in vertical stress ($\sigma_z$) due to the applied load is equal to or less than 10 percent of the applied foundation pressure. The Boussinesq-type vertical pressure under a rectangular footing, $\sigma_z$, is as follows (Reference 2.5-216):

$$\sigma_z = (p/2\pi)\{\tan^{-1}[lb/(zR_3)] + (lbz/R_3)(1/R_1^2 + 1/R_2^2)\}$$

where:

p = applied foundation pressure

l = length of footing

b = width of footing

z = depth below footing at which pressure is computed

$R_1 = (l^2 + z^2)^{0.5}$

$R_2 = (b^2 + z^2)^{0.5}$

$R_3 = (l^2 + b^2 + z^2)^{0.5}$

Settlement estimates were made using the preceding relationships and the concrete fill and rock properties given in Table 2.5-212. These estimates were made for each seismic category I and II structure, and are presented in Table 2.5-216, based on applied pressures from the foundations given in Table 2.5-215 and included in Table 2.5-216. The maximum estimated total settlement for any structure is about 0.049 in. This is also, conservatively, the maximum differential settlement within or between structures.

Note that the total and differential estimated settlements under the R/B foundation are within the limits stated in Table 2.0-201 by at least a factor of ten.

### 2.5.4.10.3 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant below-ground walls. Both active and at-rest cases are included. Active earth pressure is used for temporary retaining walls installed to facilitate construction. For these, the earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall. At-rest earth pressures are used for basement walls. Hydrostatic pressures are based on the groundwater table being 1 ft

below grade. This is conservative based on the predicted groundwater levels in the power block area summarized in Section 2.4.12. An area wide surcharge pressure of 500 psf is used. Lateral pressures due to compaction are not included; these pressures are controlled by compacting backfill with light equipment near structures. The soil properties used in the calculation of lateral earth pressures are from Table 2.5-212.

For the active lateral earth pressure case, earthquake-induced horizontal ground accelerations are addressed by the application of $k_h \cdot g$. Vertical ground accelerations ($k_v \cdot g$) are considered negligible and were ignored (Reference 2.5-217). The peak low frequency acceleration of 0.31g was used for developing the seismic active earth pressure diagram. Use of the peak high frequency acceleration was considered overly conservative given the low magnitude (energy) of this earthquake.

The method described in ASCE 4-98 Section 3.5.3.2 (Reference 2.5-218) can be used to estimate the dynamic component of seismic at-rest lateral earth pressure for the below-grade walls of the power block structures. Reference 2.5-218 provides an elastic solution that is demonstrated by a nomograph. In the nomograph, a dimensionless normalized in-situ lateral stress at 1.0g horizontal earthquake acceleration is developed for a normalized depth at a given Poisson's ratio. The appropriate site specific at-rest pressure is calculated from the nomograph at various depths intervals using the site-specific acceleration and Poisson's ratio.

Figure 2.5-229 through Figure 2.5-234 illustrate the stage of construction where excavation is complete and the basement walls are in place. Although not shown in the figures, structural backfill will be placed around the basement walls up to the final grade. Referring to, for example, Section B-B' in Figure 2.5-230, the temporary vertical wall on the left side of the figure will be used to facilitate construction. This wall will hold back the natural in-situ soil (Zone IIA and IIB saprolites) which will be in an active condition. Once structural backfill is placed between this temporary wall and the wall of the R/B, the backfill against the R/B wall will be in an at-rest condition since the R/B wall will not deflect. In this situation, the lateral earth pressures against the vertical excavation support wall can have some influence on the earth pressure against the RB wall. Thus, active earth pressures due to the Zone IIA and IIB saprolites are included here.

The lateral earth pressure diagrams for the active cases (including seismic) is given in Figure 2.5-253. The lateral earth pressure diagram (static only) for the at-rest case is shown in Figure 2.5-254. The seismic component of the at-rest lateral earth pressure is presented in more detail in Appendix 3NN. A typical seismic lateral earth pressure diagram (based on the Reference 2.5-218 method) is shown in Figure 3NN-22. Figure 3NN-22 also includes plots of the site-specific total lateral earth pressure and the standard plant total lateral earth pressure (which include both the static and dynamic components). All of these figures show lateral pressures at about 40 ft depth, i.e., at about the bottom of the R/B (and the PS/B).

Note that the lateral pressures in Figures 2.5-253 and 2.5-254 are best estimate pressures with a factor of safety of 1. Appropriate safety factors need to be incorporated into the wall structural design. The factor of safety against a gravity wall or structure foundation sliding is normally taken as 1.1 when seismic pressures are included. The same factor of safety is applied against a wall overturning.

### 2.5.4.11 Design Criteria

**NAPS ESP COL 2.5-7**

Applicable design criteria are covered in various sections. The criteria summarized below are geotechnical criteria and also geotechnical-related criteria that pertain to structural design.

Section 2.5.4.8 specifies that the acceptable factor of safety against liquefaction of site soils is ≥1.1.

Bearing capacity and settlement criteria are presented in Section 2.5.4.10. Table 2.5-215 provides allowable bearing capacity values for the seismic category I and II structures. A minimum factor of safety of 3 is used when applying bearing capacity equations. This factor of safety is also applied against breakout failure due to uplift forces on buried piping.

Section 2.5.4.10 also discusses factors of safety related to lateral earth pressures. The lateral pressures shown in Figures 2.5-253 and 2.5-254 have a factor of safety of 1. A factor of safety of 1.1 should be used in the analyses of sliding and overturning due to these lateral loads when the seismic component is included.

Section 2.5.5.2 specifies that the minimum acceptable long-term static factor of safety against slope stability failure is 1.5. Section 2.5.5.3

methods used for the SSI analyses are discussed further in Technical Report MUAP-10001 (Reference 3.7-201).

Replace the fourth and fifth paragraphs in DCD Subsection 3.7.2.4 with the following.

The SSI analyses are performed as previously described by analyzing the seismic models in ACS SASSI (Reference 3.7-17R) in conjunction with the generic soil profiles with depths to rock as discussed in Section 3.7.1.3, including a profile which simulates a site with hard rock at a depth of 100 ft. The profiles have dynamic properties that are strain-compatible with the CSDRS input ground motion as discussed in Section 3.7.1.3, and the input ground motion time histories are applied separately in each direction in the SASSI analyses.

### 3.7.2.4.1 Requirements for Site-Specific SSI Analysis of US-APWR Standard Plant

**NAPS COL 3.7(25)**
**NAPS DEP 3.7(1)**

Replace the first paragraph in DCD Subsection 3.7.2.4.1 with the following.

The site-specific SSI analyses for the R/B-PCCV-containment internal structure and PS/Bs are performed using the program ACS SASSI (Reference 3.7-17R). Sets of analyses are performed on the R/B-PCCV-containment internal structure and PS/Bs to evaluate the effects of site-specific seismological and geotechnical conditions and the incoherence of the input ground motion. As discussed earlier in Section 3.7, the analyses utilize site-specific SSI Input motion documented in Section 3.7.1 and Appendix 3OO for evaluation of the standard design against the spectral exceedances of the standard plant CSDRS. The modeling and site-specific SSI analyses of the R/B-PCCV-containment internal structure (also referred to herein as the R/B complex) and PS/Bs are addressed in Appendix 3NN.

**NAPS COL 3.7(26)**

Replace the second paragraph in DCD Subsection 3.7.2.4.1 with the following.

The site-specific SSI analyses of the UHSRS, ESWPT, and PSFSVs are also performed using the computer program ACS SASSI (Reference 3.7-17R). The SASSI analyses for these structures are performed using the same methodology as the site-specific SASSI analyses of the R/B complex and PS/Bs, except that the appropriate

site-specific SSI Input motion for each structure is used and the incoherency of the input motion is not considered. The SSI analyses and results for the UHSRS, ESWPT, and PSFSVs are addressed in further detail in Appendix 3LL.

**NAPS COL 3.7(20)**

Site-specific SSI analyses are required to confirm that the seismic designs of the A/B and T/B are suitable for the site-specific seismic input motion and the range of site-specific subgrade conditions, in accordance with the guidance given in SRP 3.7.2.

**NAPS COL 3.7(25)**

Replace the first sentence of the third paragraph in DCD Subsection 3.7.2.4.1 with the following.

The site-specific seismic response analyses address factors that affect the response of the combined soil-structure dynamic system that include, but are not limited to, the following.

Replace the last two sentences of the fifth paragraph in DCD Subsection 3.7.2.4.1 with the following.

The value of $C_V$ used to determine bounding values for all subgrade and backfill shear moduli in the SSI analyses is at least 0.5, corresponding to a well investigated site.

**NAPS COL 3.7(8)**
**NAPS COL 3.7(25)**
**NAPS COL 3.7(26)**
**NAPS DEP 3.7(1)**
RAI 02.05.02-1 **NAPS DEP 3.7(2)**

Replace the sixth, seventh, and eighth paragraphs with the following.

The SSI analyses use stiffness and damping properties of the subgrade materials that are compatible with the strains generated by the site-specific design earthquake which is based on the FIRS discussed in Section 3.7.1.1. In accordance with guidance of SRP 3.7.2, the soil material damping considered in Appendices 3OO and 3NN does not exceed 15%.

All standard plant and site-specific seismic category I and II buildings and structures are founded on sound rock or on a layer of fill concrete placed on the sound rock (Zone III and/or Zone III-IV rock as described in Section 2.5.4 and SSAR (R9) Subsection 2.5.1.2.3). The R/B complex, the PSFSVs, and the PS/Bs are founded completely on Zone III-IV bedrock or on fill concrete placed on Zone III-IV bedrock.

The fill concrete has a minimum design compressive strength of 2,500 psi as stated in Section 2.5.4 and a best estimate shear wave velocity of 7,000 ft/s as stated in Appendix 3OO. The fill concrete is installed as a leveling layer and to fill the space between the basemat

and the top of competent rock. Fill concrete may be used as "dental" fill in any areas where additional removal of materials below the anticipated bottom of excavation is required in order to reach competent rock. To ensure that the effects of the fill concrete on the seismic response are adequately captured, the fill concrete is included in analyses to develop the SSI Input motions, except for the ESWPT and UHSRS pipe chase, as discussed further in Appendix 3OO.

Section 2.5.4 discusses the dynamic properties of the rock subgrade. The dynamic properties of the Zone III-IV slightly to moderately weathered rock and underlying Zone IV parent bedrock are strain independent. The Zone III rock exhibits reduction of shear modulus due to strain. The reduction in shear modulus is taken into account for the soil/rock column amplification/attenuation analyses used to develop the FIRS and SSI Input motions in Appendix 3OO, and is used in the modeling of the subgrade in the SASSI analyses. Figures 3NN-1 through 3NN-6 in Appendix 3NN present the sets of site-specific S-wave, PWave, and damping profiles used in the SASSI validation analyses for the R-B and PS/Bs.

As previously mentioned in DCD Subsection 3.7.2.4, the seismic design of the standard plant buildings does not rely on the backfill present on the sides of the building to derive lateral or structural support. Further, the seismic designs of the site-specific seismic category I structures, including the UHSRS, ESWPT, and PSFSVs, also do not rely on backfill for lateral or structural support. The designs of exterior below-grade walls do consider applicable earth pressures generated by the design earthquake.

For purposes of site-specific seismic analysis, the water table for each structure is considered based on groundwater levels reported in Section 2.4.12. The P-wave velocities of the saturated fill concrete and rock layers exceed the P-wave velocity of the water (5,000 ft/s). Therefore, the water table elevation does not affect the P-wave velocities of these materials as used in the SSI analyses. To account for the presence of groundwater, the Poisson's ratio for backfill materials is adjusted as discussed in Appendix 3OO, Section 3OO.2, to obtain the corresponding values for the strain-compatible backfill P-wave velocities used in the validation analyses documented in Appendix 3NN.

The same dynamic models used for the standard plant analyses are used for the site-specific SSI validation analyses. The properties of the ACS

SASSI (Reference 3.7-17R) seismic models have been verified by SSI analyses of the buildings resting on the surface of a hard rock subgrade that simulates fixed base conditions. The results of the SASSI analyses match the results from the time history analyses of fixed base models.

**NAPS COL 3.7(23)**
**NAPS DEP 3.7(1)**

Replace the last sentence of the ninth paragraph in DCD Subsection 3.7.2.4.1 with the following.

The results of the site-specific SSI analysis documented in Appendix 3NN demonstrate that the standard plant seismic design of structural members envelopes the site-specific seismic responses (demands) for the R/B complex and the PS/Bs. Tables 3NN-20A, 3NN-20B, 3NN-20C, 3NN-20D, and 3NN-20E demonstrate that the standard plant seismic design loading for the R/B complex envelopes the responses (demands) obtained from the site-specific seismic analyses of the R/B complex. Tables 3NN-21A and 3NN-21B demonstrate that the standard plant seismic design loading for the PS/Bs envelopes the seismic responses (demands) obtained from the site-specific seismic analyses of the PS/Bs. Figure 3NN-22 demonstrates that the total (static and dynamic) site-specific lateral earth pressures on basement walls ~~due to earthquake loading~~ are enveloped by the US-APWR standard design.

RAI 02.05.02-24

Broadened ISRS are developed for the R/B complex and PS/Bs as described in Section 3.7.2.5. The standard plant and site-specific 5% damping broadened ISRS for the R/B complex are presented in Figure 3NN-23. The standard plant and site-specific 5% damping broadened ISRS for the PS/Bs are presented in Figure 3NN-21. The standard plant broadened ISRS do not envelope all of the corresponding site-specific broadened ISRS at all frequencies.

The seismic designs of standard plant subsystems affected by site-specific exceedances of the standard plant input motion are required to be evaluated to confirm suitability of the designs in accordance with SRP 3.7.3. To evaluate the exceedances of the standard plant broadened ISRS, the seismic design of each standard plant subsystem is required to be reviewed against the seismic responses corresponding to the site-specific broadened ISRS and obtained from the site-specific input motion. At locations where envelopment of the applicable site-specific broadened ISRS does not occur, the suitability of the standard design is to be confirmed by re-analysis using site-specific input motion. Where the standard plant subsystem design envelopes the

### 3.7.2.8 Interaction of Non-Category I Structures with Seismic Category I Structures

**NAPS COL 3.7(10)**

Replace the last sentence of the fifth paragraph in DCD Subsection 3.7.2.8 with the following.

Structure-to-structure interactions, which could potentially influence the seismic response levels, will not occur because all seismic category I and II structures are founded on stiff rock or on fill concrete placed on the stiff rock (Zone III or Zone III-IV rock as described in Section 2.5.4 and SSAR Subsection 2.5.1.2.3). Seismic Category I and II structures are also separated by expansion joints which prevent seismic interaction.

Site-specific conditions do not result in structure-to-structure interaction effects or exceedances of the assumed pressure distributions used for the US-APWR standard plant design. As stated in FSAR Subsection 3.7.2.4.1, Figure 3NN-22 of Appendix 3NN demonstrates that the total (static and dynamic) site-specific lateral earth pressures on basement walls ~~due to earthquake loading~~ are enveloped by the US-APWR standard design.

**NAPS COL 3.7(9)**

Replace the seventh paragraph in DCD Subsection 3.7.2.8 with the following.

The site-specific category I SSCs are the UHSRS, the ESWPT, and the PSFSVs. The layout design of the site-specific seismic Category I SSCs ensures that there are no adjacent non-seismic Category I structures which may adversely affect these structures, to protect them from structural failure of non-seismic category I structures.

#### 3.7.2.8.6 PS/Bs

**NAPS DEP 3.7(3)**

Replace the first sentence in the first paragraph in DCD Subsection 3.7.2.8.6 with the following.

The US-APWR standard plant PS/Bs are Seismic Category I structures and their seismic modeling and analyses are described in Technical Reports MUAP-10001 and MUAP-10006 (References 3.7-201

Three sets of broadened ISRS are developed that represent the enveloped response for different site conditions considered that are obtained from SSI analyses of: 1) surface foundation with coherent motion, 2) embedded foundation with coherent motion and 3) surface foundation with incoherent motion. The ISRS obtained from the SSI analyses using coherent input ground motion are compared to quantify the amplification of the ISRS due to the embedment effects. The final broadened incoherent ISRS curves are obtained by adjusting the ISRS developed from the SSI analyses of surface foundation with incoherent input ground motion to incorporate the amplifications that are due to the embedment effects. These ISRS that include foundation embedment and ground motion incoherency effects define the site-specific seismic demands for Seismic Category I and II SSCs and subsystems.

Figure 3NN-23 (Sheets 1 through 143) presents the 5 percent critical damping broadened ISRS developed for the R/B complex from the site-specific SSI analysis results. Figure 3NN-21 (Sheets 1 through 12) presents the 5 percent critical damping broadened ISRS obtained for the PS/B from the site-specific SSI analysis results. Each of the plots in the figures shows two site-specific ISRS representing the response of the structure under coherent and incoherent input ground motion. The comparison of the two sets of site-specific ISRS illustrates the effect of the ground motion incoherency on the seismic response at particular building locations. The corresponding 5 percent critical damping standard design ISRS obtained from the standard plant SSI analyses are also included in the figures for comparison.

### 3NN.6 ~~Seismic/Dynamic~~ Lateral Earth Pressure

The standard plant is based on ~~a dynamic~~ lateral earth pressure for a water table location of 1 ft below grade. The site specific observations are as follows:

- The post-construction piezometric head contour map (Figure 2.4-216) indicates that maximum groundwater level elevations in the power block area range from about 83.4 to 86.0 m (270.0 to 284.4 ft)
- The maximum groundwater level elevation in the power block RB/PCCV area of Unit 3 is 282.3 ft or 7.7 ft below the design plant grade elevation of 290 ft NAVD88 (290.86 ft NGVD29).
- The maximum groundwater level elevation near the UHSRS is 284.4 ft or 5.6 ft below the design plant grade elevation.

- Figure 2.4-216 shows that the water levels for the PS/Bs and PSFSVs are below Elevation 282.3 ft. However site specific analysis presented below was performed based on a ground water level of 283 ft.

Lateral earth pressure loads are applied to all below-grade exterior walls that are in contact with embedment soil. The earth pressure loads are applied along the whole embedment height of the walls conservatively assuming that during an earthquake there is no separation between the walls and the soil. Hydrostatic pressures are applied at elevations below the nominal water table elevation specified (7 ft below the nominal plant grade i.e., Elevation 283 ft). Hydrostatic pressure due to ground water is also included in the design/analysis of below-grade exterior walls where expansion joints are present. The site-specific dynamic (and static) lateral pressure distributions are calculated using material properties and data for granular structural fill consistent with Table 2.5-212.

The site-specific static lateral earth pressure on the subgrade exterior walls is computed using methods described in Subsection 2.5.4.10.3. The site-specific static lateral earth pressure is computed considering a Rankine at-rest pressure coefficient and using classic formulas in which static earth pressure, including hydrostatic pressure, increases linearly with depth. Lateral load on below-grade walls due to surcharge loading at the surface is also considered. Lateral load due to surcharge loading is computed by multiplying the Rankine at-rest pressure coefficient for granular structural fill times the surcharge area load. Figure 2.5-254 shows a general static earth pressure diagram for nonyielding walls which includes a lateral load due to surcharge.

The site-specific dynamic lateral pressure on the subgrade exterior walls consists of dynamic earth pressure and hydrodynamic water pressure.

The dynamic earth pressure distributions are determined using Wood's soil pressure distributions presented in Figure 3.5-1 of ASCE 4-98 (Reference 3NN-2). To calculate the dynamic earth pressure using Wood's soil pressure distribution, the weighted average soil unit weight is determined as follows:

$$\gamma = \frac{\gamma_m H_w + (\gamma_s - \gamma_w)(H - H_w)}{H}$$

Where $\gamma_m$ and $\gamma_s$ are the moist and saturated unit weight of the sidewall backfill, respectively; $H_w$ is the ground water table depth (7 ft); $\gamma_w$ is unit weight of water and H is the depth of foundation bottom (38.83 ft).

The hydrodynamic water pressure distributions are calculated using Westergaard formula (Reference 3NN-6):

$$P_W = \frac{7}{8}\frac{a_h}{g}\gamma_w\sqrt{z_w h}$$

Where, $a_h$ is horizontal seismic coefficient (peak ground acceleration) in g's, $Z_w$ and h are the depth of calculation point and total depth of water (38.83-7 =31.83 ft), respectively.

The total seismic (dynamic) lateral load on the wall is determined as the sum of the Wood's dynamic soil pressures and Westergaad hydrodynamic water pressures. SASSI analyses on the embedded model of PS/B were performed to validate the calculated total seismic lateral loads. Four soil cases, denoted as ELB, EBE-EPS, EBE-WPS and EUB representing lower bound, BE for the east PS/B, BE for the west PS/B and upper bound soils, respectively, are considered in analyses (refer to Section 3NN.3.2 for details).

The SASSI soil pressure distributions are developed using following procedure:

1. For each soil cases, the maximum stress responses on the center of the near field (sidewall backfill) soil elements to each of three input motions are extracted from the SASSI runs.

2. For each soil case, co-linear responses (response in the same direction) from the three directions of input motion are combined using SRSS.

3. The SRSS combined stresses normal to the wall are considered as the soil pressures due to the three direction input motions.

Results of the analysis has been presented graphically in Figure 3NN-22. For comparison, Figure 3NN-22 shows site specific seismic lateral load (curve ~~Unit 3~~ NA3 Dynamic Wood~~'s~~+Westergaard), SASSI analysis results (curves EBE-EPS SRSS, EBEWPS SRSS, ELB SRSS and EUB SRSS), and the seismic earth pressure loading used in the standard design (Standard Plant Design curve) for a typical section of NS direction

wall. ~~It demonstrates~~ Figure 3NN-22 also includes plots of the Unit 3 total lateral earth pressure and the standard plant total lateral earth pressure which include both the static and dynamic components. These plots demonstrate that the ~~seismic~~ total (static and dynamic) earth pressure loading used in the standard design envelopes the site-specific loading on the basement wall, which includes the effect of the SRSS soil pressures obtained from site-specific SASSI analyses. ~~It may be noted that, in Figure 3NN-22, at some lower elevations, the SASSI analysis result curve slightly exceeds the Standard Plant Design seismic lateral loading curve. However, this exceedance is compensated for by a larger difference in the upper portion of the wall between SSI analysis results and site-specific loadings as determined using Wood's and Westergaard's pressure distributions.~~

## 3NN.7 References

3NN-1 *An Advanced Computational Software for 3D Dynamic Analysis Including Soil Structure Interaction*, ACS SASSI Version 2.3.0, Ghiocel Predictive Technologies, Inc., September, 2009.

3NN-2 *Seismic Analysis of Safety-Related Nuclear Structures*, American Society of Civil Engineers, ASCE 4-98, Reston, Virginia, 2000.

3NN-3 Abrahamson, N. 2007. "*Effects of Seismic Motion Incoherency Effects*" Palo Alto, CA, EPRI TR-1015111.

3NN-4 Tseng and Lilhanand.1997. "*Soil-Structure Interaction Analysis Incorporating Spatial Incoherence of Ground Motions*", Electric Power Research Institute, Palo Alto, CA, EPRI TR-102631.

3NN-5 US NRC Interim Staff Guidance ISG-01, "*Seismic Issues Associated with High Frequency Ground Motion in Design Certification and Combined License Applications*", ML081330698, May 19, 2008.

3NN-6 Westergaard, H., 1933. "*Water Pressures on Dams during Earthquakes*," Transactions of ASCE, Paper No. 1835, pp418-433.

3NN-7 *Seismic Design Bases of the US-APWR Standard Plant*, MUAP-10001, Revision 1, Mitsubishi Heavy Industries, Ltd., May 2010.

**Figure 3NN-22 US-APWR PS/B Embedded Model Seismic Lateral Soil Pressure Distribution**

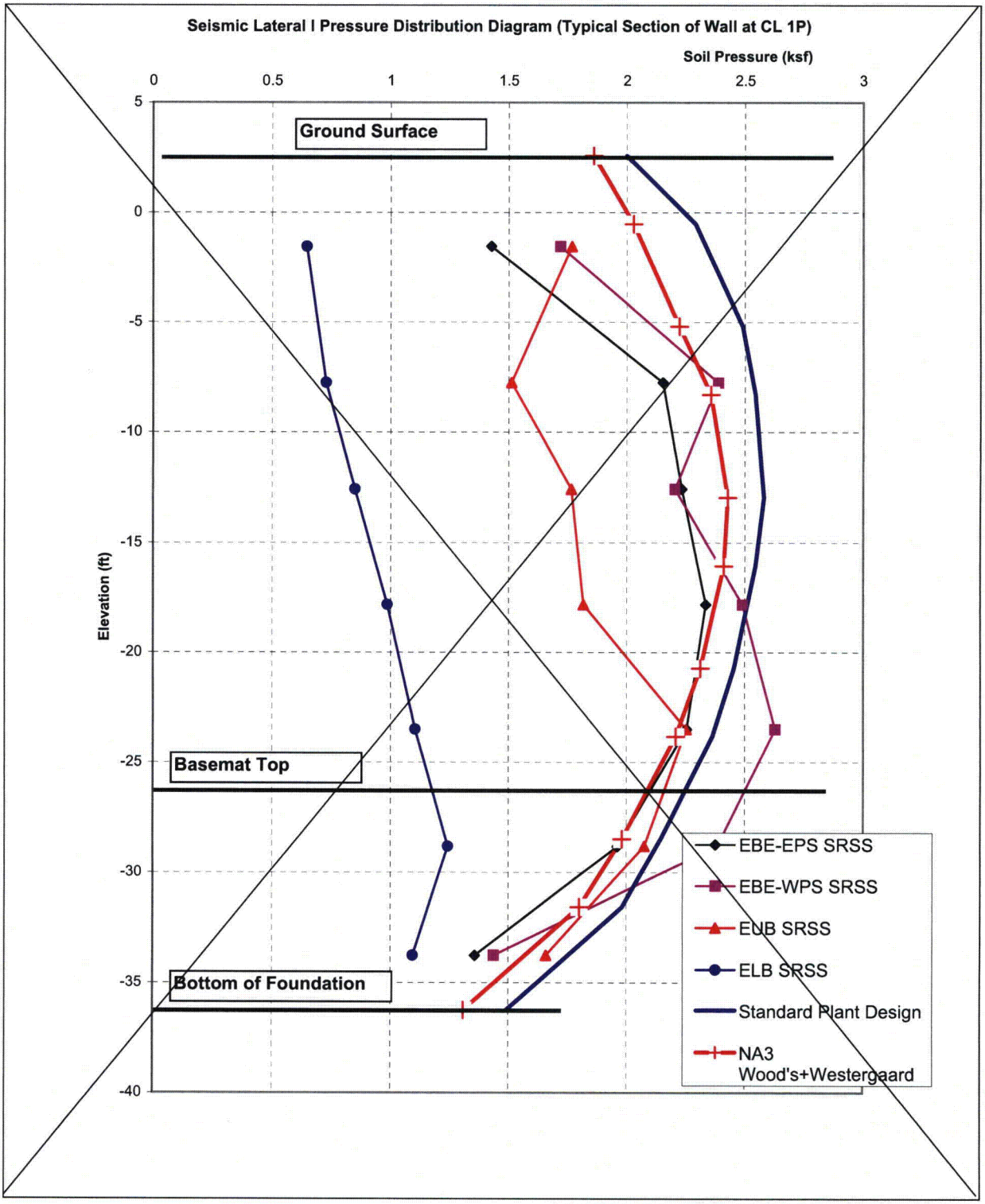
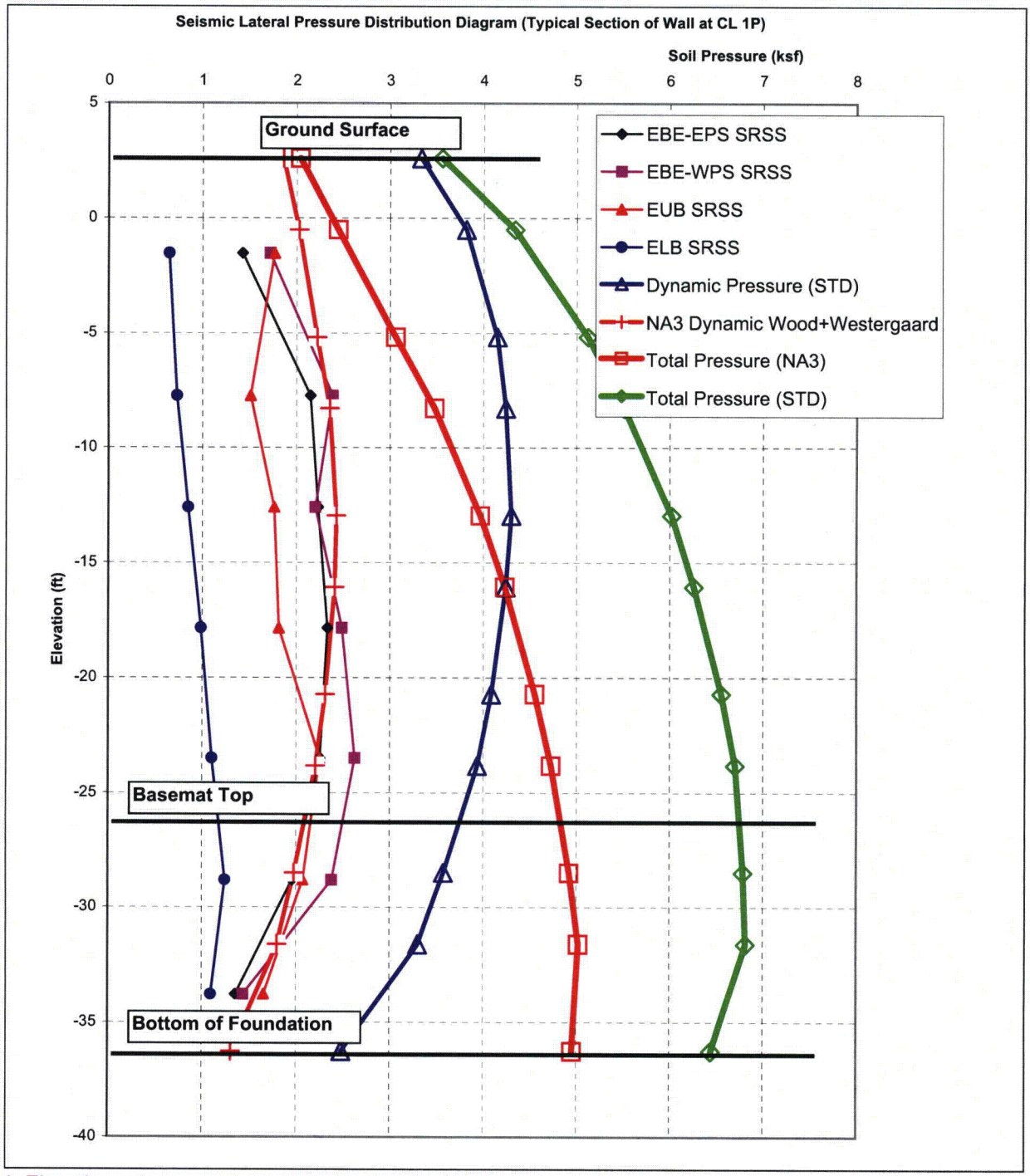


**Figure 3NN-22 US-APWR PS/B Embedded Model Seismic Lateral Soil Pressure Distribution**




1. Elevations shown are based on the standard plant design reference elevations, not Unit 3 values.
2. STD indicates standard plant design.
3. Total pressure plots include static and dynamic pressures.