REVISION SUMMARY

Section	Changes	Reason for Change
Part 2 - Final Safety Analysis Report		
Table 1.8-201	Added 3A, 3C.7.6, and 3G, and deleted 3A.1 from NAPS DEP 3.7.1	Sections added for NAPS DEP 3.7.1
Table 1.8-203	Deleted 3A.2 and Appendix 3C	Changes related to new NAPS DEP 3.7.1 content
Table 1.9-205	Added Section “3.7.1” to CR-6728 where-discussed entries	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
2.5.2.6.2.2	Revised to define soil columns as “full” soil columns and added the V_{S30} for the partial column PBS RS	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1	Added statement to specify the figure that presents SSI input response spectra for FWSC; added citation to section that provides additional discussion of site-dependent, at-grade SSE	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.1	Changed “OBE” to “site-dependent OBE”	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.1.3, 3.7.1.1.4.2.3	Changed “3.7.2.4.1” to “3.7.2.4”	Section 3.7.2.4.1 deleted
3.7.1.1.4.1.1, 3.7.1.1.4.1.2, 3.7.1.1.4.1.3	Deleted “Although not used in the SSI analyses”	No longer correct because the fully embedded cases include the soil properties
3.7.1.1.4.2.1, 3.7.1.1.4.2.2	Revised to state that PBSRS are “full column”	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.1.5.1; Table 3.7.1-220	Added description of and reference to new table; added Table 3.7.1-220, which provides the 1984 M6.2 Morgan Hill earthquake seed time history	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.1.5.1.1	Revised to indicate that ratio values are higher than the binned averages given in NUREG and CR/6728 and added statement to clarify which figure contains partial column PBSRS results and an explanation of PBSRS results	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.

Section	Changes	Reason for Change
Part 2 - Final Safety Analysis Report		
3.7.1.1.5.1.2	Added statement to clarify which figure contains partial column PBSRS results and an explanation of PBSRS results	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.1.6	Aligned title with section content; added reference to discussion of OBE exceedance checks	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.1.2	Added reference to Section 3A.13.2 for site-specific SSI analysis damping values	RAI 03.07.02-14
Figures 3.7.1-295 thru 3.7.1-306	Added figures that compare PBSRS with site-specific SSI input	Address items discussed with the NRC in the April 1, 2015 and April 15, 2015 public meetings.
3.7.2	Added references to 3A sections and ASC SASSI	The new sections include the SSI analyses information, relocated from Section 3.7.2.4. The ACS SASSI is added to 3C.7.6 as a site-specific code used in SSI analyses sensitivity cases.
3.7.2.2	Added references to new 3A sections; deleted “specific”	New 3A sections provide SSI analyses information, relocated from Section 3.7.2.4
3.7.2.4	Added the stiffness properties to the items that are the same as the DCD and changed “basements” to “below-grade portions of the buildings.”	To be consistent with site-specific seismic analyses approach
	Added references to new 3A sections; deleted Section 3.7.2.4.1, Tables 3.7.2-201 through 3.7.2-224, and Figures 3.7.2-201 through 3.7.2-282	SSI analyses updated and moved to Appendix 3A
3.7.2.8, 3.7.2.8.1, 3.7.2.8.2, 3.7.2.8.3, 3.7.2.8.4	RAI 03.07.02-16, Site-Specific SSSI Effects	
3.8	Deleted introduction	To be consistent with site-specific seismic analyses approach
3.8.4	Deleted content; added references to new sections in Appendix 3G; deleted Section 3.8.4 figures	To be consistent with site-specific seismic analyses approach

Section	Changes	Reason for Change
Part 2 - Final Safety Analysis Report		
3.8.5	Deleted content; added references to new sections in Appendix 3G; deleted Section 3.8.5 tables	To be consistent with site-specific seismic analyses approach
3A, 3A.9 thru 3A.19	Added updated SSI analysis description	New Unit 3 site-specific SSI analyses performed.
3C.7.4	Changed LMA from “NAPS CDI” to “NAPS DEP 3.7.1”	Correction
3C.7.4.2	Changed “50 Hz” to “70 Hz”	RAI 03.07.02-10 Rev. 1, to reflect using higher frequencies
3C.7.6	Added ASC SASSI information	Used for certain sensitivity analyses in SSI analyses
3G, 3G.7 thru 3G.10	Added sections to Appendix 3G that address site-specific design details and evaluation results of Seismic Category I structures	New Unit 3 site-specific seismic analyses performed
Part 10 - ITAAC/Tier 1		
2.4.2; Table 2.4.2-1	Added ITAAC for structural fill on the sides of Seismic Category I structures	Address item discussed with the NRC in the April 15, 2015 public meetings
Tables 2.4.15-1, 2.4.16-1, 2.4.17-1 & 2.4.18-1	RAI 03.07.02-16, Site-Specific SSSI Effects	

NAPS SUP 1.8-3

Table 1.8-201 Departures from the Referenced Certified Design

Number	Subject	FSAR Section
NAPS DEP 3.7-1	Seismic Spectra Exceedance	1.3 Table 1.3-4R Table 1.11-201 2.0 Table 2.0-2R Table 2.0-201 Figures 2.0-201 through 2.0-204 3.7 3.7.1 3.7.2 (and associated figures) 3.7.3.13 3.7.4.4 3.7.5 3.8.4 (and associated figures) 3.8.5 (and associated figures) 3A ~~3A.1~~ 3C.7.4 3C.7.5 3C.7.6 3G 4.2.4.2 19.2.3.2.4 Table 19.2-4R (Note 1) 19A.8.3
NAPS DEP 8.1-1	Figure 8.1-1, Sheet 1, Electrical Power Distribution System	Figure 8.1-1R
NAPS DEP 8.1-2	RG 1.204 Compliance	Table 1.9-202 8.1.5.2.4 Table 8.1-1R 8.2.1.2.1

NAPS SUP 1.8-5

Table 1.8-203 Conceptual Design Information (CDI)

Item in DCD	CDI in DCD adopted as actual design	CDI in DCD replaced with actual design	Evaluation	FSAR Section
~~Appendix 3A.2 ESBWR Standard Site Plan~~		~~X~~	~~Site-specific general site plan provided~~	~~Section 3A.2~~ ~~Figure 2.1-201~~
~~Appendix 3C Computer Programs Used in the Design and Analysis of Seismic Category I Structures~~		~~X~~	~~Site-specific computer codes used in site-specific SSI Analysis described in Section 3.7.2~~	~~Appendix 3C.7.4~~ ~~Appendix 3C.7.5~~
9.2.1 Plant Service Water Table 9.2-2 Figure 9.2-1		X	Site-specific system description and design characteristics described	9.2.1 Table 9.2-2R Figure 9.2-1R
9.2.3 Makeup Water System Table 9.2-9		X	Site-specific system description and design characteristics described	9.2.3 Table 9.2-9R
9.2.4 Potable and Sanitary Water Systems		X	Site-specific system description and design characteristics described	9.2.4 Figure 9.2-202 Figure 9.2-203
9.2.10 Station Water System		X	Site-specific system description and design characteristics described	9.2.10 Table 9.2-203 Table 9.2-204 Figure 9.2-204 Figure 9.2-205
9.3.9 Hydrogen Water Chemistry System		X	Site-specific system description and design characteristics described	9.3.9
9.3.11 Zinc Injection System		X	Site-specific system description and design characteristics described	9.3.11

NAPS SUP 1.9-2

Table 1.9-205 NUREG Reports Cited

NUREG No.	Issue Date	Title	Comment/ Section Where Discussed
CR-5613	1990	Features along the Atlantic Seaboard: U.S. Nuclear Regulatory Commission	2.5
CR-5730	1999	Paleoseismology Study Northwest of the New Madrid Seismic Zone: U.S. Nuclear Regulatory Commission	2.5
CR-5750	2/1999	Rates of Initiating Events at U.S. Nuclear Power Plants: 1987 - 1995	19AA.2
CR-6372	1997	Recommendations for Probabilistic Seismic Hazard Analysis – Guidance on Uncertainty and Use of Experts	2.5
CR-6624	11/1999	Recommendations for Revision of Regulatory Guide 1.78	2.2
CR-6697	11/2000	Development of Probabilistic RESRAD 6.0 and RESRAD-BUILD 3.0 Computer Codes	2.4
CR-6728	10/2001	Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines	2.5, 3.7.1
CR-6890	12/2005	Reevaluation of Station Blackout Risk at Nuclear Power Plants	19AA.2

CB columns at the elevations above ($V/H_{WUS,soil}$ / $V/H_{WUS,rock})_{frequency-shifted}$ is simply unity, and the applicable V/H ratio $V/_{HCEUS,soil}$ is just equal to $V/H_{CEUS,rock}$, the middle CEUS hard rock V/H ratio relationship, specified for 0.2g < PGA ≤ 0.5g, given by the NUREG (Reference 2.5-385). See Figure 2.5.2-314.

Upon application of Equation 2.5.2.6-4, the V/H ratios and corresponding $FIRS_V(f)$ for the RB/FB Full Column Outcrop, CB Full Column Outcrop, RB/FB Partial Column Outcrop, and CB Partial Outcrop are tabulated in Table 2.5.2-222, 2.5.2-223, 2.5.2-224, and 2.5.2-225, respectively, and the $FIRS_V(f)$ are shown in Figures 2.5.2-307, 2.5.2-308, 2.5.2-309, and 2.5.2-310, respectively.

2.5.2.6.2.2 Vertical PBSRS for RB/FB and CB

In the development of the $DRS_V(f)$ from Equation 2.5.2.6-8 for the PBSRS for RB/FB and CB, the corresponding $DRS_H(f)$ is given as the horizontal PBSRS in Section 2.5.2.6.1.5.

As presented earlier, the appropriate $V/H_{CEUS,rock}$ for all of the FIRS, as well as the GMRS and PBSRS, is the middle CEUS hard rock V/H ratio relationship, specified for 0.2g < PGA ≤ 0.5g, given by NUREG/CR-6728.

The remaining term of Equation 2.5.2.6-8 is the frequency-shifted version of the ratio $V/H_{WUS,soil}$ /$V/H_{WUS,rock}$, where the numerator and denominator are given by the GA11 V/H model. In the determination of the "soil" and "rock" V/H_{WUS} ratios, the four magnitude-distance pairs of controlling earthquakes (10^{-4} and 10^{-5}, HF and LF), given in Table 2.5.2-218, are considered. For the denominator $V/H_{WUS,rock}$ the V_{S30} is fixed to 5,000 ft/s—equivalent to using V_{S30} fixed to 9,200 ft/s, as discussed earlier. For the numerator $V/H_{WUS,soil}$ the V_{S30} is set to the pair of values corresponding to the full soil columns and finished grade horizon considered for the PBSRS:

Soil Column	Elevation (ft)	V_{S30}
RB/FB Full Column	290 {finished grade}	3,423 ft/s
CB Full Column	290 {finished grade}	2,439 ft/s

Considering the V_{S30} of 2,439 ft/s, Figure 2.5.2-315 shows the four magnitude-distance versions of the ratio $V/H_{WUS,soil}$ /$V/H_{WUS,rock}$, where they are very similar, with the envelope being given by the **M** 6.1 at a

distance of 15 km. Figure 2.5.2-316 shows the four frequency-shifted versions of the ratio $V/H_{WUS,soil}$ /$V/H_{WUS,rock}$.

Shown in Figure 2.5.2-317, the $V/H_{CEUS,rock}$ has been multiplied by each of the four frequency-shifted versions of the ratio $V/H_{WUS,soil}$ / $V/H_{WUS,rock}$ to give V/H ratios considering the V_{S30} of 2,439 ft/s. As a comparison, this figure also shows $V/H_{CEUS,rock}$ and the often-considered V/H ratio from RG 1.60. For frequencies less than about 2.5 Hz, the V/H ratio developed here is slightly greater than that given by RG 1.60. For frequencies above about 3.5 Hz the RG 1.60 V/H ratio is 1.0, while the V/H ratio developed here initially decreases between about 3.5 Hz and 7.5 Hz, then increases to a V/H ratio greater than 1.0 between about 40 and 85 Hz. The high frequency V/H exceedance of 1.0 reflects the character of the $V/H_{CEUS,rock}$ for moderate to high ground motions. The lower value dip in the V/H ratio between about 2 and 20 Hz is addressed further below.

Similar to the process above for determining $V/H_{CEUS,soil}$ ratios for a V_{S30} of 2,439 ft/s (743 m/s) in Figure 2.5.2-317, Figure 2.5.2-318 shows a similar plot of $V/H_{CEUS,soil}$ ratios for the second V_{S30} of 3,423 ft/s (1.043 m/s). This second V/H ratio is equal to or slightly higher than that for the V_{S30} of 2,439 ft/s, except for slightly lower V/H values in the peak range of 35 to 75 Hz.

For the partial soil column analysis of the RB/FB and CB, the V_{S30} values corresponding to the partial column PBSRS are 4,910 ft/sec and 4,070 ft/sec, respectively. As discussed earlier, the GA11 model does not change for V_{S30} values greater than about 5,000 ft/sec and, following the V/H approach, sites with V_{S30} values greater than about 5,000 ft/sec would yield a V/H ratio which is equal to the $V/H_{CEUS,rock}$ ratio. Based on the V_{S30} values for the partial column PBSRS for RB/FB and CB cases being slightly less than 5,000 ft/sec and the additional step in the methodology in which a single V/H ratio is developed from the envelope of all of the soil and rock profiles, the V/H ratios for the partial column PBSRS are considered and ultimately included in the recommended single envelope V/H ratio. The partial column PBSRS spectra are discussed in Sections 3.7.1.1.4.2.1 and 3.7.1.1.4.2.2.

For the purpose a single V/H ratio for application to the PBSRS $DRS_H(f)$, Figure 2.5.2-319 shows an *initial* $V/H_{CEUS,soil}$ ratio for the PBSRS, based on the envelope of all eight $V/H_{CEUS,soil}$ ratios presented above.

Chapter 3 Design of Structures, Components, Equipment, and Systems

3.1 Conformance with NRC General Design Criteria

This section of the referenced DCD is incorporated by reference with no departures or supplements.

3.2 Classification of Structures, Systems and Components

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add the following sentence at the end of Section 3.2.

STD CDI There are no site-specific or RTNSS systems beyond the scope of the DCD.

Table 3.2-1 Classification Summary

Replace the note for System P73 with the following.

STD CDI The site-specific plant design includes the HWCS. See Section 9.3.9 for further details

Replace the note for System P74 with the following.

NAPS CDI The site-specific plant design includes the Zinc Injection System. See Section 9.3.11 for further details.

Replace the note for System U78 with the following.

NAPS CDI The site-specific plant design does not include the cold machine shop.

3.3 Wind and Tornado Loadings

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

NAPS SUP 3.3-1

3.3.2.4 Extreme Hurricane Winds

Section 2.3 defines the site-specific extreme hurricane wind speed in accordance with RG 1.221, “Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants.” The site-specific extreme hurricane wind speed is less than the maximum tornado wind speed listed in Table 2.0-201.

3.4 Water Level (Flood) Design

This section of the referenced DCD is incorporated by reference with no departures or supplements.

3.5 Missile Protection

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.5.1.4 Missiles Generated by Natural Phenomena

Add the following paragraph after the fourth paragraph.

NAPS SUP 3.5-3

Section 2.3 defines the site-specific extreme hurricane winds in accordance with RG 1.221, "Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants." The site-specific extreme hurricane wind speed is less than the maximum tornado wind speed listed in Table 2.0-201. Table 3.5-201 lists the NA3 site hurricane missile spectrum and velocities in accordance with the guidance in RG 1.221. Potential missiles generated by the NA3 site-specific extreme hurricane wind are less severe than the missiles generated by the standard plant design basis tornado.

3.5.1.5 Site Proximity Missiles (Except Aircraft)

Add the following sentence after the first sentence in the first paragraph.

STD SUP 3.5-1

Site-specific missile sources are addressed in Section 2.2.

3.5.1.6 Aircraft Hazards

Add the following at the end of the first paragraph.

STD SUP 3.5-2

Site-specific aircraft hazard analysis and the site-specific critical areas are addressed in Section 2.2.

Table 3.5-201 NA3 Site-Specific Hurricane Missile Spectrum

Missile Type	Missile Velocity[a]
Automobile (4,000 lb)	74 mph horizontal 58 mph vertical
6” Schedule 40 Pipe (287 lb)	55 mph horizontal 58 mph vertical
1” Solid Steel Sphere (0.147 lb)	48 mph horizontal 58 mph vertical

(a) Missile velocities based on RG 1.221 Table 2 for 140 mph site-specific extreme hurricane wind speed

3.6 Protection Against Dynamic Effects Associated with the Postulated Rupture of Piping

This section of the referenced DCD is incorporated by reference with no departures or supplements.

3.7 Seismic Design

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.7.1 Seismic Design Parameters

Replace the last four sentences of this section with following.

NAPS DEP 3.7-1

SSE design ground motion for purposes of seismic design, analysis, and qualification of Unit 3 plant structures, systems, and components, is defined by two sets of ground motion acceleration response spectra:

- the single envelope design ground motion response spectra or Certified Seismic Design Response Spectra (CSDRS) described in Section 3.7.1.1.3 that defines the SSE design motion for seismic design of ESBWR Standard Plant, and
- the site-specific FIRS described in Section 3.7.1.1.4.2, representative of the Unit 3 site specific seismological and geological conditions.

Figures 2.0-201 through 2.0-204 present these 5 percent damped acceleration response spectra that define the site-specific design ground motion as a free-field outcrop motion at the foundation bottom of each Seismic Category I structure. In addition, Figure 3.7.1-285 presents the SSI input response spectra for the FWSC at the average elevation of the bottom of the concrete fill (Elevation 220 ft) as further discussed in Section 3.7.1.1.4.2.3. DCD Figures 2.0-1 and 2.0-2 present the standard design CSDRS.

For each structure and each equipment location within the buildings, in-structure response spectra (ISRS) are developed. The site-specific ISRS that exceed the standard design ISRS, are used in conjunction with the standard design ISRS for seismic design and qualification of equipment and components.

This approach applies to SSCs that are required to withstand SSE loads. Similarly, other SSCs that are specifically required to meet SSE seismic demands are designed, analyzed, and qualified using the process in

Sections 3.7.1 and 3.7.2 for applying the CSDRS and site-specific FIRS. The same approach is applied for the Seismic Category II and Radwaste Building structures.

The Unit 3 plant-shutdown OBE response spectrum limit is established to ensure: 1) that, if not exceeded, plant equipment designed to withstand seismic loads have margin to the design ground motion response spectra; and 2) that the seismic instrumentation system alerts plant operators within four hours of an event (as described in Section 3.7.4.3) and supports decision-making within eight hours of an event as to whether or not to shut down the plant following a seismic event.

The plant shutdown OBE is defined as one-third of the SSE. The following two sets of horizontal and vertical response spectra serve as the reference against which OBE exceedance checks are performed at grade for the purpose of plant shutdown:

(a) One-third of the CSDRS presented in Figures 2.0-201 and 2.0-202 that define the free-field ground motion at the bottom of the RB/FB and CB foundations; and

(b) One-third of the ~~performance-based surface response spectra (PBSRS) presented in Figure 2.5.2-311 that define the Unit 3 site-specific free-field ground motion at grade~~ site-dependent SSE at-grade as described in Section 3.7.1.1.6

Because all safety related SSCs are designed, analyzed, and qualified to meet both the CSDRS and site-specific FIRS, plant shutdown is required if both response spectra in (a) and (b) are exceeded, as described in Section 3.7.4.4. Exceedance of the response spectra (a) and (b) is evaluated independently (i.e., an envelope of the two response spectra is not used). For example, a response spectrum that falls below the envelope of the two response spectra but exceeds (b) at a low frequency and exceeds (a) at a higher frequency is considered as exceeding the OBE, thus requiring shutdown of the plant if other criteria as discussed in Section 3.7.4.4 are also met.

The use of CSDRS in (a) above as the basis for defining the OBE at grade for the purpose of plant shutdown is conservative since it neglects the amplifications of the standard design ground motion as it propagates from the bottom of the RB/FB and CB foundations to the plant grade. The OBE ground motion defined with the criteria above constitutes a single

OBE ground motion for the entire site. See Section 3.7.4.2 for discussion on seismic monitoring instrumentation.

3.7.1.1 Design Ground Motion

Add the following at the end of this section.

NAPS SUP 3.7-7

As shown in Figures 2.0-201 through 2.0-204, the site-specific FIRS calculated for Seismic Category I structures of Unit 3 exceed the CSDRS. Therefore, site-specific SSI analyses of these structures are carried out using the site-specific seismic design parameters described in this section. The site-specific seismic design parameters are developed as described in detail in Sections 3.7.1.1.4 and 3.7.1.1.5. These design parameters include the SSI input strain compatible soil profiles, SSI input response spectra, and SSI input acceleration time histories for the following Seismic Category I structures:

- Reactor Building/Fuel Building
- Control Building
- Fire Water Service Complex

The development of the site-dependent SSE at-grade and the site-dependent OBE at-grade spectra are described in Section 3.7.1.1.6.

3.7.1.1.3 Single Envelope Ground Motion

Add the following at the beginning of this section.

NAPS DEP 3.7-1

This section provides information regarding the single envelope ground response spectra used for seismic design of the ESBWR Standard Plant. The comparisons in Figures 2.0-201 through 2.0-204 show that Unit 3 sites-specific FIRS exceed these single envelope ground response spectra, thus indicating that it is necessary to perform the site-specific SSI analysis presented in ~~Section 3.7.2.4.1~~ Section 3.7.2.4, using the site-specific design parameters described in Sections 3.7.1.1.4 and 3.7.1.1.5. Structures, systems, and components are seismically designed, analyzed, and qualified to multiple response spectra for both generic and site-specific seismic and subgrade conditions, as described in Section 3.7.2, and for equipment seismic qualification as described in Section 3.10. The CSDRS is one of two spectra used for ensuring that SSCs meet the requirements for seismic design adequacy.

DCD Table 3.7-2 provides the single envelope design ground motion at foundation level for the structures that are designed to meet Seismic Category I requirements and which is referred to as the CSDRS for the ESBWR Standard Plant. The site-specific SSI analyses indicate that additional ground motion response spectra apply, as described in Section 3.7.2.

The information below relates to the CSDRS used for seismic design of the ESBWR Standard Plant. These design ground motion response spectra are used in conjunction with the site-specific ground motion response spectra, as described in Section 3.7.1.1.4.2.

NAPS SUP 3.7-1

3.7.1.1.4 Site-Specific Design Ground Motion Response Spectra

3.7.1.1.4.1 SSI Strain-Compatible Soil Profiles

Best estimate (BE), lower bound (LB), and upper bound (UB) soil properties are calculated consistent with the FIRS for each Seismic Category I structure from their corresponding probabilistic full column site response analysis results and are presented in the following sections. The details of the site response analysis are described in Section 2.5.2.5.

3.7.1.1.4.1.1 SSI Strain-Compatible Soil Profiles for the RB/FB

From the probabilistic full column site response analyses of the RB/FB soil column set (presented in Section 2.5.2.5), a set of 60 strain-compatible soil properties is obtained for each of the 4 input rock cases (10^{-4} and 10^{-5} annual-frequency-of-exceedance (1E-4 and 1E-5 hazard level) low frequency (LF) and high frequency (HF) as described in Section 2.5.2.4). The log-mean (μ_{ln}) and log-standard deviation (log-SD, σ_{ln}) for each of the 4 sets of shear wave velocity (V_s) and damping ratios are calculated. These values are used to establish the log-mean and log-SD of the strain compatible properties that are consistent to FIRS motions using A_R and *DF* factors (described in Section 2.5.2.6), calculated for the acceleration response spectra (ARS) at the ground surface.

The simulated (randomized) profiles described in Section 2.5.2.5 include a variation in the thickness of different strata (a stratum is defined as a thickness of rock or soil having the same initial dynamic and static properties). For deterministic SSI analysis, the strain-compatible soil profiles are obtained from the strain-compatible soil properties of the simulated profiles using the data for each soil layer type within the profile.

The stratum log-mean and log-standard deviation strain compatible properties are used with the BE thicknesses for each stratum to obtain the LB, BE, and UB SSI input strain-compatible soil profiles.

The log-mean values of the FIRS-consistent strain-compatible damping ratios are used to determine the BE damping ratio profile. The LB and UB values for the strain compatible damping ratios are calculated as plus/minus one log-standard deviation from the log-mean values by applying Equations 3.7.1.1-1 and 3.7.1.1-2.

Lower bound and upper bound V_s corresponding to FIRS are calculated by applying Equations 3.7.1.1-1 and 3.7.1.1-2 in conjunction with the FIRS-consistent log-mean and log-standard deviation as described above. In these equations, *M* refers to damping or shear wave velocity, μ_{ln} refers to the log-mean of FIRS-consistent shear wave velocity or damping ratio and σ_{ln} refers to the corresponding log-standard deviations. However, lower bound shear wave velocity profiles are calculated as the minimum resulting from Equation 3.7.1.1-1 and $(V_S)_{FIRS}/(\sqrt{1.5})$, and upper bound shear wave velocity profiles are calculated as the maximum resulting from Equation 3.7.1.1-2 and $(V_S)_{FIRS} \times \sqrt{1.5}$ to satisfy the minimum variation requirements of ASCE 4-98, where $(V_S)_{FIRS}$ is the best estimate strain compatible shear wave velocity corresponding to the FIRS level of motion.

$$M_{LB} = \exp(\ln(\mu_{ln}) - \sigma_{ln}) \quad (3.7.1.1\text{-}1)$$

$$M_{UB} = \exp(\ln(\mu_{ln}) + \sigma_{ln}) \quad (3.7.1.1\text{-}2)$$

Primary (P- or compression) wave velocity V_P is calculated using Equation 3.7.1.1-3 (Reference 3.7-201), where Poisson's ratio ν values, at different depths, are provided in Section 2.5.4.

$$V_P = V_S\sqrt{\frac{2-2\nu}{1-2\nu}} \quad (3.7.1.1\text{-}3)$$

For soil layers below water table, a minimum P-wave velocity of 4800 ft/sec is maintained and, as needed, the Poisson's ratio is adjusted to obtain the minimum P-wave velocity. The maximum value of Poisson's ratio used is 0.48 to avoid numerical instability.

Figure 3.7.1-201 presents the SSI shear wave velocity profiles for the RB/FB. SSI damping and P-wave velocity profiles for this structure are presented in Figures 3.7.1-202 and 3.7.1-203, respectively. The depth of 0 ft in Figures 3.7.1-201, 3.7.1-202, and 3.7.1-203 corresponds to the top

of the soil column at Elevation 290 ft. The lower shear wave and P-wave velocities are used in conjunction with the higher damping values to form the LB profile, and vice versa for the UB profile. Table 3.7.1-201 presents the digital values for the RB/FB SSI input strain-compatible soil profiles. The provided soil profiles correspond to the fully embedded SSI analysis of the RB/FB. The top 17 ft of this profile (the top 7 layers in Table 3.7.1-201) correspond to saprolite and are removed in the partially embedded SSI analysis of the RB/FB.

As described in Section 2.5.4, adjacent to the structure, the in-situ saprolite is replaced by structural fill and Zone III rock is replaced by concrete fill. The strain-compatible properties for the structural fill for the RB/FB are obtained following the steps described above for the in-situ profile and applied to a companion fill profile for the RB/FB. The companion RB/FB profile is identical to the in-situ profile except that randomized saprolite and Zone III rock properties are replaced with randomized structural fill and concrete fill properties, respectively.

~~Although not used in the SSI analyses, the~~ The lower bound and upper bound shear wave velocities, P-wave velocities, and damping ratios for the structural fill compatible with FIRS are calculated following the methodology described above and presented in Table 3.7.1-202. The same table provides the LB, BE, and UB values for the concrete fill to be used in the SSI analysis model. The concrete fill is considered as linear material for the purpose of site response and SSI analyses.

3.7.1.1.4.1.2 **SSI Strain-Compatible Soil Profiles for the CB**

The SSI strain-compatible soil profiles for the CB are calculated from the probabilistic full column site response analyses of the CB soil column set (presented in Section 2.5.2.5) following the same approach as described above for the RB/FB structure.

Figure 3.7.1-204 presents the SSI shear wave velocity profiles for the CB. SSI damping and P-wave velocity profiles for this structure are presented in Figures 3.7.1-205 and 3.7.1-206, respectively. The depth of 0 ft in Figures 3.7.1-204, 3.7.1-205, and 3.7.1-206 corresponds to the top of the soil column at Elevation 290 ft. The lower shear wave and P-wave velocities are used in conjunction with the higher damping values to form the LB profile, and vice versa for the UB profile. Table 3.7.1-203 presents the digital values for the CB SSI input strain-compatible soil profiles. The provided soil profiles correspond to the fully embedded SSI analysis of

the CB. The top 25 ft of this profile (the top 10 layers in Table 3.7.1-203) correspond to saprolite and are removed in the partially embedded SSI analysis of the CB.

As described in Section 2.5.4, adjacent to the structure, the in-situ saprolite is replaced by structural fill and Zone III rock is replaced by concrete fill. Table 3.7.1-204 provides the LB, BE, and UB values for the concrete fill to be used in the SSI analysis model. The concrete fill is considered as linear material for the purpose of site response and SSI analyses. ~~Although not used in the SSI analyses, the~~ The strain compatible properties for the structural fill for the CB are similarly obtained as those for the RB/FB and presented in Table 3.7.1-204.

3.7.1.1.4.1.3 **SSI Strain-Compatible Soil Profiles for the FWSC**

The SSI strain-compatible soil profiles for the FWSC are calculated from the probabilistic full column site response analyses of the FWSC soil column set (presented in Section 2.5.2.5) following the same approach as described above for the RB/FB structure.

Figure 3.7.1-207 presents the SSI shear wave velocity profiles for the FWSC. SSI damping and P-wave velocity profiles for this structure are presented in Figures 3.7.1-208 and 3.7.1-209, respectively. The lower shear wave and P-wave velocities are used in conjunction with the higher damping values to form the LB profile, and vice versa for the UB profile. Table 3.7.1-205 presents the digital values for the FWSC SSI input strain-compatible soil profiles.

As described in Section 2.5.4, for the FWSC, the foundation of the structure is supported by concrete fill situated on Zone III-IV rock. Adjacent to the structure, the in-situ saprolite is replaced by structural fill and Zone III rock is replaced by concrete fill. Table 3.7.1-206 provides the LB, BE, and UB values for the concrete fill to be used in the SSI analysis model. The concrete fill is considered as linear material for the purpose of site response and SSI analyses. ~~Although not used in the SSI analyses, the~~ The strain compatible properties for the structural fill for the FWSC are similarly obtained as those for the RB/FB and presented in Table 3.7.1-206.

3.7.1.1.4.2 **Site-Specific SSI Input Response Spectra**

The FIRS for all Seismic Category I structures are presented in Section 2.5.2.6. For each Seismic Category I structure, the site-specific SSI input response spectra are obtained from its corresponding FIRS by

ensuring that the requirements of ISG-17 (Reference 3.7-202) with regards to the adequacy of the input motion for embedded SSI analyses are met. This verification is referred to as the NEI check in reference to the Nuclear Energy Institute (NEI) white paper (Reference 3.7-203). Once the NEI check is done for a given FIRS and any necessary adjustments are made, the resulting spectra is termed the "SSI input response spectra."

In addition, the site-specific SSI input response spectra are augmented by the broadband horizontal and vertical response spectra defined in RG 1.60 anchored at 0.1g to satisfy the minimum design ground motion requirements of 10 CFR 50, Appendix S. The resulting ARS are labeled as "Final SSI Input Response Spectra." The development of these spectra for all Seismic Category I structures is described in the following sections.

3.7.1.1.4.2.1 SSI Input Response Spectra for the RB/FB

The site-specific SSI input response spectra are calculated for SSI analysis of the RB/FB structure as partially embedded (only considering embedment in rock) and as fully embedded. The corresponding partial column outcrop FIRS and full column outcrop FIRS for this structure as well as the full column performance-based surface response spectra (PBSRS) are presented in Section 2.5.2.6.

The NEI check is conducted for the RB/FB by convolving the full column and partial column outcrop FIRS (from the foundation level, Elevation 224 ft NAVD88) through their corresponding LB, BE, and UB strain compatible soil profiles of the RB/FB (Section 3.7.1.1.4.1), and comparing the envelope of the resulting top-of-the-column ARS with the corresponding full column and partial column PBSRS. The partial column PBSRS are calculated consistent with the partially embedded SSI analyses, at the top of the partial columns (after removal of the saprolite layers) using the same methodology as described for the full column PBSRS in Section 2.5.2.6. The horizontal FIRS are convolved to the top of the soil column using vertically propagating shear waves and the vertical FIRS are convolved to the surface through vertically propagating P-waves. Shear wave damping is used for both horizontal and vertical analyses. The analyses are carried out linearly with no further degradation of the strain-compatible profiles. The horizontal and vertical 5 percent damped ARS at the top of the soil columns corresponding to each SSI input soil profile are determined and the horizontal and vertical

envelope resulting from the LB, BE, and UB soil columns for the structure is compared to the horizontal and vertical PBSRS.

For each direction (horizontal or vertical) and each embedment configuration (fully or partially embedded FIRS), if the envelope of the LB, BE, and UB ARS (at the top of the SSI input soil column) does not envelope the corresponding PBSRS, the FIRS must be adjusted. The frequency dependent adjustment factor is either unity or the ratio of PBSRS to the envelope of LB, BE, and UB ARS, whichever is greater. In order to satisfy the NEI check, this adjustment factor is applied to the computed FIRS at the foundation level to yield the full column and partial column horizontal and vertical SSI input response spectra for the RB/FB.

Figures 3.7.1-210 and 3.7.1-211 present the envelope of the ground surface ARS for the horizontal and vertical full column FIRS, respectively. Figures 3.7.1-212 and 3.7.1-213 present the horizontal and vertical envelope ARS at surface as well as their corresponding FIRS and PBSRS. For the RB/FB full column FIRS, the adjustment occurs for the horizontal FIRS below 6.9 Hz with the largest adjustment factor being 1.27. For the vertical FIRS, the adjustment is much more significant, especially between frequencies of 1 Hz and 20 Hz with the maximum adjustment factor being 1.79. The adjusted full column FIRS for RB/FB are referred to as the full column SSI input response spectra for RB/FB and are also presented in Figures 3.7.1-212 and 3.7.1-213.

The NEI check for the partial column FIRS for RB/FB are carried out in a similar manner. The corresponding figures are provided in Figures 3.7.1-214 through 3.7.1-217. The surface ARS and PBSRS corresponding to the partial columns are calculated consistent with the partially embedded SSI analyses, at the top of the partial soil columns after removal of the saprolite layers. The necessary adjustment factors for RB/FB partial column FIRS are less than 1.01 for both horizontal and vertical directions. The adjusted partial column FIRS are referred to as the partial column SSI input response spectra for RB/FB and are presented in Figures 3.7.1-216 and 3.7.1-217.

For the full column analyses (applicable to fully embedded SSI analyses), the final horizontal and vertical SSI input response spectra are calculated as the envelope of the full column SSI input response spectra and the minimum required response spectra which are adopted from the horizontal and vertical broadband spectra defined in RG 1.60 and anchored at 0.1g. Similarly, for the partial soil column analyses

(applicable to SSI analyses of the structures as partially embedded), the final horizontal and vertical SSI input motions are calculated as the envelope of the partial column SSI input response spectra and the minimum required response spectra. These final SSI input response spectra are presented in Figures 3.7.1-218 through 3.7.1-220 and tabulated in Table 3.7.1-207. These spectra are used as target ARS for development of SSI input time histories in subsequent analyses.

3.7.1.1.4.2.2 **SSI Input Response Spectra for the CB**

The site-specific SSI input response spectra are calculated for SSI analysis of the CB structure as partially embedded (only considering embedment in rock) and as fully embedded. The corresponding partial column outcrop FIRS and full column outcrop FIRS for this structure as well as the full column PBSRS are presented in Section 2.5.2.6.

The SSI input response spectra for the CB are obtained after adjusting the FIRS as necessary for the NEI check following the same approach as described for RB/FB. For the CB, Figures 3.7.1-221 and 3.7.1-222 present the envelope of the ground surface ARS for the horizontal and vertical full column FIRS, respectively. Figures 3.7.1-223 and 3.7.1-224 present the horizontal and vertical envelope ARS at surface as well as their corresponding FIRS and PBSRS. Where the PBSRS exceed the envelope of surface ARS, the FIRS is adjusted (upward adjustment only, i.e., adjustment factor is always larger than one) by the ratio of the PBSRS to the envelope of surface ARS at each frequency. For the CB full column FIRS, the adjustment occurs for the horizontal FIRS below 0.4 Hz with the largest adjustment factor being 1.05. For the vertical full column FIRS, the adjustment is much more significant, especially between frequencies of 2 Hz and 15 Hz with the maximum adjustment factor being 1.39. The adjusted full column FIRS for CB are referred to as the full column SSI input response spectra for CB and are presented in Figures 3.7.1-223 and 3.7.1-224.

The NEI check for the partial column FIRS for the CB is carried out in a similar manner. The corresponding figures are provided in Figures 3.7.1-225 through 3.7.1-228. The surface ARS and PBSRS corresponding to the partial columns are calculated consistent with the partially embedded SSI analyses, at the top of the partial soil columns after removal of the saprolite layers. The necessary adjustment factors for the CB partial column FIRS are less than 1.07 for horizontal and less than 1.16 for vertical directions. The adjusted partial column FIRS are

referred to as the partial column SSI input response spectra for CB and are presented in Figures 3.7.1-227 and 3.7.1-228.

For the full column analyses (applicable to fully embedded SSI analyses), the final horizontal and vertical SSI input response spectra are calculated as the envelope of the full column SSI input response spectra and the minimum required response spectra which are adopted from the horizontal and vertical broadband spectra defined in RG 1.60 and anchored at 0.1g. Similarly, for the partial soil column analyses (applicable to SSI analyses of the structures as partially embedded), the final horizontal and vertical SSI input motions are calculated as the envelope of the partial column SSI input response spectra and the minimum required response spectra. These final SSI input response spectra are presented in Figures 3.7.1-229 through 3.7.1-231 and tabulated in Table 3.7.1-208. These spectra are used as target ARS for development of SSI input time histories in subsequent analyses.

3.7.1.1.4.2.3 **SSI Input Response Spectra for the FWSC**

Two sets of site-specific SSI input response spectra with control motion, defined at the bottom of the FWSC foundation (Elevation 282 ft NAVD88) and at the average elevation of the bottom of the concrete fill (Elevation 220 ft NAVD88), are calculated for SSI analysis of the FWSC. The geologic outcrop FIRS for this structure is calculated at Elevation 282 ft corresponding to the bottom of the foundation and top of the considered FWSC soil column, as presented in Section 2.5.2.6. The FWSC geologic outcrop FIRS also represent the PBSRS for the FWSC soil column. Additional NEI check for the FWSC SSI input response spectra is not warranted because the SSI results from the application of the SSI input motions at the top of the soil column and Elevation 220 ft are enveloped by the design basis for this structure as described in ~~Section 3.7.2.4.1~~ Section 3.7.2.4.

The final horizontal and vertical SSI input response spectra at Elevation 282 ft for FWSC are calculated as the envelope of the geologic outcrop FIRS and the minimum required response spectra which are adopted from the horizontal and vertical broadband spectra defined in RG 1.60 and anchored at 0.1g. The final SSI input response spectra at Elevation 282 ft are presented in Figures 3.7.1-232 through 3.7.1-234 and tabulated in Table 3.7.1-209. Similarly, the final horizontal and vertical SSI input response spectra at Elevation 220 ft are calculated as the envelope of the design response spectra (DRS) at Elevation 220 ft

and the minimum required response spectra. The DRS at Elevation 220 ft are calculated consistent with the FIRS for FWSC using the same methodology as described in Sections 2.5.2.5 and 2.5.2.6. The final SSI input response spectra at Elevation 220 ft are presented in Figures 3.7.1-283 through 3.7.1-285. These spectra are used as target ARS for development of SSI input time histories in subsequent analyses.

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3.7.1.1.5 Site-Specific Design Ground Motion Time History

3.7.1.1.5.1 SSI Input Acceleration Time Histories

Corresponding to each set of horizontal and vertical final SSI input response spectra, described in Section 3.7.1.1.4.2, a three component set (two horizontal and one vertical) of spectrum compatible acceleration time histories is developed for use as input time histories for SSI analysis. The starting seed time histories are selected from the database of acceleration time histories in NUREG/CR-6728 (Reference 3.7-204). The candidate time histories were considered from the CEUS rock database bin with magnitudes between moment magnitude (**M**) 6 and **M**7 and distances between 10 km and 50 km. This magnitude-distance bin was selected based on the high frequency deaggregation of the PSHA having mean magnitude and distance values of **M**5.9 and 22 km for the 10^{-4} hazard level, and **M**6.1 and 15 km for the 10^{-5} hazard level (Section 2.5.2.4). For the low frequency hazard deaggregation, the results are a magnitude of **M**7.1 and a distance of 340 km for the 10^{-4} and **M**6.4 and a distance of 21 km for the 10^{-5} hazard levels (Section 2.5.2.4). Based on the large distance associated with the low frequency 10^{-4} controlling event, the selected seed input acceleration time history for the spectral matching procedure was governed by the high frequency controlling events.

In selecting a candidate acceleration time history set from the applicable magnitude-distance bin from NUREG/CR-6728, the following aspects of a given time history set were considered:

- Similarity between the spectral shape of the candidate acceleration time history and the target spectrum
- Total time history duration of at least 20 seconds
- Zero-lag cross correlation coefficient between any two components of acceleration time histories should be less than 0.16

- Appropriate magnitude and distance values relative to the controlling event values
- Non-stationary phasing consistent with seismological principals.
- Uniform normalized Arias intensity curves

Following these selection guidelines, the strong ground motion time history from the 1984 **M**6.2 Morgan Hill earthquake recorded at the station Gilroy–Gavilan College was selected as the seed input time history set for the spectral matching presented here. This selected time history from the CEUS magnitude-distance database bin of NUREG/CR-6728 is based on the original empirical recording from the Morgan Hill earthquake at the Gilroy–Gavilan College station with the additional modification of the empirical time history to adjust for more CEUS hard rock conditions. The details for this selected seed time history, including the filter corners, peak ground motion parameters, associated ratios and duration values from the NUREG/CR-6728 database, are listed in Table 3.7.1-220.

3.7.1.1.5.1.1 **SSI Input Acceleration Time Histories for the RB/FB**

Two sets of three statistically independent acceleration time histories of motions (i.e., two horizontal and one vertical component) are developed for the full column and partial column final SSI input motion spectra. These time histories are modified to be spectrum compatible following Option 1, Approach 2 of SRP 3.7.1. Additionally, the power spectrum density (PSD) of the spectrum matched time histories are computed to allow for the conclusion that these spectrum compatible time histories do not contain any significant gaps in energy over the SRP 3.7.1 defined frequency range.

The input seed time histories are modified to be spectrum compatible using the computer program RSPM. The baseline correction program BLINE, a component program of RSPM, is also used in the process after each iteration of RSPM.

For each time history, the average ratio between the acceleration time history response spectrum and the corresponding target acceleration response spectrum (both at a spectral damping of 5 percent) was greater than 1.0. In addition, the spectral matching criteria given in SRP 3.7.1 for Option 1, Approach 2 were satisfied in each spectral matching case.

For the RB/FB partial column spectrally matched time histories, the comparisons between the scaled spectrum compatible acceleration response spectra and the target spectra and boundary range are plotted in Figures 3.7.1-235 through 3.7.1-237 for the first horizontal direction (H1), second horizontal direction (H2) and the vertical direction (UP), respectively. Similar plots for the RB/FB full column spectrally matched time histories are presented in Figures 3.7.1-238 through 3.7.1-240.

The zero-lag cross-correlation for each three component sets of spectrum compatible acceleration time histories are computed to verify the acceptability of the acceleration time histories. The zero-lag cross-correlations for the partial column and full column RB/FB cases are listed in Table 3.7.1-210. These computed values are all less than the required minimum value of 0.16.

~~In addition to the cross-correlation values, the peak ground motion parameters and associated ratios are listed in Table 3.7.1-211. Based on the target spectra used in the spectral matching procedure being a composite of both the high frequency and low frequency cases (i.e., the deaggregation values are bi-modal), the resulting PGV/PGA and PGA*PGD/PGV^2 ratios do not fall within the bin values reported in NUREG/CR-6728. The PGA, PGV, and PGD refer to the peak ground acceleration, peak ground velocity, and peak ground displacement, respectively. This observed deviation from the reported bin values is caused by the relatively large PGA value from the high frequency case (i.e., small magnitude event at relative close distances) compared to the intermediate and longer spectral period range PGV and PGD which is controlled by the low frequency case (i.e., large magnitude event at a relatively large distance). Given this understanding of the composite nature of the target spectra used in the spectral matching procedure, the peak ground motion parameter values and associated ratios are acceptable.~~

~~The total duration of the time histories is approximately 30 seconds, which is greater than the required minimum of 20 seconds. In addition, the Arias durations (5-75% Duration) given in Table 3.7.1-211, are longer than the minimum value of 6 seconds defined in SRP 3.7.1.]~~

In addition to the cross-correlation values, the peak ground motion parameters and associated ratios are listed in Table 3.7.1-211. PGA, PGV, and PGD refer to the peak ground acceleration, peak ground velocity, and peak ground displacement, respectively. Overall, these peak

values (i.e., PGA, PGV, and PGD) are consistent with the averaged bin values provided in NUREG/CR-6728. The target spectra used in the spectral matching procedure are a composite of both the high frequency and low frequency cases (i.e., the deaggregation values are bi-modal), which results in PGV/PGA and PGA*PGD/PGV^2 ratios that deviate from the average bin values reported in NUREG/CR-6728. This observed deviation from the reported bin values is caused by the relatively large PGA value from the high frequency case (i.e., small magnitude event at a relatively close distance) compared to the intermediate and longer spectral period range PGV and PGD which are partially controlled by the low frequency case (i.e., large magnitude event at a relatively large distance). This leads to a slightly lower PGV/PGA ratio and a larger PGA*PGD/PGV^2 ratio. Given this understanding of the composite nature of the target spectra used in the spectral matching procedure, the peak ground motion parameter values and associated ratios in Table 3.7.1-211 are acceptable.

The total duration of the time histories is approximately 30 seconds, which is greater than the required minimum of 20 seconds. In addition, the Arias durations (5-75 percent Duration) given in Table 3.7.1-211, are longer than the minimum value of 6 seconds defined in SRP 3.7.1 and fall within the bin value ranges given in NUREG/CR-6728.

The acceleration, velocity, and displacement time histories for the H1, H2, and UP spectrally matched to the RB/FB partial column final SSI input response spectra are respectively provided in Figures 3.7.1-241 through 3.7.1-243. Similar figures for the RB/FB full column case are presented in Figures 3.7.1-244 through 3.7.1-246.

The one-sided PSD is computed following SRP 3.7.1, Appendix B, Equation 1, based on a defined time window, Fourier Transform and associated duration, TD. This duration represents the duration of near maximum and nearly stationary power of an acceleration time history and should be consistent with the time window used in the calculation of the Fourier Transform. Several different quantitative definitions of strong motion duration may be defined, while the guidance provided in NUREG/CR-5347, Appendix B, qualitatively states that the duration should be the time range over which the power is nearly constant and near maximum. This can variously be estimated by analysis of the normalized Arias Intensity of a given acceleration time history, while a

common estimate of duration is based on the 5-75 percent duration window for the normalized Arias Intensity.

For each spectrum compatible time history, two PSDs were computed based on different time windows for the Fourier Transform and the associated TD duration values. For the first case, the full time history (i.e., total time history length of 29.98 seconds) was used for the Fourier Transform and the TD was estimated from the extrapolation of the computed 5–75 percent values to 0–100 percent values. This extrapolation was implemented to account for the fact that the Fourier Transform is based on the full time window of the given acceleration time history and the expectation that the stationary characteristics of the given time history would not be over the entire time window length, but rather an extrapolated time window based on the 5–75 percent duration value. The use of the full time history for the calculation of the Fourier Transform and PSD function can be considered to be more consistent with the application of the time histories in the SSI analyses which use the full time history in estimating the full frequency content of the given time history.

The second PSD is computed based on taking a windowed section of the given time history for the Fourier Transform. This window duration is based on the 5–75 percent normalized Arias Intensity duration. Prior to the calculation of the Fourier Transform, a cosine taper was applied immediately prior to the start and immediately following the end of this windowed time history and for the PSD calculation the TD value for the denominator was taken as the 5–75 percent duration value.

For each PSD the prescribed smoothing window of ± 20 percent was used. The PSD results for the RB/FB partial column are provided in Figures 3.7.1-268 through 3.7.1-270. The PSD results for the RB/FB full column are provided in Figures 3.7.1-271 through 3.7.1-273. As expected, the amplitude of the computed PSDs varies as a function of spectral frequency. It can also be observed that the selection of the time window and associated TD value can produce different characteristics of the PSDs as a function of frequency and also in absolute amplitude. The variation between these PSDs is not significant in most cases, indicating a certain degree of robustness in the two approaches.

For the H1 component, the largest difference between the PSDs computed using the full window length and the 5–75 percent truncated window length is in the frequency range of about 25–50 Hz. For the full

record length cases, the PSD is nearly constant over the frequency range of about 30–40 Hz with a slight dip in the PSD around 25 Hz. In contrast, the PSDs computed from the truncated 5–75 percent window are smoothly attenuating over the frequency range of 25–50 Hz. These observations indicate that the full window time history contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the H2 component, the same observations in the high frequency range of 25–50 Hz are noted. For the truncated 5–75 percent windowed cases, the PSD is again more smoothly varying and is approximately constant or slightly less for frequencies between about 25-40 Hz. These observations also indicate that the full window time history for the H2 component contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the vertical component, the same comparison plots of the computed PSD using the two different time windows are presented. However, unlike the two horizontal cases, the PSDs from the truncated 5–75 percent windowed cases do not show a reduction in the relative amplitude of the PSD in the 30–40 Hz frequency range. In general the same relative increase in amplitude around the 30 Hz frequency and relative decrease in the 20–25 Hz range is observed for both time window PSD calculations. This variation in the PSD is similar in amplitude to the horizontal components using the full time window and could indicate that for the vertical component the relative increase in energy in the 30–40 Hz range is present within the truncated 5–75 percent duration time window. For the truncated 5–75 percent windowed cases, additional minima are observed for frequencies less than about 1 Hz and around 2 Hz. These observations of the relative minima in the vertical component PSDs can be interpreted as an indication of the non-stationarity of the time history because these relative minima are not observed for longer duration windows based on the 5–85 percent and 5–95 percent normalized Arias Intensity values.

The input time histories needed for SSI analysis of the RB/FB with embedded foundation (for both partially embedded and fully embedded cases) are in-column (within) motions corresponding to each of the SSI

strain-compatible soil profiles. As such, for each case (partially embedded or fully embedded), each of the outcrop acceleration time histories (H1, H2, and UP), is used as input at the foundation level of the RB/FB to a SHAKE2000 soil column model of the SSI strain compatible soil profiles (LB, BE and UB), and their corresponding in-column time histories are obtained at the same horizon. The horizontal acceleration time histories are applied using strain compatible shear wave velocities and the vertical acceleration time history is applied using corresponding P-wave velocities. The strain-compatible shear wave damping is used for both vertical and horizontal analyses. The analyses are carried out linearly with no further degradation of the strain-compatible shear modulus and damping profiles. These analyses result in a total of 18 in-column SSI input time histories corresponding to the three SSI strain compatible profiles (LB, BE, and UB), the three time history components (H1, H2, and UP), and two embedment cases (full column or partial column). These time histories are used as input in the subsequent SSI analyses of the structure.

As described in Section 3.7.1.1.4.2.1, the NEI check for the full column and partial column SSI analyses of the RB/FB are performed directly for the response spectra using Random Vibration Theory (RVT). Because the SSI analyses are performed using the spectrally matched time-histories, a confirmatory NEI check is performed using the surface ARS corresponding to the spectrally matched time-histories for the partially embedded and fully embedded SSI analyses of the building.

For the purpose of the confirmatory NEI check for the RB/FB, the PBSRS corresponding to the RB/FB soil column is used as opposed to the site PBSRS, which is the envelope of the RB/FB and CB DRS at the top of their respective soil columns. The horizontal and vertical components of the full column and partial column RB/FB PBSRS are calculated using the same methodology as described for the full column PBSRS in Section 2.5.2.6, except that only the DRS corresponding to the appropriate RB/FB soil column is used. The PBSRS are smooth spectra because they are obtained from an RVT based analysis. Therefore, the comparison is made between the PBSRS and the ±20 percent smoothed envelope of the surface ARS obtained for each direction of the input time-history in each of the LB, BE, and UB soil profiles. The comparison of the smoothed surface ARS obtained from the SSI input time-histories and the RB/FB PBSRS for each direction, and each considered

embedment condition, are shown in Figures 3.7.1-295 through 3.7.1-300. Within the frequency range used for the spectral matching (0.2 to 100 Hz), there are some instances where the PBSRS slightly exceeds the enveloped ARS. These exceedances are limited to 2 percent or less and are considered negligible. These comparisons confirm that the ARS corresponding to the SSI input time-histories calculated at the surface of the SSI soil profiles for the RB/FB do not exhibit any gaps in the site response frequencies at the surface of the SSI input soil columns and, except for negligible exceedance, bound the PBSRS for the building, thus satisfying the intent of the NEI check requirements.

3.7.1.1.5.1.2 **SSI Input Acceleration Time Histories for the CB**

Two sets of three statistically independent acceleration time histories of motions (i.e., two horizontal and one vertical component) are developed for the full column and partial column final SSI input motion spectra for the CB. The same methodology described in Section 3.7.1.1.5.1.1 for the RB/FB is used to develop these time histories.

For the CB partial column spectrally matched time histories, the comparisons between the scaled spectrum compatible acceleration response spectra and the target spectra and boundary range are plotted in Figures 3.7.1-247 through 3.7.1-249 for the H1, H2 and UP directions, respectively. Similar plots for the CB full column spectrally matched time histories are presented in Figures 3.7.1-250 through 3.7.1-252.

The zero-lag cross-correlations for the partial column and full column CB cases are listed in Table 3.7.1-212. These computed values are all less than the required minimum value of 0.16.

In addition to the cross-correlation values, the peak ground motion parameters and associated ratios are listed in Table 3.7.1-213. Since the discussion provided for the RB/FB in Section 3.7.1.1.5.1.1 regarding the composite nature of the target spectra used in the spectral matching procedure is also applicable to the CB, the peak ground motion parameter values and associated ratios are acceptable.

The total duration of the time histories is approximately 30 seconds, which is greater than the required minimum of 20 seconds. In addition, the Arias durations (5-75% Duration), given in Table 3.7.1-213 are longer than the minimum value of 6 seconds defined in SRP 3.7.1.

The acceleration, velocity and displacement time histories for the H1, H2, and UP spectrally matched to the CB partial column final SSI input

response spectra are respectively provided in Figures 3.7.1-253 through 3.7.1-255. Similar figures for the CB full column case are presented in Figures 3.7.1-256 through 3.7.1-258.

The one-sided PSD is computed following SRP 3.7.1, Appendix B, Equation 1, based on a defined time window, Fourier Transform and associated duration, TD. This duration represents the duration of near maximum and nearly stationary power of an acceleration time history and should be consistent with the time window used in the calculation of the Fourier Transform. Several different quantitative definitions of strong motion duration may be defined, while the guidance provided in NUREG/CR-5347, Appendix B, qualitatively states that the duration should be the time range over which the power is nearly constant and near maximum. This can variously be estimated by analysis of the normalized Arias Intensity of a given acceleration time history, while a common estimate of duration is based on the 5–75 percent duration window for the normalized Arias Intensity.

For each spectrum compatible time history, two PSDs were computed based on different time windows for the Fourier Transform and the associated TD duration values. For the first case, the full time history (i.e., total time history length of 29.98 seconds) was used for the Fourier Transform and the TD was estimated from the extrapolation of the computed 5–75 percent values to 0–100 percent values. This extrapolation was implemented to account for the fact that the Fourier Transform is based on the full time window of the given acceleration time history and the expectation that the stationary characteristics of the given time history would not be over the entire time window length, but rather an extrapolated time window based on the 5–75 percent duration value. The use of the full time history for the calculation of the Fourier Transform and PSD function can be considered to be more consistent with the application of the time histories in the SSI analyses which use the full time history in estimating the full frequency content of the given time history.

The second PSD is computed based on taking a windowed section of the given time history for the Fourier Transform. This window duration is based on the 5–75 percent normalized Arias Intensity duration. Prior to the calculation of the Fourier Transform, a cosine taper was applied immediately prior to the start and immediately following the end of this

windowed time history and for the PSD calculation the TD value for the denominator was taken as the 5–75 percent duration value.

For each PSD the prescribed smoothing window of ± 20 percent was used. The PSD results for the CB partial column are provided in Figures 3.7.1-274 through 3.7.1-276. The PSD results for the CB full column are provided in Figures 3.7.1-277 through 3.7.1-279. As expected, the amplitude of the computed PSDs varies as a function of spectral frequency. It can also be observed that the selection of the time window and associated TD value can produce different characteristics of the PSDs as a function of frequency and also in absolute amplitude. The variation between these PSDs is not significant in most cases, indicating a certain degree of robustness in the two approaches.

For the H1 component, the largest difference between the PSDs computed using the full window length and the 5–75 percent truncated window length is in the frequency range of about 25–50 Hz. For the full record length cases, the PSD is nearly constant over the frequency range of about 30–40 Hz with a slight dip in the PSD around 25 Hz. In contrast the PSDs computed from the truncated 5–75 percent window are smoothly attenuating over the frequency range of 25–50 Hz. These observations indicate that the full window time history contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the H2 component, the same observations in the high frequency range of 25–50 Hz are noted. For the truncated 5–75 percent windowed cases, the PSD is again more smoothly varying and is approximately constant or slightly less for frequencies between about 25–40 Hz. These observations also indicate that the full window time history for the H2 component contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the vertical component the same comparison plots of the computed PSD using the two different time windows are presented. However, unlike the two horizontal cases, the PSDs from the truncated 5–75 percent windowed cases do not show a reduction in the relative amplitude of the PSD in the 30–40 Hz frequency range. In general the same relative increase in amplitude around the 30 Hz frequency and relative decrease

in the 20–25 Hz range is observed for both time window PSD calculations. This variation in the PSD is similar in amplitude to the horizontal components using the full time window and could indicate that for the vertical component the relative increase in energy in the 30–40 Hz range is present within the truncated 5–75 percent duration time window. For the truncated 5–75 percent windowed cases, additional minima are observed for frequencies less than about 1 Hz and around 2 Hz. These observations of the relative minima in the vertical component PSDs can be interpreted as an indication of the non-stationarity of the time history because these relative minima are not observed for longer duration windows based on the 5–85 percent and 5–95 percent normalized Arias Intensity values.

The input time histories needed for SSI analysis of the CB with embedded foundation (for both partially embedded and fully embedded cases) are in-column (within) motions corresponding to each of the SSI strain-compatible soil profiles. As such, for each case (partially embedded or fully embedded), each of the outcrop acceleration time histories (H1, H2, and UP), is used as input at the foundation level of the CB to a SHAKE2000 soil column model of the SSI strain-compatible soil profiles (LB, BE and UB), and their corresponding in-column time histories are obtained at the same horizon. The horizontal acceleration time histories are applied using strain-compatible shear wave velocities and the vertical acceleration time history is applied using corresponding P-wave velocities. The strain-compatible shear wave damping is used for both vertical and horizontal analyses. The analyses are carried out linearly with no further degradation of the strain-compatible shear modulus and damping profiles. These analyses result in a total of 18 in-column SSI input time histories corresponding to the three SSI strain compatible profiles (LB, BE, and UB), the three time history components (H1, H2, and UP), and two embedment cases (full column or partial column). These time histories are used as input in the subsequent SSI analyses of the structure.

As described in Section 3.7.1.1.4.2.2, the NEI check for the full column and partial column SSI analyses of the CB are performed directly for the response spectra using RVT. Because the SSI analyses are performed using the spectrally matched time-histories, a confirmatory NEI check is performed using the surface ARS corresponding to the spectrally

matched time-histories for the partially embedded and fully embedded SSI analyses of the building.

For the purpose of the confirmatory NEI check for the CB, the PBSRS corresponding to the CB soil column is used as opposed to the site PBSRS, which is the envelope of the RB/FB and CB DRS at the top of their respective soil columns. The horizontal and vertical components of the full column and partial column CB PBSRS are calculated using the same methodology as described for the full column PBSRS in Section 2.5.2.6, except that only the DRS corresponding to the appropriate CB soil column is used. The PBSRS are smooth spectra because they are obtained from an RVT based analysis. Therefore, the comparison is made between the PBSRS and the ±20 percent smoothed envelope of the surface ARS obtained for each direction of the input time-history in each of the LB, BE, and UB soil profiles. The comparison of the smoothed surface ARS obtained from the SSI input time-histories and the CB PBSRS for each direction, and each considered embedment condition, are shown in Figures 3.7.1-301 through 3.7.1-306. Within the frequency range used for the spectral matching (0.2 to 100 Hz), there are some instances where the PBSRS slightly exceeds the enveloped ARS. These exceedances are limited to 3 percent or less and are considered negligible. These comparisons confirm that the ARS corresponding to the SSI input time-histories calculated at the surface of the SSI soil profiles for the CB do not exhibit any gaps in the site response frequencies at the surface of the SSI input soil columns and, except for negligible exceedance, bound the PBSRS for the building, thus satisfying the intent of the NEI check requirements.

3.7.1.1.5.1.3 **SSI Input Acceleration Time Histories for the FWSC**

Two sets of three statistically independent acceleration time histories of motions (i.e., two horizontal and one vertical component) are developed for the final SSI input motion spectra at Elevation 282 ft and Elevation 220 ft for the FWSC. The same methodology described in Section 3.7.1.1.5.1.1 for the RB/FB is used to develop these time histories.

For the FWSC spectrally matched time histories at Elevation 282 ft, the comparisons between the scaled spectrum compatible acceleration response spectra and the target spectra and boundary range are plotted in Figures 3.7.1-259 through 3.7.1-261 for the H1, H2 and UP directions,

respectively. Similar plots for the FWSC spectrally matched time histories at Elevation 220 ft are presented in Figures 3.7.1-286 through 3.7.1-288.

The zero-lag cross correlations for the FWSC spectrally matched time histories at Elevations 282 ft and 220 ft are listed in Tables 3.7.1-214 and 3.7.1-218, respectively. These computed values are all less than the required minimum value of 0.16.

In addition to the cross correlation values, the peak ground motion parameters and associated ratios for the spectrally matched time histories at Elevations 282 ft and 220 ft are listed in Tables 3.7.1-215 and 3.7.1-219, respectively. Since the discussion provided for the RB/FB in Section 3.7.1.1.5.1.1 regarding the composite nature of the target spectra used in the spectral matching procedure is also applicable to the FWSC, the peak ground motion parameter values and associated ratios are acceptable.

The total duration of the time histories is approximately 30 seconds, which is greater than the required minimum of 20 seconds. In addition, the Arias durations (5-75% Duration), given in Tables 3.7.1-215 and 3.7.1-219 are longer than the minimum value of 6 seconds defined in SRP 3.7.1.

The acceleration, velocity and displacement time histories for the H1, H2, and UP spectrally matched to the FWSC final SSI input response spectra at Elevation 282 ft are provided in Figures 3.7.1-262 through 3.7.1-264, respectively.

Similar plots for the acceleration, velocity and displacement time histories for the H1, H2, and UP spectrally matched to the FWSC final SSI input response spectra at Elevation 220 ft are provided in Figures 3.7.1-289 through 3.7.1-291, respectively.

The one-sided PSD is computed following SRP 3.7.1, Appendix B, Equation 1, based on a defined time window, Fourier Transform and associated duration, TD. This duration represents the duration of near maximum and nearly stationary power of an acceleration time history and should be consistent with the time window used in the calculation of the Fourier Transform. Several different quantitative definitions of strong motion duration may be defined, while the guidance provided in NUREG/CR-5347, Appendix B, qualitatively states that the duration should be the time range over which the power is nearly constant and near maximum. This can variously be estimated by analysis of the

normalized Arias Intensity of a given acceleration time history, while a common estimate of duration is based on the 5–75 percent duration window for the normalized Arias Intensity.

For each spectrum compatible time history, two PSDs were computed based on different time windows for the Fourier Transform and the associated TD duration values. For the first case, the full time history (i.e., total time history length of 29.98 seconds) was used for the Fourier Transform and the TD was estimated from the extrapolation of the computed 5–75 percent values to 0–100 percent values. This extrapolation was implemented to account for the fact that the Fourier Transform is based on the full time window of the given acceleration time history and the expectation that the stationary characteristics of the given time history would not be over the entire time window length, but rather an extrapolated time window based on the 5–75 percent duration value. The use of the full time history for the calculation of the Fourier Transform and PSD function can be considered to be more consistent with the application of the time histories in the SSI analyses which use the full time history in estimating the full frequency content of the given time history.

The second PSD is computed based on taking a windowed section of the given time history for the Fourier Transform. This window duration is based on the 5–75 percent normalized Arias Intensity duration. Prior to the calculation of the Fourier Transform, a cosine taper was applied immediately prior to the start and immediately following the end of this windowed time history and for the PSD calculation the TD value for the denominator was taken as the 5–75 percent duration value.

For each PSD the prescribed smoothing window of ± 20 percent was used. The PSD results for the FWSC at Elevation 282 ft are provided in Figures 3.7.1-280 through 3.7.1-282. The PSD results for the FWSC at Elevation 220 ft are provided in Figures 3.7.1-292 through 3.7.1-294. As expected, the amplitude of the computed PSDs varies as a function of spectral frequency. It can also be observed that the selection of the time window and associated TD value can produce different characteristics of the PSDs as a function of frequency and also in absolute amplitude. The variation between these PSDs is not significant in most cases, indicating a certain degree of robustness in the two approaches.

For the H1 component, the largest difference between the PSDs computed using the full window length and the 5–75 percent truncated

window length is in the frequency range of about 25–50 Hz. For the full record length cases, the PSD is nearly constant over the frequency range of about 30–40 Hz with a slight dip in the PSD around 25 Hz. In contrast the PSDs computed from the truncated 5–75 percent window are smoothly attenuating over the frequency range of 25–50 Hz. These observations indicate that the full window time history contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the H2 component, the same observations in the high frequency range of 25–50 Hz are noted. For the truncated 5–75 percent windowed cases, the PSD is again more smoothly varying and is approximately constant or slightly less for frequencies between about 25–40 Hz. These observations also indicate that the full window time history for the H2 component contains energy in this high frequency range outside of the 5–75 percent duration time window and would not indicate a potential gap in frequency content, but rather an increase in the 30–40 Hz frequency range.

For the vertical component the same comparison plots of the computed PSD using the two different time windows are presented. However, unlike the two horizontal cases, the PSDs from the truncated 5–75 percent windowed cases do not show a reduction in the relative amplitude of the PSD in the 30–40 Hz frequency range. In general the same relative increase in amplitude around the 30 Hz frequency and relative decrease in the 20–25 Hz range is observed for both time window PSD calculations. This variation in the PSD is similar in amplitude to the horizontal components using the full time window and could indicate that for the vertical component the relative increase in energy in the 30–40 Hz range is present within the truncated 5–75 percent duration time window. For the truncated 5–75 percent windowed cases, additional minima are observed for frequencies less than about 1 Hz and around 2 Hz. These observations of the relative minima in the vertical component PSDs can be interpreted as an indication of the non-stationarity of the time history because these relative minima are not observed for longer duration windows based on the 5–85 percent and 5–95 percent normalized Arias Intensity values.

The input time histories needed for SSI analysis of the FWSC with the control motion at Elevation 220 ft are in-column (within) motions

corresponding to each of the SSI strain-compatible soil profiles. As such, each of the outcrop acceleration time histories (H1, H2, and UP) at Elevation 220 ft, is used as input at the same elevation to a SHAKE2000 soil column model of the FWSC SSI strain-compatible soil profiles (LB, BE and UB), and their corresponding in-column time histories are obtained at Elevation 220 ft. The horizontal acceleration time histories are applied using strain-compatible shear wave velocities and the vertical acceleration time history is applied using corresponding P-wave velocities. The strain-compatible shear wave damping is used for both vertical and horizontal analyses. The analyses are carried out linearly with no further degradation of the strain-compatible shear modulus and damping profiles. These analyses result in a total of nine in-column SSI input time histories corresponding to the three SSI strain compatible profiles (LB, BE, and UB) and the three time history components (H1, H2, and UP). These time histories are used as input in the subsequent SSI analyses of the FWSC with the control input motion at Elevation 220 ft.

3.7.1.1.6 ~~Site-Dependent At-Grade SSE and OBE Response Spectra~~ Site-Dependent SSE At-Grade and Site-Dependent OBE At-Grade Response Spectra

The site-dependent SSE at grade is defined by enveloping the following two spectra:

1. PBSRS calculated at grade (Elevation 290 ft) from full soil column analyses for RB/FB and CB and,

2. The minimum required response spectra defined as the RG 1.60 broadband horizontal and vertical response spectra at 5 percent damping anchored to 0.1g at PGA to satisfy the requirements of SRP 3.7.1.

The site-dependent OBE at grade is defined as one-third of the site-dependent SSE at grade. The site-dependent OBE response spectra at grade will serve as one reference against which OBE exceedance checks are to be performed for the purpose of plant shutdown, as described in Section 3.7.1. Section 3.7.4.4 includes the criteria that are used to determine whether a plant shutdown is required following a seismic event.

The horizontal and vertical PBSRS at grade is presented in Section 2.5.2.6. The horizontal and vertical 5 percent damped

site-dependent SSE spectra at grade are presented in Figures 3.7.1-265 and 3.7.1-266, respectively.

The horizontal and vertical free-field site-dependent OBE at grade are calculated as one-third of the site-dependent SSE at grade and presented in Figure 3.7.1-267.

The 5 percent damped pseudo velocity response spectra (VRS) for site-dependent OBE at grade is determined by dividing the ARS values at each frequency point (f) by $2\pi f$. The digital values for the site-dependent SSE and OBE at grade are presented in Tables 3.7.1-216 and 3.7.1-217, respectively.

3.7.1.2 Percentage of Critical Damping Values

Add the following at the end of the first paragraph.

NAPS DEP 3.7-1

OBE structural damping values consistent with RG 1.61 Revision 1 are used in the Unit 3 site-specific SSI analyses unless SSE damping in DCD Table 3.7-1 is justified by stress demand. Section 3A.13.2 describes the damping values used in the Unit 3 site-specific SSI analyses.

3.7.1.3 Supporting Media for Seismic Category I Structures

Add the following at the end of the first paragraph.

NAPS SUP 3.7-3

The Seismic Category I structures for Unit 3 have concrete mat foundations founded on rock or concrete fill on rock.

NAPS SUP 3.7-1

Table 3.7.1-201 Strain-Compatible SSI Input Properties for RB/FB (In-situ Material)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-RB/FB Vs (ft/sec)	BE-RB/FB Vp (ft/sec)	BE-RB/FB Damping (%)	LB-RB/FB Vs (ft/sec)	LB-RB/FB Vp (ft/sec)	LB-RB/FB Damping (%)	UB-RB/FB Vs (ft/sec)	UB-RB/FB Vp (ft/sec)	UB-RB/FB Damping (%)
1	2.00	0	0.125	908	2224	2.07	616	1508	3.50	1339	3279	1.22
2	2.50	2	0.125	875	2498	3.38	534	1523	7.02	1435	4095	1.63
3	2.50	4.5	0.125	875	2498	3.38	534	1523	7.02	1435	4095	1.63
4	2.50	7	0.13	1302	5471	2.89	814	4152	5.52	2081	8745	1.52
5	2.50	9.5	0.13	1302	5471	2.89	814	4152	5.52	2081	8745	1.52
6	2.50	12	0.13	1887	5082	2.50	1259	4800	4.40	2829	7616	1.42
7	2.50	14.5	0.13	1887	5082	2.50	1259	4800	4.40	2829	7616	1.42
8	3.00	17	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
9	3.00	20	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
10	3.00	23	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
11	3.00	26	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
12	3.00	29	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
13	3.00	32	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
14	3.00	35	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
15	2.00	38	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
16	3.00	40	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
17	3.00	43	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
18	3.00	46	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
19	4.00	49	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
20	3.00	53	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33

NAPS SUP 3.7-1

Table 3.7.1-201 Strain-Compatible SSI Input Properties for RB/FB (In-situ Material) *(continued)*

				BE-RB/FB			LB-RB/FB			UB-RB/FB		
Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	Vs (ft/sec)	Vp (ft/sec)	Damping (%)	Vs (ft/sec)	Vp (ft/sec)	Damping (%)	Vs (ft/sec)	Vp (ft/sec)	Damping (%)
21	3.00	56	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
22	3.00	59	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
23	4.00	62	0.145	4324	10593	0.58	3212	7869	1.02	5821	14259	0.33
24	3.00	66	0.163	5449	13347	1.00	4037	9888	1.82	7355	18017	0.55
25	4.00	69	0.163	5449	13347	1.00	4037	9888	1.82	7355	18017	0.55
26	4.00	73	0.163	5449	13347	1.00	4037	9888	1.82	7355	18017	0.55
27	4.00	77	0.163	5449	13347	1.00	4037	9888	1.82	7355	18017	0.55
28	4.00	81	0.163	5449	13347	1.00	4037	9888	1.82	7355	18017	0.55
29	4.00	85	0.163	5178	12682	1.00	3471	8501	1.82	7724	18920	0.55
30	4.00	89	0.163	5178	12682	1.00	3471	8501	1.82	7724	18920	0.55
31	4.00	93	0.163	5178	12682	1.00	3471	8501	1.82	7724	18920	0.55
32	4.00	97	0.163	5178	12682	1.00	3471	8501	1.82	7724	18920	0.55
33	5.00	101	0.163	5178	12682	1.00	3471	8501	1.82	7724	18920	0.55
34	4.00	106	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
35	5.00	110	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
36	5.00	115	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
37	5.00	120	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
38	5.00	125	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
39	5.00	130	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
40	5.00	135	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55

NAPS SUP 3.7-1

Table 3.7.1-201 Strain-Compatible SSI Input Properties for RB/FB (In-situ Material) *(continued)*

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-RB/FB			LB-RB/FB			UB-RB/FB		
				Vs (ft/sec)	Vp (ft/sec)	Damping (%)	Vs (ft/sec)	Vp (ft/sec)	Damping (%)	Vs (ft/sec)	Vp (ft/sec)	Damping (%)
41	5.00	140	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
42	5.00	145	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
43	5.00	150	0.164	8800	15678	1.00	7185	12801	1.82	10778	19201	0.55
44		155	0.164	9200	16390	1.00	7512	13383	1.82	11268	20074	0.55

The top 7 layers correspond to saprolite and are removed in the partially embedded SSI analysis of the RB/FB.
Groundwater table is considered at the top of the fourth layer at Elevation 283 ft.

NAPS SUP 3.7-1

Table 3.7.1-202 Strain-Compatible SSI Input Properties for RB/FB (Structural Fill and Concrete Fill)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-RB/FB Vs (ft/sec)	BE-RB/FB Vp (ft/sec)	BE-RB/FB Damping (%)	LB-RB/FB Vs (ft/sec)	LB-RB/FB Vp (ft/sec)	LB-RB/FB Damping (%)	UB-RB/FB Vs (ft/sec)	UB-RB/FB Vp (ft/sec)	UB-RB/FB Damping (%)
1	2.00	0	0.130	734	1372	2.18	531	993	3.12	1014	1897	1.52
2	2.50	2	0.130	649	1213	4.24	418	781	7.00	1007	1884	2.57
3	2.50	4.5	0.130	649	1213	4.24	418	781	7.00	1007	1884	2.57
4	2.50	7	0.130	710	3619	5.13	444	2262	8.45	1135	4800	3.11
5	2.50	9.5	0.130	710	3619	5.13	444	2262	8.45	1135	4800	3.11
6	2.50	12	0.130	736	3752	5.80	469	2392	9.40	1154	4800	3.58
7	2.50	14.5	0.130	736	3752	5.80	469	2392	9.40	1154	4800	3.58
Concrete Fill		17	0.145	7000	10909	1.00	6000	9350	1.80	8000	12467	0.55

Groundwater table is considered at the top of the fourth layer at Elevation 283 ft.

NAPS SUP 3.7-1

Table 3.7.1-203 Strain-Compatible SSI Input Properties for CB (In-situ Material)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-CB Vs (ft/sec)	BE-CB Vp (ft/sec)	BE-CB Damping (%)	LB-CB Vs (ft/sec)	LB-CB Vp (ft/sec)	LB-CB Damping (%)	UB-CB Vs (ft/sec)	UB-CB Vp (ft/sec)	UB-CB Damping (%)
1	2.50	0	0.125	873	1773	2.78	590	1198	4.83	1292	2625	1.60
2	2.50	2.5	0.125	873	1773	2.78	590	1198	4.83	1292	2625	1.60
3	2.50	5	0.125	1007	4231	3.93	611	2567	7.42	1659	6975	2.08
4	2.50	7.5	0.125	1007	4800	3.93	611	3114	7.42	1659	6975	2.08
5	2.50	10	0.125	1007	4800	3.93	611	3114	7.42	1659	6975	2.08
6	2.50	12.5	0.125	1007	4800	3.93	611	3114	7.42	1659	6975	2.08
7	2.50	15	0.13	1419	5966	3.78	990	4800	6.53	2036	8557	2.19
8	2.50	17.5	0.13	1419	5966	3.78	990	4800	6.53	2036	8557	2.19
9	2.50	20	0.13	1419	5966	3.78	990	4800	6.53	2036	8557	2.19
10	2.50	22.5	0.13	1419	5966	3.78	990	4800	6.53	2036	8557	2.19
11	2.50	25	0.145	2024	6184	0.62	1545	4800	1.06	2651	8100	0.36
12	2.50	27.5	0.145	2024	6184	0.62	1545	4800	1.06	2651	8100	0.36
13	2.50	30	0.145	2024	6184	0.62	1545	4800	1.06	2651	8100	0.36
14	2.50	32.5	0.145	2024	6184	0.62	1545	4800	1.06	2651	8100	0.36
15	2.50	35	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
16	2.50	37.5	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
17	2.50	40	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
18	2.50	42.5	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
19	2.50	45	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
20	1.50	47.5	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35

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Table 3.7.1-203 Strain-Compatible SSI Input Properties for CB (In-situ Material) *(continued)*

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-CB Vs (ft/sec)	BE-CB Vp (ft/sec)	BE-CB Damping (%)	LB-CB Vs (ft/sec)	LB-CB Vp (ft/sec)	LB-CB Damping (%)	UB-CB Vs (ft/sec)	UB-CB Vp (ft/sec)	UB-CB Damping (%)
21	3.50	49	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
22	2.50	52.5	0.145	2475	7561	0.63	1841	5625	1.14	3327	10163	0.35
23	2.50	55	0.145	2660	8127	0.53	2172	6635	0.96	3258	9953	0.29
24	2.50	57.5	0.145	2660	8127	0.53	2172	6635	0.96	3258	9953	0.29
25	2.50	60	0.145	2660	8127	0.53	2172	6635	0.96	3258	9953	0.29
26	2.50	62.5	0.145	2660	8127	0.53	2172	6635	0.96	3258	9953	0.29
27	1.00	65	0.163	6483	13861	1.00	5293	11318	1.82	7940	16976	0.55
28	3.00	66	0.163	6483	13861	1.00	5293	11318	1.82	7940	16976	0.55
29	3.00	69	0.163	6483	13861	1.00	5293	11318	1.82	7940	16976	0.55
30	3.00	72	0.163	6483	13861	1.00	5293	11318	1.82	7940	16976	0.55
31	3.00	75	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
32	2.00	78	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
33	5.00	80	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
34	5.00	85	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
35	5.00	90	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
36	5.00	95	0.163	7942	15135	1.00	6485	12358	1.82	9727	18536	0.55
37	5.00	100	0.163	7942	15135	1.00	6485	12358	1.82	9727	18536	0.55
38	5.00	105	0.164	8655	15657	1.00	7067	12784	1.82	10600	19176	0.55
39	5.00	110	0.164	8655	15657	1.00	7067	12784	1.82	10600	19176	0.55
40	5.00	115	0.164	8242	15707	1.00	6730	12824	1.82	10094	19236	0.55

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Table 3.7.1-203 Strain-Compatible SSI Input Properties for CB (In-situ Material) *(continued)*

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-CB Vs (ft/sec)	BE-CB Vp (ft/sec)	BE-CB Damping (%)	LB-CB Vs (ft/sec)	LB-CB Vp (ft/sec)	LB-CB Damping (%)	UB-CB Vs (ft/sec)	UB-CB Vp (ft/sec)	UB-CB Damping (%)
41	5.00	120	0.164	8242	15707	1.00	6730	12824	1.82	10094	19236	0.55
42	5.00	125	0.164	8658	16198	1.00	7069	13225	1.82	10604	19838	0.55
43	5.00	130	0.164	8658	16198	1.00	7069	13225	1.82	10604	19838	0.55
44	5.00	135	0.164	8822	15491	1.00	7203	12648	1.82	10805	18972	0.55
45	5.00	140	0.164	8822	15491	1.00	7203	12648	1.82	10805	18972	0.55
46	5.00	145	0.164	9340	16897	1.00	7626	13796	1.82	11439	20694	0.55
47	5.00	150	0.164	9340	16897	1.00	7626	13796	1.82	11439	20694	0.55
48	5.00	155	0.164	9198	17208	1.00	7510	14050	1.82	11265	21075	0.55
49	5.00	160	0.164	9198	17208	1.00	7510	14050	1.82	11265	21075	0.55
50		165	0.164	9200	15729	1.00	7512	12843	1.82	11268	19264	0.55

The top 10 layers correspond to saprolite and are removed in the partially embedded SSI analysis of the CB.
Groundwater table is considered at the top of the fourth layer at Elevation 282.5 ft.

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Table 3.7.1-204 Strain-Compatible SSI Input Properties for CB (Structural Fill and Concrete Fill)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-CB Vs (ft/sec)	BE-CB Vp (ft/sec)	BE-CB Damping (%)	LB-CB Vs (ft/sec)	LB-CB Vp (ft/sec)	LB-CB Damping (%)	UB-CB Vs (ft/sec)	UB-CB Vp (ft/sec)	UB-CB Damping (%)
1	2.50	0	0.130	649	1214	3.14	440	823	4.98	958	1791	1.98
2	2.50	2.5	0.130	649	1214	3.14	440	823	4.98	958	1791	1.98
3	2.50	5	0.130	747	1397	4.41	486	909	7.17	1149	2149	2.72
4	2.50	7.5	0.130	747	3809	4.41	486	2477	7.17	1149	4800	2.72
5	2.50	10	0.130	751	3831	5.21	478	2436	8.38	1182	4800	3.24
6	2.50	12.5	0.130	751	3831	5.21	478	2436	8.38	1182	4800	3.24
7	2.50	15	0.130	759	3872	5.92	492	2507	9.11	1173	4800	3.84
8	2.50	17.5	0.130	759	3872	5.92	492	2507	9.11	1173	4800	3.84
9	2.50	20	0.130	813	4147	5.67	515	2626	8.93	1284	4800	3.60
10	2.50	22.5	0.130	813	4147	5.67	515	2626	8.93	1284	4800	3.60
Concrete Fill		25	0.145	7000	10909	1.00	6000	9350	1.80	8000	12467	0.55

Groundwater table is considered at the top of the fourth layer at Elevation 282.5 ft.

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Table 3.7.1-205 Strain-Compatible SSI Input Properties for FWSC (In-situ Material)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-FWSC Vs (ft/sec)	BE-FWSC Vp (ft/sec)	BE-FWSC Damping (%)	LB-FWSC Vs (ft/sec)	LB-FWSC Vp (ft/sec)	LB-FWSC Damping (%)	UB-FWSC Vs (ft/sec)	UB-FWSC Vp (ft/sec)	UB-FWSC Damping (%)
1	3.00	0	0.125	742	3783	4.18	483	2460	6.63	1141	4800	2.64
2	3.00	3	0.125	742	3783	4.18	483	2460	6.63	1141	4800	2.64
3	3.00	6	0.125	742	3783	4.18	483	2460	6.63	1141	4800	2.64
4	3.00	9	0.125	742	3783	4.18	483	2460	6.63	1141	4800	2.64
5	3.00	12	0.125	979	4800	5.00	605	3083	8.25	1585	6661	3.03
6	3.00	15	0.125	979	4800	5.00	605	3083	8.25	1585	6661	3.03
7	3.00	18	0.125	979	4800	5.00	605	3083	8.25	1585	6661	3.03
8	4.00	21	0.125	979	4800	5.00	605	3083	8.25	1585	6661	3.03
9	4.00	25	0.125	979	4800	5.00	605	3083	8.25	1585	6661	3.03
10	4.00	29	0.13	1416	5952	3.97	1018	4800	6.08	1970	8280	2.60
11	3.00	33	0.13	1955	5974	2.89	1431	4800	4.37	2672	8164	1.92
12	2.00	36	0.13	1955	5974	2.89	1431	4800	4.37	2672	8164	1.92
13	4.00	38	0.145	2503	7647	0.64	1907	5825	1.14	3286	10040	0.36
14	4.00	42	0.145	2503	7647	0.64	1907	5825	1.14	3286	10040	0.36
15	4.00	46	0.145	2503	7647	0.64	1907	5825	1.14	3286	10040	0.36
16	4.00	50	0.145	2503	7647	0.64	1907	5825	1.14	3286	10040	0.36
17	4.00	54	0.145	2693	8228	0.53	2150	6568	0.94	3373	10306	0.30
18	4.00	58	0.145	2693	8228	0.53	2150	6568	0.94	3373	10306	0.30
19	3.00	62	0.163	6483	13861	1.00	4803	10269	1.82	8751	18711	0.55
20	3.00	65	0.163	6483	13861	1.00	4803	10269	1.82	8751	18711	0.55

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Table 3.7.1-205 Strain-Compatible SSI Input Properties for FWSC (In-situ Material) *(continued)*

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-FWSC Vs (ft/sec)	BE-FWSC Vp (ft/sec)	BE-FWSC Damping (%)	LB-FWSC Vs (ft/sec)	LB-FWSC Vp (ft/sec)	LB-FWSC Damping (%)	UB-FWSC Vs (ft/sec)	UB-FWSC Vp (ft/sec)	UB-FWSC Damping (%)
21	3.00	68	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
22	4.00	71	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
23	4.00	75	0.163	6983	14018	1.00	5701	11445	1.82	8552	17168	0.55
24	3.00	79	0.164	7942	15135	1.00	6485	12358	1.82	9727	18536	0.55
25	4.00	82	0.164	7942	15135	1.00	6485	12358	1.82	9727	18536	0.55
26	4.00	86	0.164	7942	15135	1.00	6485	12358	1.82	9727	18536	0.55
27	3.00	90	0.164	8655	15657	1.00	7067	12784	1.82	10600	19176	0.55
28	4.00	93	0.164	8655	15657	1.00	7067	12784	1.82	10600	19176	0.55
29	4.00	97	0.164	8655	15657	1.00	7067	12784	1.82	10600	19176	0.55
30	3.00	101	0.164	8242	15707	1.00	6730	12824	1.82	10094	19236	0.55
31	4.00	104	0.164	8242	15707	1.00	6730	12824	1.82	10094	19236	0.55
32	4.00	108	0.164	8242	15707	1.00	6730	12824	1.82	10094	19236	0.55
33	3.00	112	0.164	8658	16198	1.00	7069	13225	1.82	10604	19838	0.55
34	4.00	115	0.164	8658	16198	1.00	7069	13225	1.82	10604	19838	0.55
35	4.00	119	0.164	8658	16198	1.00	7069	13225	1.82	10604	19838	0.55
36	3.00	123	0.164	8822	15491	1.00	7203	12648	1.82	10805	18972	0.55
37	4.00	126	0.164	8822	15491	1.00	7203	12648	1.82	10805	18972	0.55
38	4.00	130	0.164	8822	15491	1.00	7203	12648	1.82	10805	18972	0.55
39	3.00	134	0.164	9340	16897	1.00	7626	13796	1.82	11439	20694	0.55
40	4.00	137	0.164	9340	16897	1.00	7626	13796	1.82	11439	20694	0.55

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Table 3.7.1-205 Strain-Compatible SSI Input Properties for FWSC (In-situ Material) *(continued)*

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-FWSC Vs (ft/sec)	BE-FWSC Vp (ft/sec)	BE-FWSC Damping (%)	LB-FWSC Vs (ft/sec)	LB-FWSC Vp (ft/sec)	LB-FWSC Damping (%)	UB-FWSC Vs (ft/sec)	UB-FWSC Vp (ft/sec)	UB-FWSC Damping (%)
41	4.00	141	0.164	9340	16897	1.00	7626	13796	1.82	11439	20694	0.55
42	3.00	145	0.164	9198	17208	1.00	7510	14050	1.82	11265	21075	0.55
43	4.00	148	0.164	9198	17208	1.00	7510	14050	1.82	11265	21075	0.55
44	4.00	152	0.164	9198	17208	1.00	7510	14050	1.82	11265	21075	0.55
45		156	0.164	9200	15729	1.00	7512	12843	1.82	11268	19264	0.55

Depth is measured with respect to the bottom of the foundation at Elevation 282 ft.
Groundwater table is considered at the top of the first layer at Elevation 282 ft.

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Table 3.7.1-206 Strain-Compatible SSI Input Properties for FWSC (Structural Fill and Concrete Fill)

Layer #	Thickness (ft)	Top-Depth (ft)	Unit Weight (kcf)	BE-FWSC Vs (ft/sec)	BE-FWSC Vp (ft/sec)	BE-FWSC Damping (%)	LB-FWSC Vs (ft/sec)	LB-FWSC Vp (ft/sec)	LB-FWSC Damping (%)	UB-FWSC Vs (ft/sec)	UB-FWSC Vp (ft/sec)	UB-FWSC Damping (%)
1	3.00	0	0.130	745	3799	4.74	479	2440	7.33	1160	4800	3.06
2	3.00	3	0.130	759	3872	4.83	491	2504	7.50	1174	4800	3.11
3	3.00	6	0.130	784	4000	4.97	503	2567	7.85	1223	4800	3.15
4	3.00	9	0.130	783	3991	5.33	507	2586	8.29	1208	4800	3.42
5	3.00	12	0.130	845	4311	5.17	526	2683	8.02	1358	4800	3.33
6	3.00	15	0.130	830	4234	5.47	509	2593	8.50	1356	4800	3.52
7	3.00	18	0.130	830	4235	5.65	518	2640	8.54	1332	4800	3.74
8	4.00	21	0.130	852	4347	5.67	520	2654	8.87	1396	4800	3.63
9	4.00	25	0.130	848	4325	5.96	514	2620	9.19	1400	4800	3.87
10	4.00	29	0.130	894	4558	5.75	551	2809	8.74	1451	4800	3.78
11	3.00	33	0.130	890	4538	5.95	531	2706	9.35	1492	4800	3.79
12	2.00	36	0.130	890	4538	5.95	531	2706	9.35	1492	4800	3.79
Concrete Fill		38	0.145	7000	10909	1.00	6000	9350	1.82	8000	12467	0.55

Depth is measured with respect to the bottom of the foundation at Elevation 282 ft.
Groundwater table is considered at the top of the first layer at Elevation 282 ft.

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Table 3.7.1-207 5% Damped Final SSI Input Response Spectra for RB/FB

	Final Full Column SSI Input Response Spectra		Final Partial Column SSI Input Response Spectra	
Frequency (Hz)	Horizontal (g)	Vertical (g)	Horizontal (g)	Vertical (g)
100	0.563	0.563	0.586	0.586
90	0.605	0.627	0.632	0.656
80	0.677	0.739	0.708	0.772
70	0.795	0.899	0.821	0.929
60	0.953	1.10	0.992	1.14
50	1.09	1.26	1.13	1.30
45	1.13	1.26	1.19	1.32
40	1.17	1.22	1.21	1.26
35	1.18	1.16	1.22	1.20
30	1.21	1.13	1.20	1.13
25	1.16	1.02	1.17	1.03
20	1.13	0.98	1.16	0.962
15	1.11	1.15	1.19	0.938
12.5	1.09	1.22	1.17	0.905
10	1.00	1.22	1.05	0.790
9	0.943	1.18	0.953	0.715
8	0.868	1.11	0.841	0.631
7	0.776	1.00	0.713	0.535
6	0.754	0.866	0.587	0.440
5	0.674	0.695	0.474	0.355
4	0.508	0.481	0.367	0.292
3	0.327	0.288	0.305	0.261
2.5	0.312	0.224	0.312	0.224
2	0.261	0.185	0.261	0.185
1.5	0.206	0.145	0.206	0.145
1.25	0.177	0.124	0.177	0.124
1	0.147	0.103	0.147	0.103

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Table 3.7.1-207 5% Damped Final SSI Input Response Spectra for RB/FB *(continued)*

	Final Full Column SSI Input Response Spectra		Final Partial Column SSI Input Response Spectra	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**	**Horizontal (g)**	**Vertical (g)**
0.9	0.135	0.0938	0.135	0.0938
0.8	0.123	0.0848	0.123	0.0848
0.7	0.110	0.0757	0.110	0.0757
0.6	0.0969	0.0664	0.0969	0.0664
0.5	0.0834	0.0569	0.0834	0.0569
0.4	0.0694	0.0470	0.0694	0.0470
0.3	0.0548	0.0368	0.0548	0.0368
0.2	0.0302	0.0202	0.0302	0.0202
0.167	0.0210	0.0141	0.0210	0.0141
0.125	0.0118	0.00798	0.0118	0.00792
0.1	0.00852	0.00639	0.00842	0.00631

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Table 3.7.1-208 5% Damped Final SSI Input Response Spectra for CB

	Final Full Column SSI Input Response Spectra		Final Partial Column SSI Input Response Spectra	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**	**Horizontal (g)**	**Vertical (g)**
100	0.749	0.749	0.800	0.838
90	0.807	0.837	0.854	0.932
80	0.896	0.977	0.948	1.08
70	1.04	1.18	1.08	1.27
60	1.26	1.45	1.33	1.60
50	1.43	1.65	1.51	1.85
45	1.48	1.65	1.57	1.88
40	1.51	1.58	1.63	1.87
35	1.56	1.53	1.67	1.81
30	1.59	1.49	1.63	1.67
25	1.53	1.34	1.61	1.52
20	1.48	1.22	1.70	1.48
15	1.45	1.23	1.79	1.46
12.5	1.42	1.29	1.74	1.37
10	1.33	1.27	1.44	1.10
9	1.26	1.23	1.22	0.928
8	1.16	1.15	0.988	0.749
7	1.03	1.03	0.786	0.594
6	0.861	0.885	0.626	0.473
5	0.679	0.708	0.498	0.375
4	0.483	0.488	0.382	0.293
3	0.322	0.292	0.305	0.261
2.5	0.312	0.224	0.312	0.224
2	0.261	0.185	0.261	0.185
1.5	0.206	0.145	0.206	0.145
1.25	0.177	0.124	0.177	0.124
1	0.147	0.103	0.147	0.103

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Table 3.7.1-208 5% Damped Final SSI Input Response Spectra for CB *(continued)*

	Final Full Column SSI Input Response Spectra		Final Partial Column SSI Input Response Spectra	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**	**Horizontal (g)**	**Vertical (g)**
0.9	0.135	0.0938	0.135	0.0938
0.8	0.123	0.0848	0.123	0.0848
0.7	0.110	0.0757	0.110	0.0757
0.6	0.0969	0.0664	0.0969	0.0664
0.5	0.0834	0.0569	0.0834	0.0569
0.4	0.0694	0.0470	0.0694	0.0470
0.3	0.0548	0.0368	0.0548	0.0368
0.2	0.0302	0.0202	0.0302	0.0202
0.167	0.0210	0.0141	0.0210	0.0141
0.125	0.0118	0.00794	0.0118	0.00793
0.1	0.00845	0.00642	0.00844	0.00633

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Table 3.7.1-209 5% Damped Final SSI Input Response Spectra at Elevation 282 ft for FWSC

	Final SSI Input Response Spectra	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**
100	0.691	0.691
90	0.708	0.734
80	0.735	0.802
70	0.783	0.886
60	0.859	0.989
50	0.972	1.12
45	1.05	1.17
40	1.14	1.18
35	1.23	1.20
30	1.35	1.26
25	1.45	1.28
20	1.59	1.31
15	1.64	1.29
12.5	1.59	1.22
10	1.57	1.18
9	1.56	1.17
8	1.51	1.13
7	1.38	1.04
6	1.21	0.908
5	1.01	0.758
4	0.766	0.575
3	0.497	0.373
2.5	0.358	0.269
2	0.261	0.186
1.5	0.206	0.145
1.25	0.177	0.124
1	0.147	0.103

NAPS SUP 3.7-1

Table 3.7.1-209 5% Damped Final SSI Input Response Spectra at Elevation 282 ft for FWSC *(continued)*

	Final SSI Input Response Spectra	
Frequency (Hz)	**Horizontal (g)**	**Vertical (g)**
0.9	0.135	0.0938
0.8	0.123	0.0848
0.7	0.110	0.0757
0.6	0.0969	0.0664
0.5	0.0834	0.0569
0.4	0.0694	0.0470
0.3	0.0548	0.0368
0.2	0.0302	0.0202
0.167	0.0210	0.0141
0.125	0.0118	0.00799
0.1	0.00853	0.00640

NAPS SUP 3.7-2

Table 3.7.1-210 Zero-Lag Cross-Correlation Coefficients for the Final Scaled Spectrum Compatible Acceleration Time-Histories for the RB/FB

RB/FB Full Profile

Components	Zero-Lag Cross-Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.018
H1 – UP	0.015
H2 – UP	-0.014

RB/FB Partial Profile

Components	Zero-Lag Cross-Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.042
H1 – UP	0.033
H2 – UP	-0.016

NAPS SUP 3.7-2

Table 3.7.1-211 Peak Ground Motion Parameters, Associated Ratios, and Strong Motion Duration Values for the Final Scaled Spectrum Compatible Acceleration Time-Histories for the RB/FB

Parameter	H1	H2	UP
RB/FB Full Profile			
PGA (g)	0.572	0.565	0.568
PGV (cm/sec)	21.622	15.950	18.257
PGD (cm)	7.964	8.745	7.279
PGV/PGA (cm/sec/g)	37.769	28.243	32.142
$PGA*PGD/PGV^2$	9.562	19.033	12.162
5-75% Duration Time (sec)	7.495	10.215	6.420
5% Duration Time (sec)	1.085	1.245	0.960
75% Duration Time (sec)	8.580	11.460	7.380
0% Extrapolated Duration Time (sec)	0.550	0.515	0.501
100% Extrapolated Duration Time (sec)	11.257	15.108	9.673
0-100% Extrapolated Duration Time (sec)	10.707	14.593	9.171
RB/FB Partial Profile			
PGA (g)	0.586	0.587	0.591
PGV (cm/sec)	21.446	14.915	16.383
PGD (cm)	7.527	8.795	7.359
PGV/PGA (cm/sec/g)	36.586	25.408	27.706
$PGA*PGD/PGV^2$	9.405	22.755	15.897
5-75% Duration Time (sec)	7.590	10.465	7.380
5% Duration Time (sec)	1.070	1.210	0.985
75% Duration Time (sec)	8.660	11.675	8.365
0% Extrapolated Duration Time (sec)	0.528	0.463	0.458
100% Extrapolated Duration Time (sec)	11.371	15.413	11.001
0-100% Extrapolated Duration Time (sec)	10.843	14.950	10.543

NAPS SUP 3.7-2

Table 3.7.1-212 Zero-Lag Cross-Correlation Coefficients for the Final Scaled Spectrum Compatible Acceleration Time-Histories for the CB

CB Full Profile

Components	Zero-Lag Cross-Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.041
H1 – UP	0.024
H2 – UP	-0.022

CB Partial Profile

Components	Zero-Lag Cross-Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.040
H1 – UP	0.031
H2 – UP	-0.025

NAPS SUP 3.7-2

Table 3.7.1-213 Peak Ground Motion Parameters, Associated Ratios, and Strong Motion Duration Values for the Final Scaled Spectrum Compatible Acceleration Time-Histories for the CB

Parameter	H1	H2	UP
CB Full Profile			
PGA (g)	0.745	0.753	0.756
PGV (cm/sec)	20.604	18.030	17.793
PGD (cm)	6.862	7.384	6.355
PGV/PGA (cm/sec/g)	27.656	23.959	23.551
PGA*PGD/PGV^2	11.808	16.759	14.870
5-75% Duration Time (sec)	7.975	10.840	6.180
5% Duration Time (sec)	1.085	1.240	0.955
75% Duration Time (sec)	9.060	12.080	7.135
0% Extrapolated Duration Time (sec)	0.515	0.466	0.514
100% Extrapolated Duration Time (sec)	11.908	15.951	9.342
0-100% Extrapolated Duration Time (sec)	11.393	15.486	8.829
CB Partial Profile			
PGA (g)	0.798	0.807	0.838
PGV (cm/sec)	19.890	16.990	15.259
PGD (cm)	7.072	7.917	6.186
PGV/PGA (cm/sec/g)	24.922	21.045	18.203
PGA*PGD/PGV^2	13.988	21.711	21.836
5-75% Duration Time (sec)	7.910	10.725	6.320
5% Duration Time (sec)	1.055	1.195	1.000
75% Duration Time (sec)	8.965	11.920	7.320
0% Extrapolated Duration Time (sec)	0.490	0.429	0.549
100% Extrapolated Duration Time (sec)	11.790	15.750	9.577
0-100% Extrapolated Duration Time (sec)	11.300	15.321	9.029

NAPS SUP 3.7-2

Table 3.7.1-214 Zero-Lag Cross-Correlation Coefficients for the Final Scaled Spectrum Compatible Acceleration Time Histories for the FWSC at Elevation 282 ft

Components	Zero-Lag Cross-Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.016
H1 – UP	-0.012
H2 – UP	0.017

NAPS SUP 3.7-2

Table 3.7.1-215 Peak Ground Motion Parameters, Associated Ratios, and Strong Motion Duration Values for the Final Scaled Spectrum Compatible Acceleration Time Histories for the FWSC at Elevation 282 ft

Parameter	H1	H2	UP
PGA (g)	0.697	0.686	0.668
PGV (cm/sec)	21.061	19.057	16.151
PGD (cm)	9.360	9.108	6.072
PGV/PGA (cm/sec/g)	30.212	27.793	24.190
$PGA*PGD/PGV^2$	14.424	16.860	15.238
5-75% Duration Time (sec)	7.120	10.185	6.505
5% Duration Time (sec)	1.085	1.340	0.955
75% Duration Time (sec)	8.205	11.525	7.460
0% Extrapolated Duration Time (sec)	0.576	0.613	0.490
100% Extrapolated Duration Time (sec)	10.748	15.163	9.783
0-100% Extrapolated Duration Time (sec)	10.171	14.550	9.293

NAPS SUP 3.7-2

Table 3.7.1-216 Site-Dependent SSE and OBE 5% Damping Acceleration Response Spectra at Grade

Frequency (Hz)	Horizontal SSE at Grade (g)	Vertical SSE at Grade (g)	Horizontal OBE at Grade (g)	Vertical OBE at Grade (g)
100	0.894	0.894	0.298	0.298
90	0.949	0.984	0.316	0.328
80	1.04	1.13	0.347	0.378
70	1.18	1.34	0.394	0.446
60	1.38	1.59	0.461	0.531
50	1.56	1.79	0.519	0.598
45	1.69	1.88	0.564	0.628
40	1.84	1.91	0.612	0.638
35	1.87	1.83	0.622	0.610
30	1.89	1.77	0.629	0.589
25	1.83	1.61	0.610	0.537
20	1.87	1.54	0.622	0.513
15	1.93	1.52	0.643	0.507
12.5	1.96	1.51	0.654	0.504
10	1.90	1.43	0.635	0.476
9	1.81	1.36	0.603	0.452
8	1.67	1.25	0.555	0.417
7	1.48	1.11	0.492	0.369
6	1.26	0.942	0.419	0.314
5	0.994	0.746	0.331	0.249
4	0.679	0.509	0.226	0.170
3	0.402	0.302	0.134	0.101
2.5	0.312	0.224	0.104	0.075
2	0.261	0.185	0.0869	0.0617
1.5	0.206	0.145	0.0686	0.0483
1.25	0.177	0.124	0.0590	0.0414
1	0.147	0.103	0.0491	0.0342

NAPS SUP 3.7-2

Table 3.7.1-216 Site-Dependent SSE and OBE 5% Damping Acceleration Response Spectra at Grade *(continued)*

Frequency (Hz)	Horizontal SSE at Grade (g)	Vertical SSE at Grade (g)	Horizontal OBE at Grade (g)	Vertical OBE at Grade (g)
0.9	0.135	0.0938	0.0451	0.0313
0.8	0.123	0.0848	0.0409	0.0283
0.7	0.110	0.0757	0.0367	0.0252
0.6	0.0969	0.0664	0.0323	0.0221
0.5	0.0834	0.0569	0.0278	0.0190
0.4	0.0694	0.0470	0.0231	0.0157
0.3	0.0548	0.0368	0.0183	0.0123
0.2	0.0302	0.0202	0.0101	0.00673
0.167	0.0210	0.0141	0.00701	0.00469
0.125	0.0118	0.00796	0.00393	0.00265
0.1	0.00850	0.00638	0.00283	0.00213

NAPS SUP 3.7-2

Table 3.7.1-217 Site-Dependent SSE and OBE 5% Damping Pseudo Velocity Response Spectra at Grade

Frequency (Hz)	Horizontal SSE at Grade (in/sec)	Vertical SSE at Grade (in/sec)	Horizontal OBE at Grade (in/sec)	Vertical OBE at Grade (in/sec)
100	0.549	0.549	0.183	0.183
90	0.648	0.673	0.216	0.224
80	0.800	0.872	0.267	0.291
70	1.04	1.17	0.346	0.392
60	1.42	1.63	0.472	0.544
50	1.92	2.21	0.638	0.736
45	2.31	2.57	0.771	0.858
40	2.82	2.94	0.940	0.980
35	3.28	3.21	1.09	1.07
30	3.87	3.62	1.29	1.21
25	4.50	3.96	1.50	1.32
20	5.74	4.74	1.91	1.58
15	7.91	6.23	2.64	2.08
12.5	9.65	7.44	3.22	2.48
10	11.7	8.78	3.90	2.93
9	12.4	9.27	4.12	3.09
8	12.8	9.61	4.27	3.20
7	13.0	9.73	4.32	3.24
6	12.9	9.66	4.29	3.22
5	12.2	9.17	4.08	3.06
4	10.4	7.83	3.48	2.61
3	8.25	6.19	2.75	2.06
2.5	7.67	5.51	2.56	1.84
2	8.01	5.69	2.67	1.90
1.5	8.43	5.94	2.81	1.98
1.25	8.71	6.10	2.90	2.03
1	9.06	6.31	3.02	2.10

NAPS SUP 3.7-2

Table 3.7.1-217 Site-Dependent SSE and OBE 5% Damping Pseudo Velocity Response Spectra at Grade *(continued)*

Frequency (Hz)	Horizontal SSE at Grade (in/sec)	Vertical SSE at Grade (in/sec)	Horizontal OBE at Grade (in/sec)	Vertical OBE at Grade (in/sec)
0.9	9.24	6.41	3.08	2.14
0.8	9.43	6.52	3.14	2.17
0.7	9.66	6.65	3.22	2.22
0.6	9.93	6.81	3.31	2.27
0.5	10.3	6.99	3.42	2.33
0.4	10.7	7.23	3.56	2.41
0.3	11.2	7.55	3.74	2.52
0.2	9.28	6.21	3.09	2.07
0.167	7.75	5.18	2.58	1.73
0.125	5.80	3.92	1.93	1.31
0.1	5.23	3.92	1.74	1.31

NAPS SUP 3.7-2

Table 3.7.1-218 Zero-Lag Cross Correlation Coefficients for the Final Scaled Spectrum Compatible Acceleration Time Histories for the FWSC at Elevation 220 ft

Components	Zero-Lag Cross Correlation Coefficient of Final Matched Time Histories
H1 – H2	0.019
H1 – UP	0.043
H2 – UP	-0.019

NAPS SUP 3.7-2

Table 3.7.1-219 Peak Ground Motion Parameters, Associated Ratios, and Strong Motion Duration Values for the Final Scaled Spectrum Compatible Acceleration Time Histories for the FWSC at Elevation 220 ft

Parameter	H1	H2	UP
PGA (g)	0.556	0.553	0.552
PGV (cm/sec)	18.131	15.590	14.323
PGD (cm)	6.523	9.159	4.814
PGV/PGA (cm/sec/g)	32.614	28.183	25.932
$PGA*PGD/PGV^2$	10.817	20.440	12.707
5-75% Duration Time (sec)	7.560	11.770	7.675
5% Duration Time (sec)	1.095	1.245	0.955
75% Duration Time (sec)	8.655	13.015	8.630
0% Extrapolated Duration Time (sec)	0.555	0.404	0.407
100% Extrapolated Duration Time (sec)	11.355	17.219	11.371
0-100% Extrapolated Duration Time (sec)	10.800	16.814	10.964

NAPS SUP 3.7-2

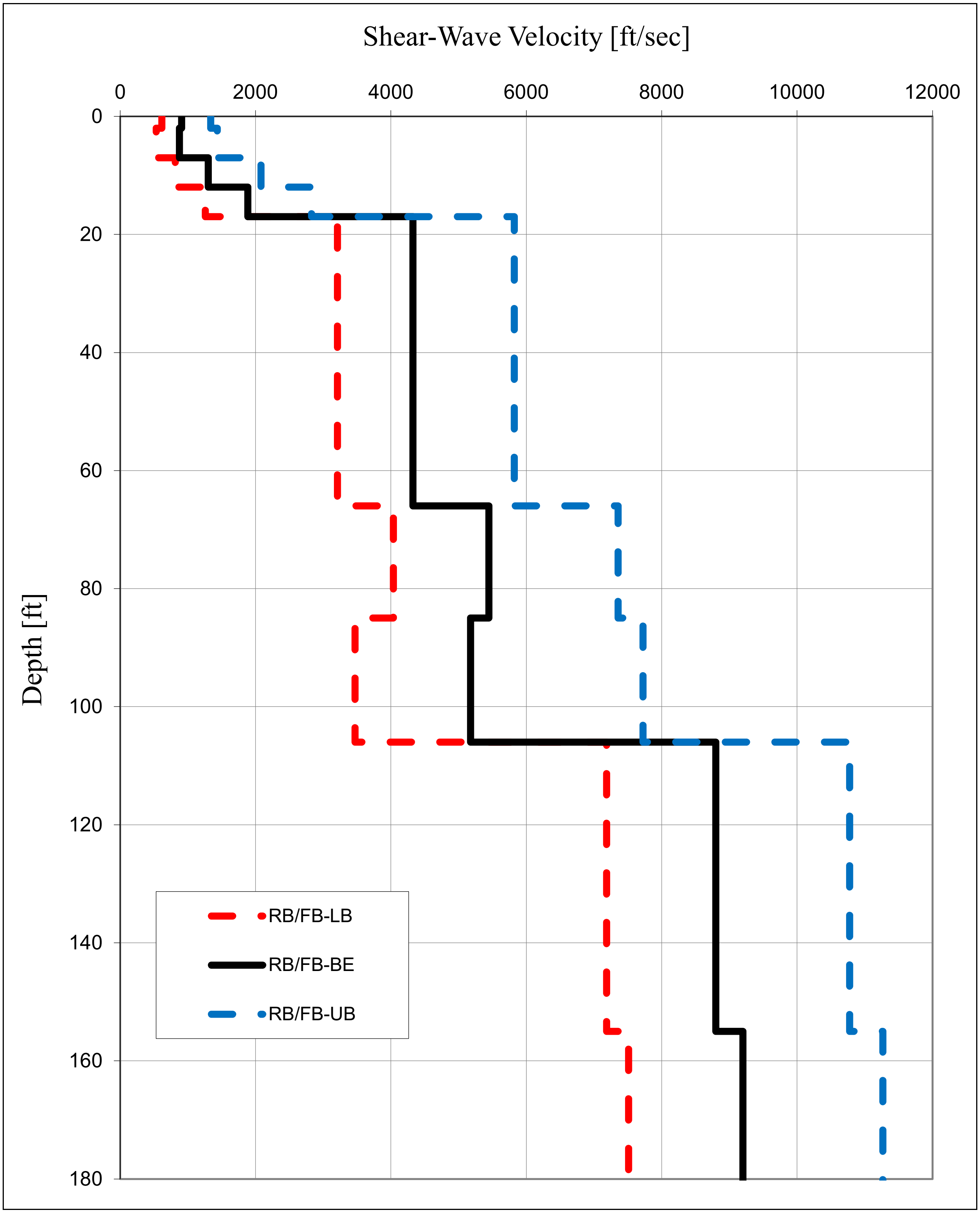
Table 3.7.1-220 **Filter Corners, Peak Ground Motion Parameters, Associated Ratios, and Strong Motion Duration Values for the Selected Seed Input Time History from the NUREG/CR-6728 Database**

Earthquake: **1984 Morgan Hill (M6.2)**
Station: **Gilroy – Gavilan College (R=16.2 km)**

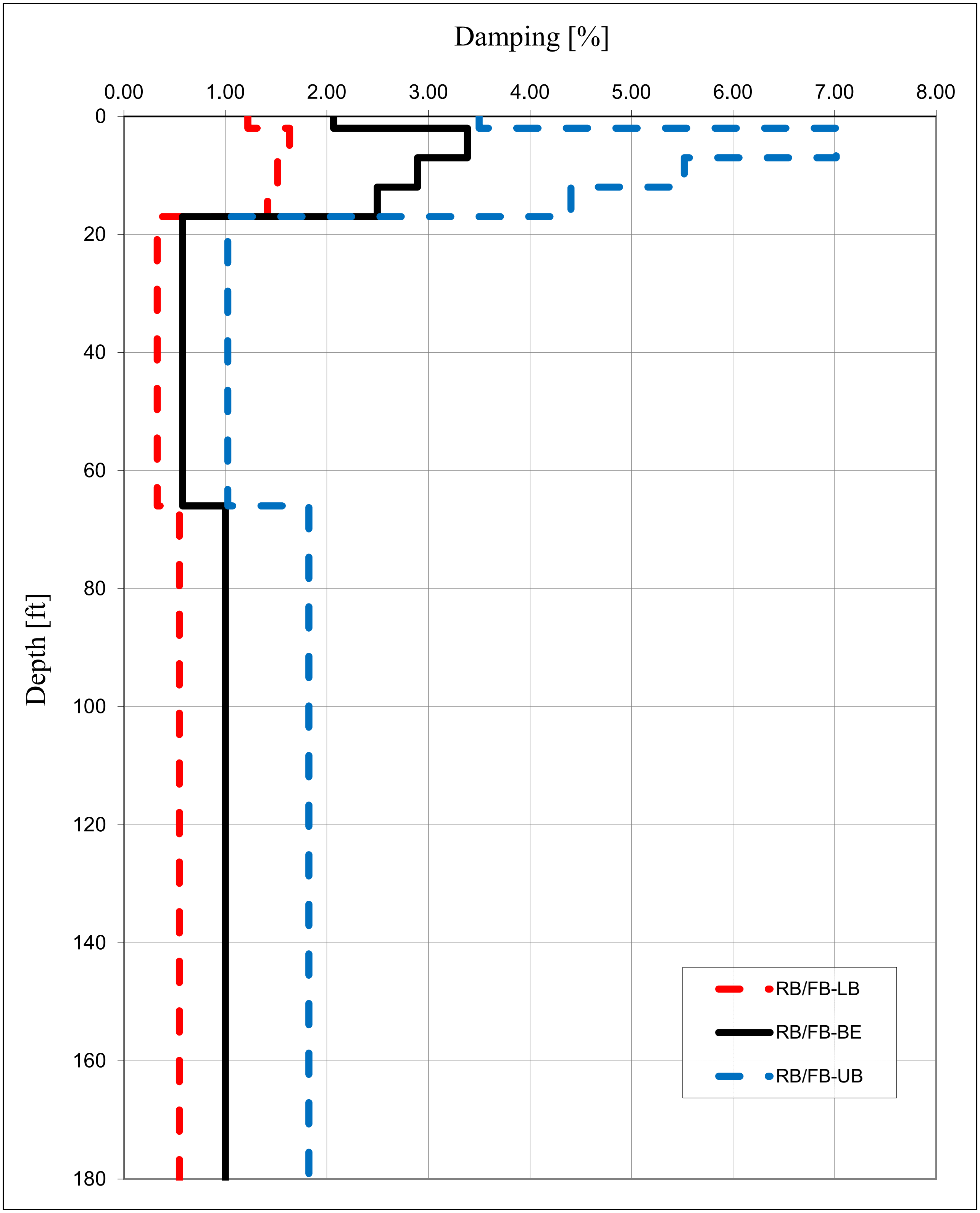
Parameter	**H1**	**H2**	**UP**
High Pass Filter (Hz)	0.10	0.10	0.50
Low Pass Filter (Hz)	30.0	30.0	42.0
PGA (g)	0.276	0.196	0.161
PGV (cm/sec)	5.2	3.6	3.5
PGD (cm)	0.83	0.97	0.55
PGV/PGA (cm/sec/g)	18.8	18.4	21.7
PGA*PGD/PGV^2	8.3	14.4	7.1
Duration Time[a] (sec)	7.1	10.3	5.7

a. Duration time is defined as the acceleration time history interval in which the normalized Arias Intensity is between 5% and 75% of the total normalized value.

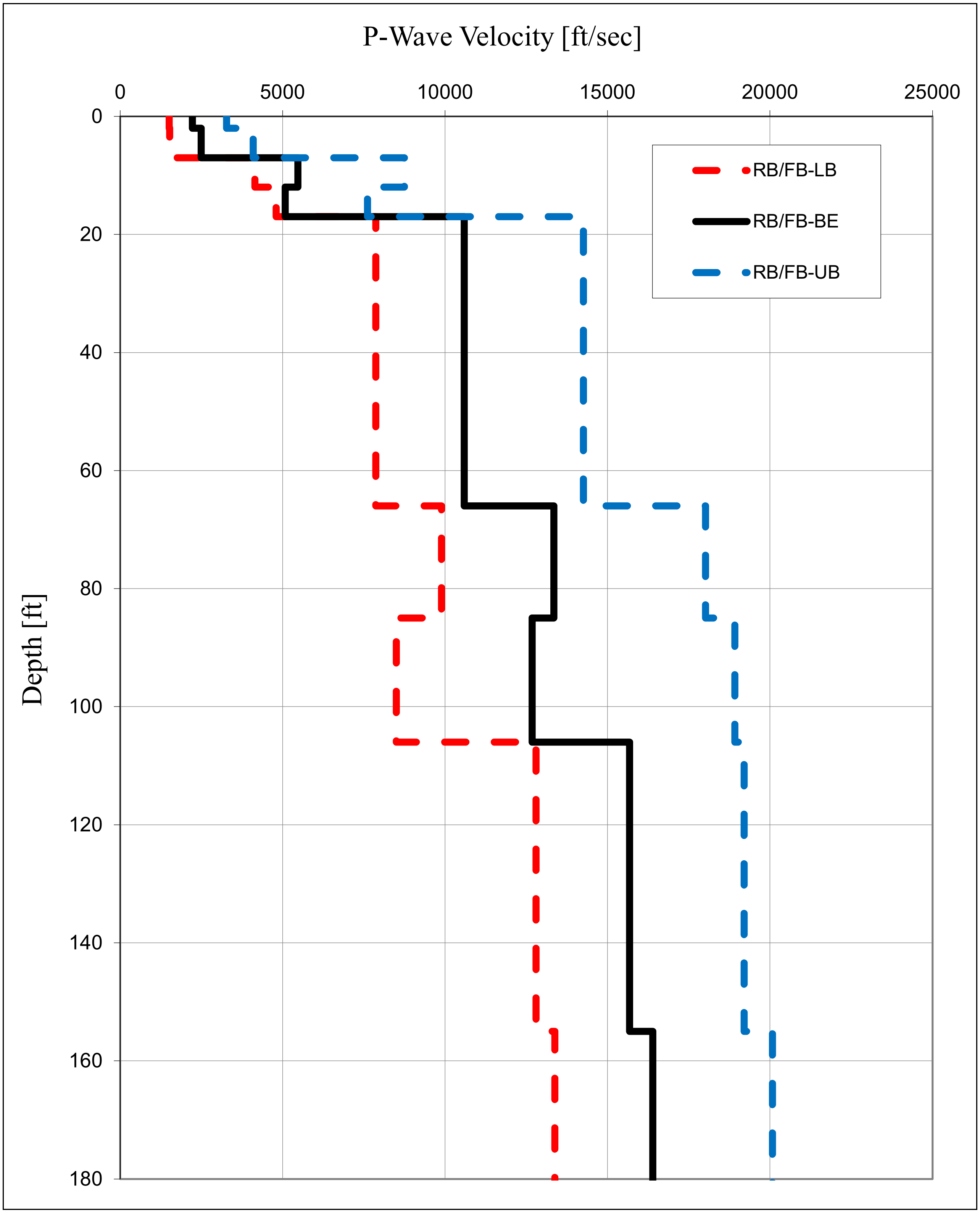
NAPS SUP 3.7-1 **Figure 3.7.1-201 SSI Input Strain Compatible Shear-Wave Velocity Profiles – RB/FB**



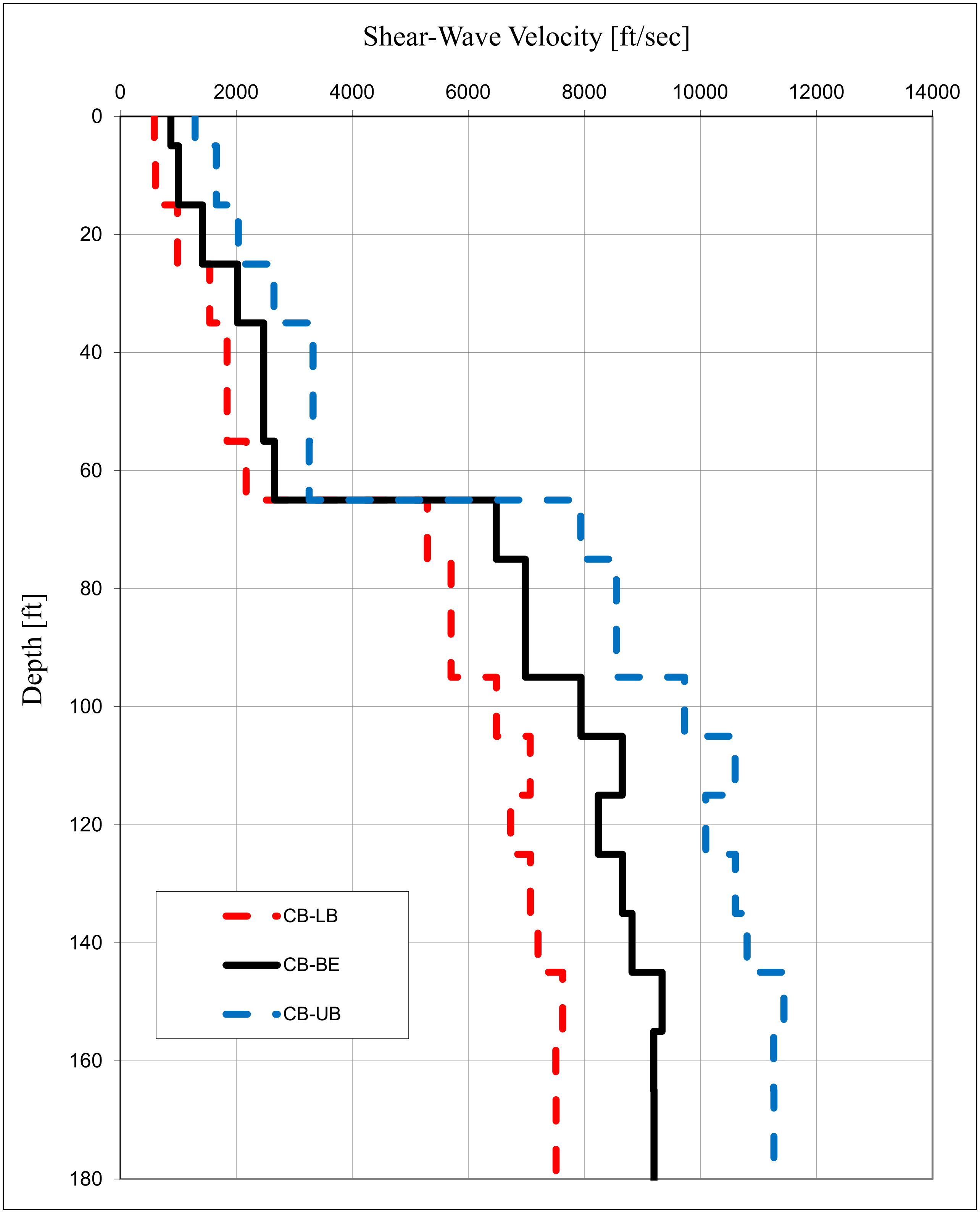
NAPS SUP 3.7-1 **Figure 3.7.1-202 SSI Input Strain Compatible Damping Profiles – RB/FB**



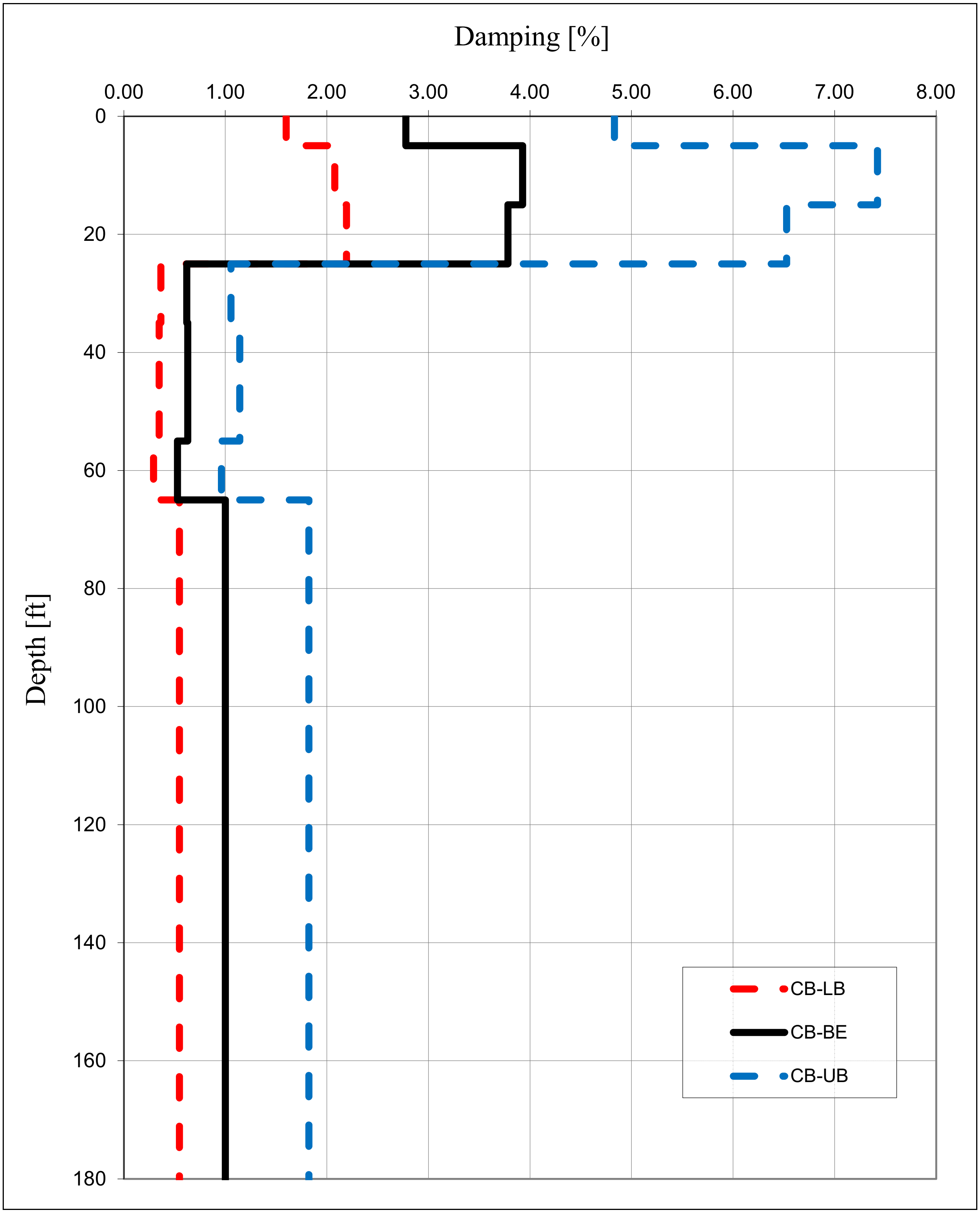
NAPS SUP 3.7-1 **Figure 3.7.1-203 SSI Input Strain Compatible P-Wave Velocity Profiles – RB/FB**



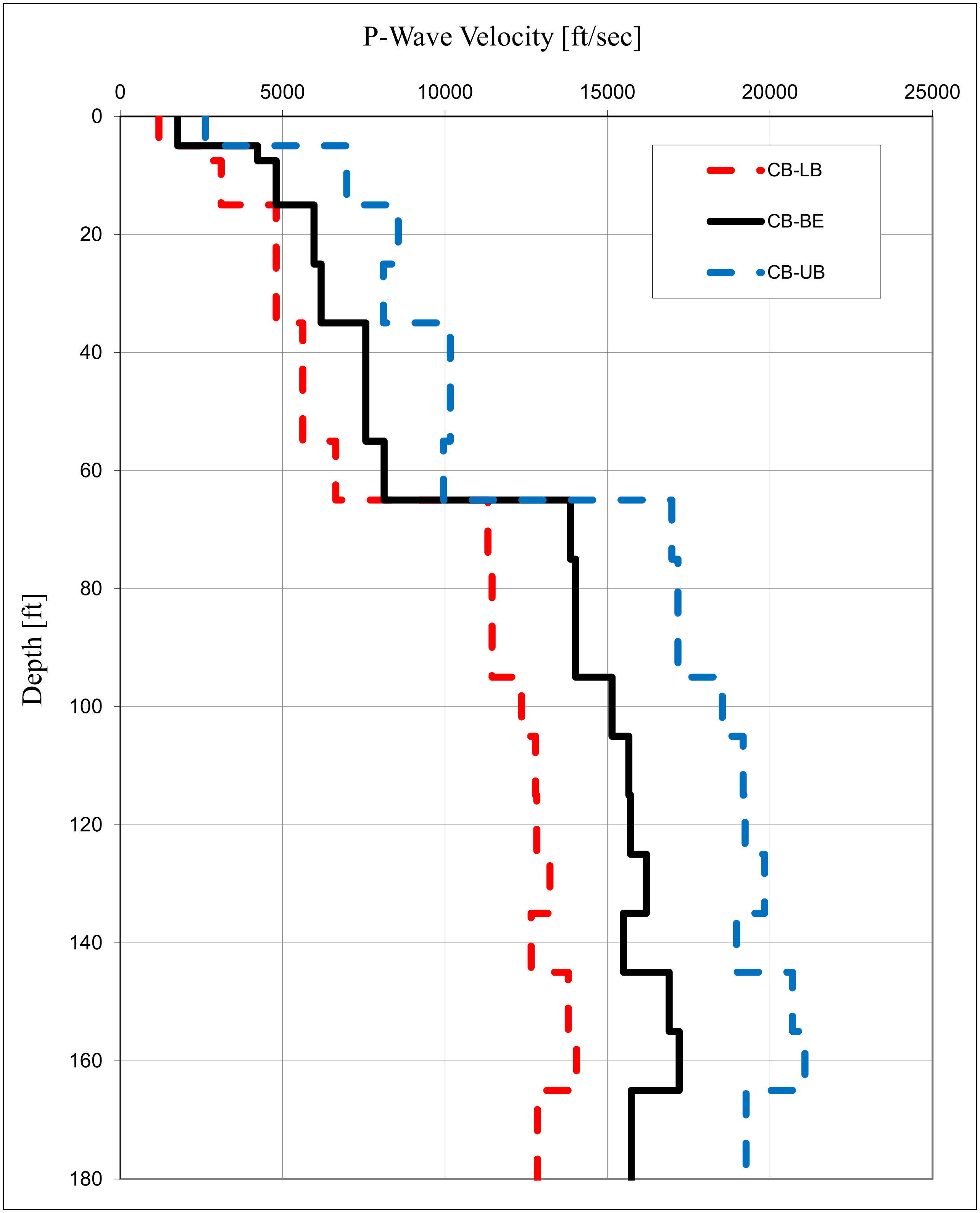
NAPS SUP 3.7-1 **Figure 3.7.1-204 SSI Input Strain Compatible Shear-Wave Velocity Profiles – CB**



NAPS SUP 3.7-1 **Figure 3.7.1-205 SSI Input Strain Compatible Damping Profiles – CB**

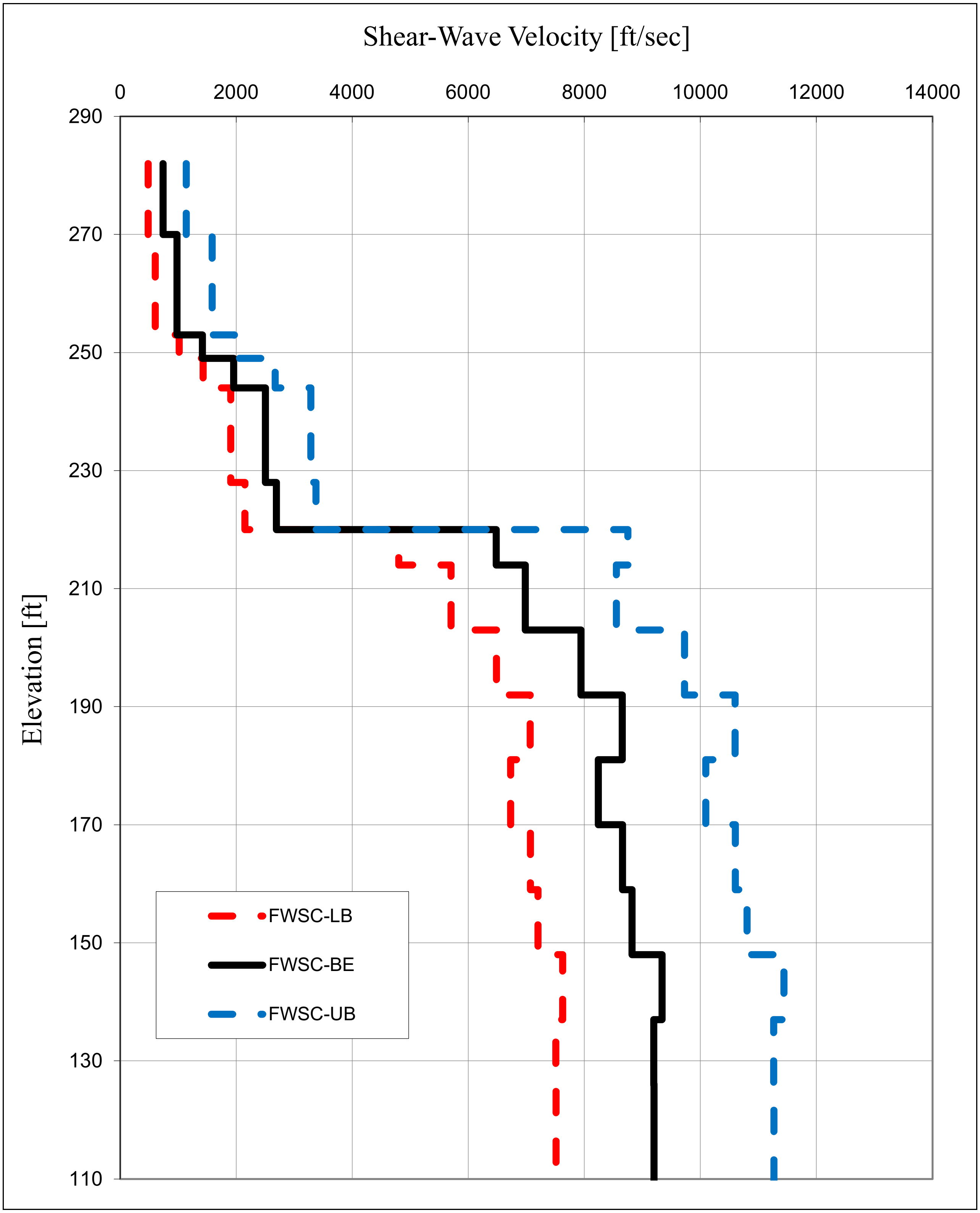


NAPS SUP 3.7-1 **Figure 3.7.1-206 SSI Input Strain Compatible P-Wave Velocity Profiles – CB**



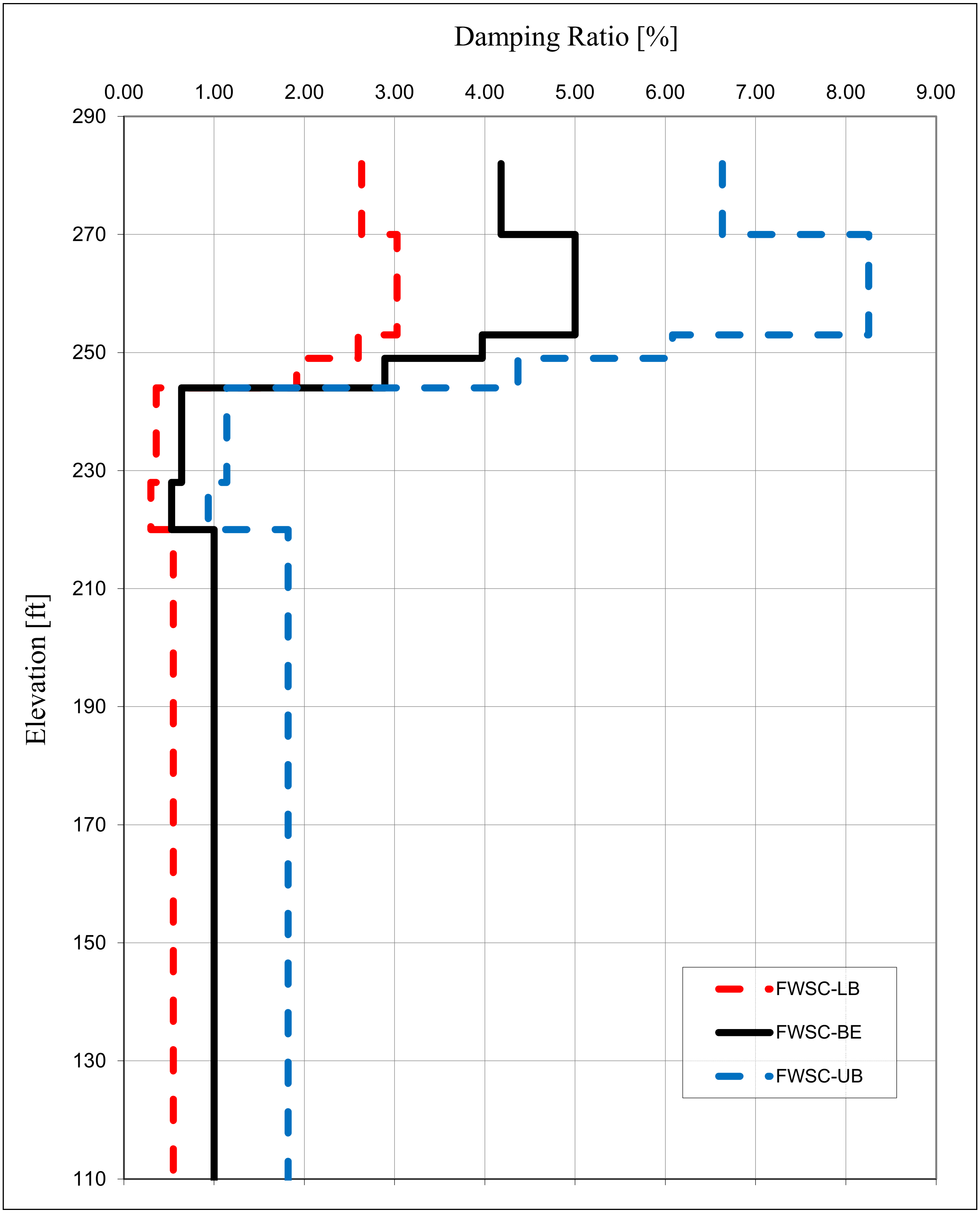
NAPS SUP 3.7-1

Figure 3.7.1-207 SSI Input Strain Compatible Shear-Wave Velocity Profiles – FWSC



NAPS SUP 3.7-1

Figure 3.7.1-208 SSI Input Strain Compatible Damping Profiles – FWSC



NAPS SUP 3.7-1 **Figure 3.7.1-209 SSI Input Strain Compatible P-Wave Velocity Profiles – FWSC**

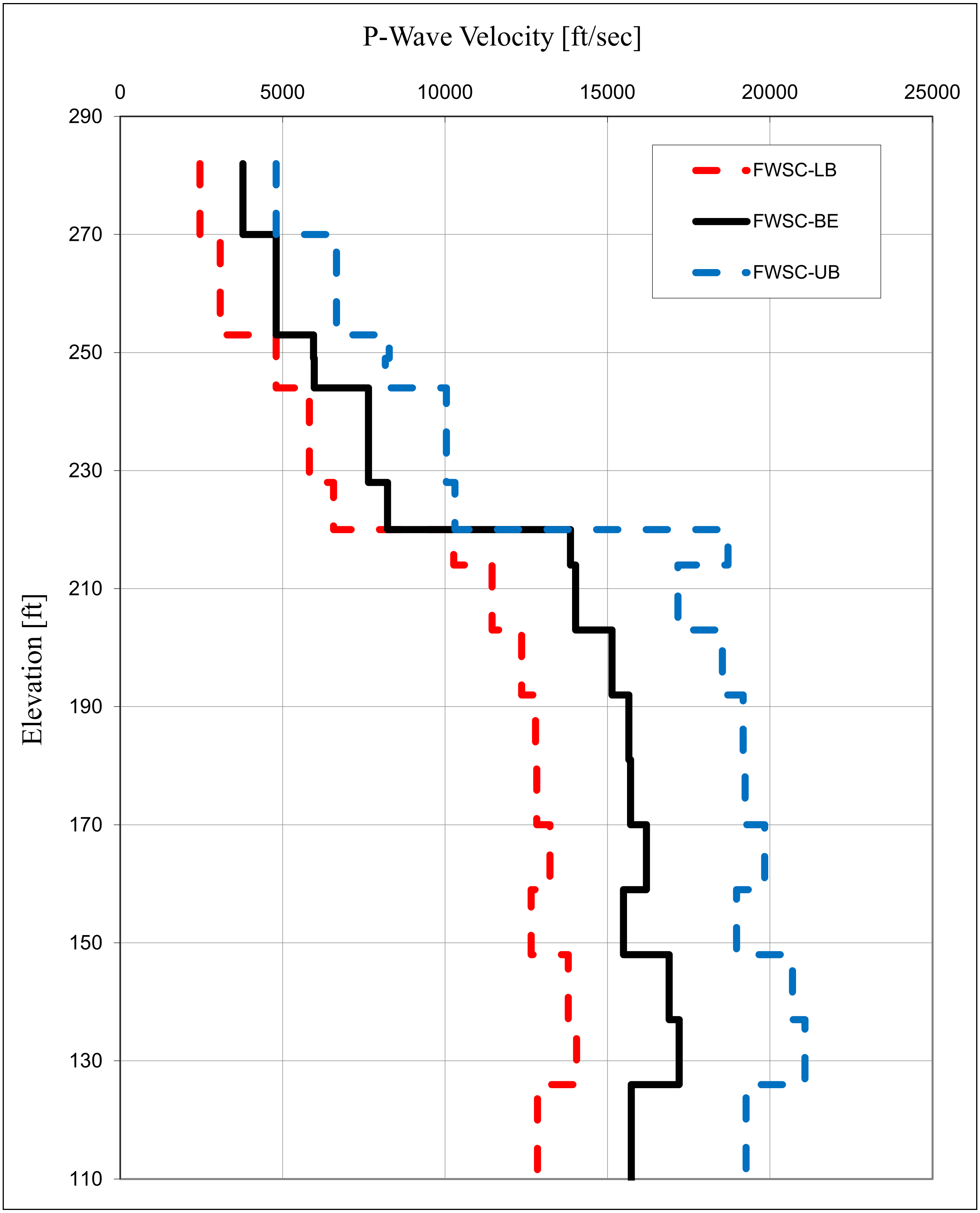
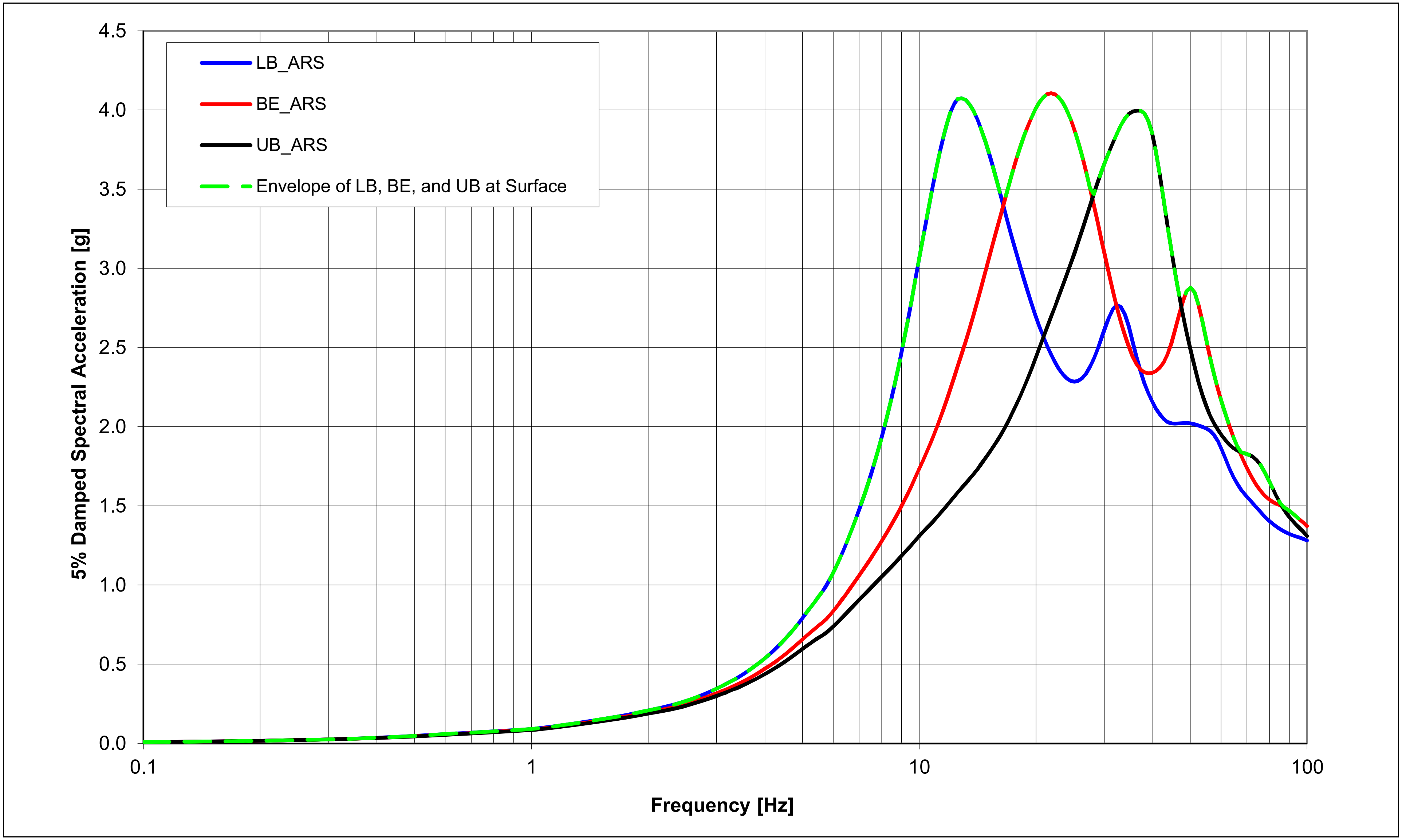
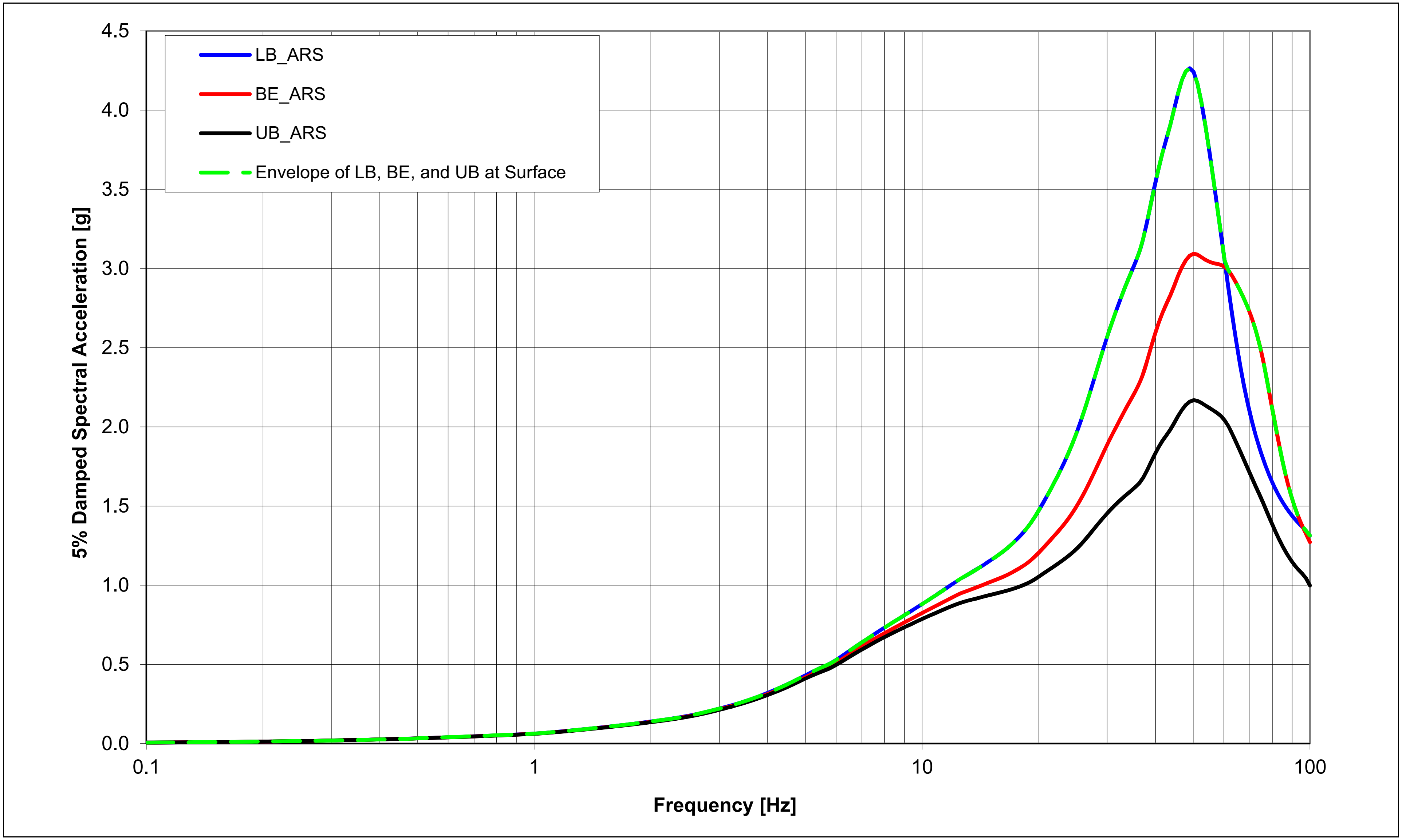


Figure 3.7.1-210 Envelope of Horizontal FIRS Propagated to the Ground Surface through Full Column SSI Input Profiles – RB/FB

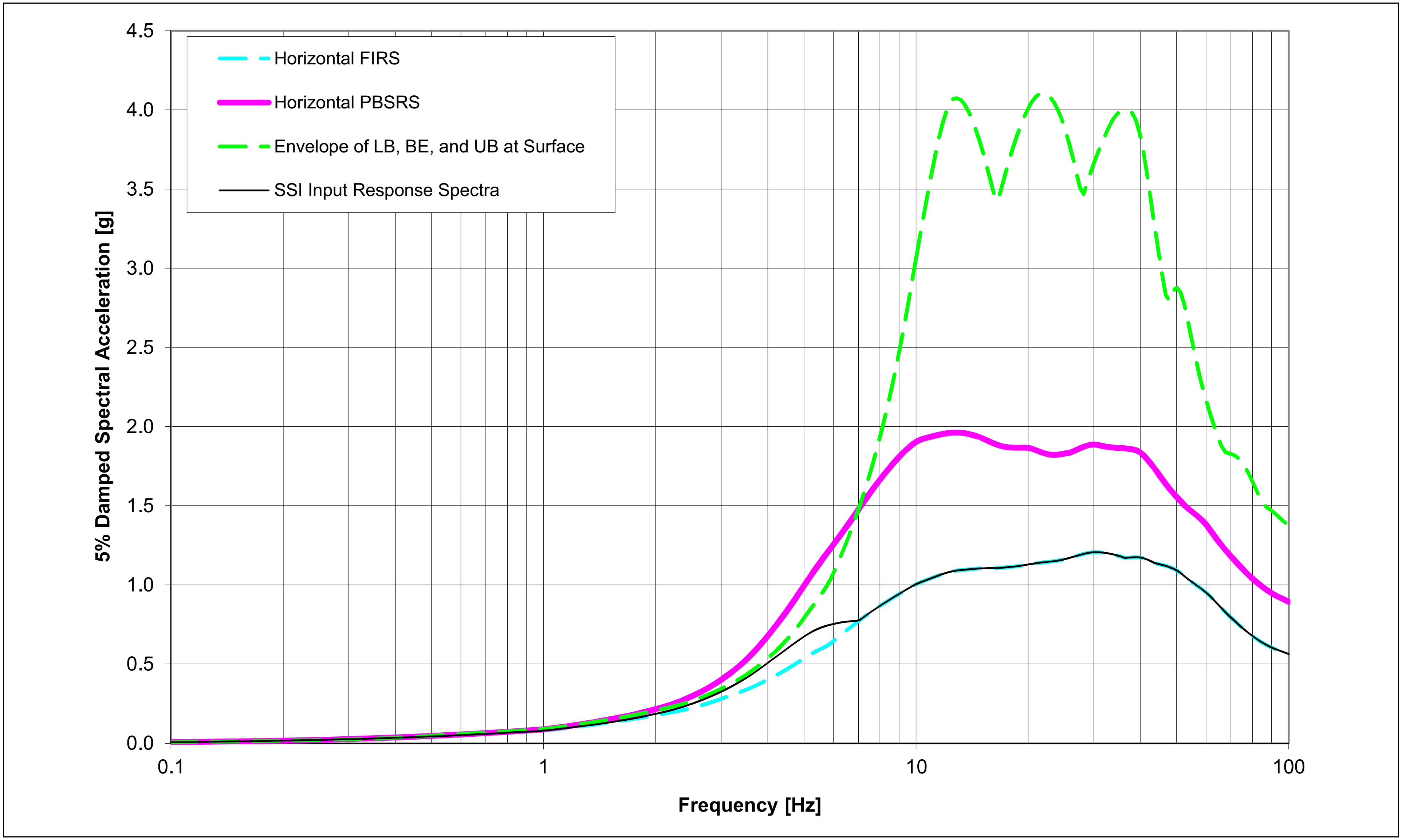


NAPS SUP 3.7-1

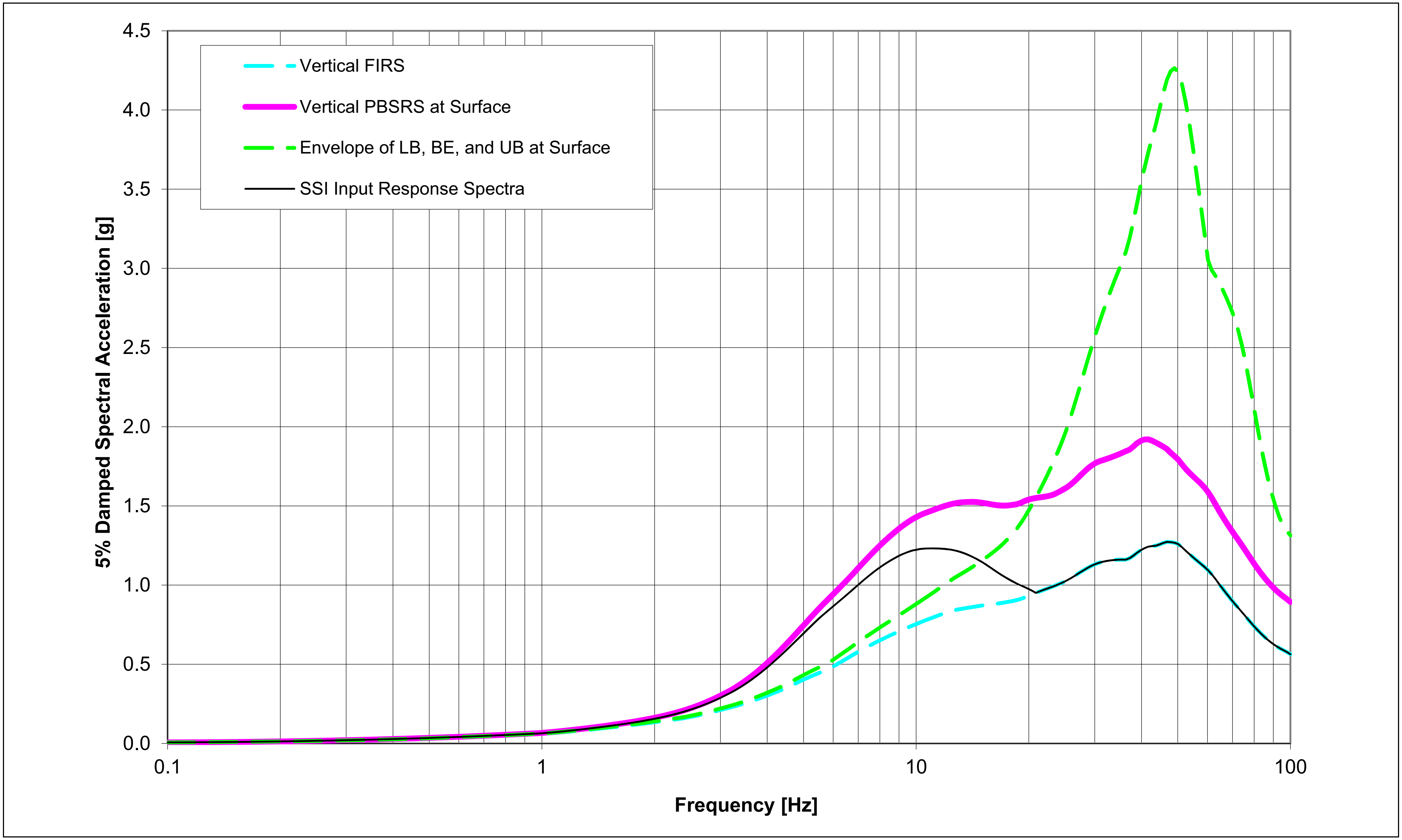
Figure 3.7.1-211 Envelope of Vertical FIRS Propagated to the Ground Surface through Full Column SSI Input Profiles – RB/FB



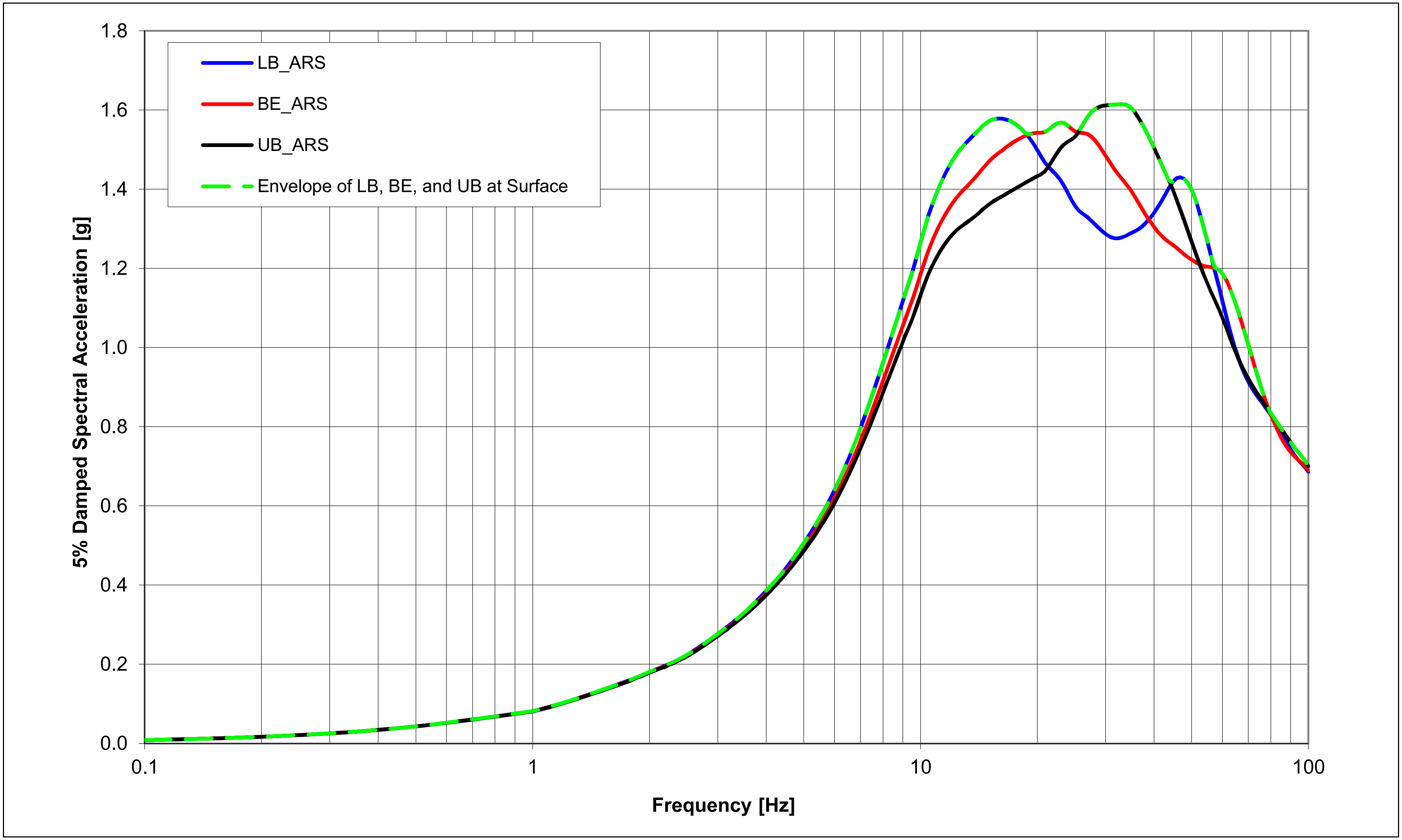
NAPS SUP 3.7-1 **Figure 3.7.1-212 NEI Check and SSI Input Response Spectra for Horizontal Full Column FIRS – RB/FB**



NAPS SUP 3.7-1 **Figure 3.7.1-213 NEI Check and SSI Input Response Spectra for Vertical Full Column FIRS – RB/FB**

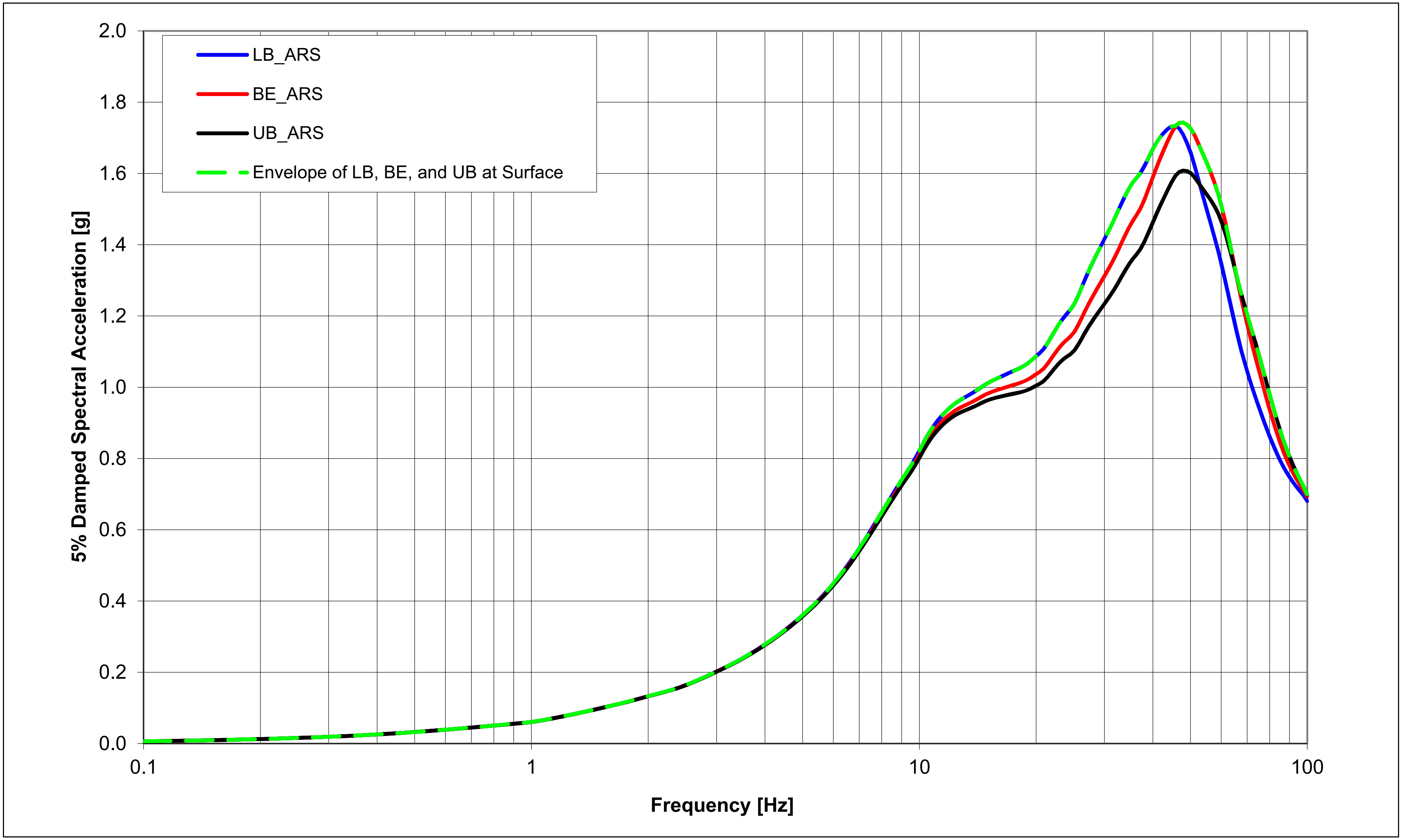


NAPS SUP 3.7-1 **Figure 3.7.1-214 Envelope of Horizontal FIRS Propagated to the Ground Surface through Partial Column SSI Input Profiles – RB/FB**

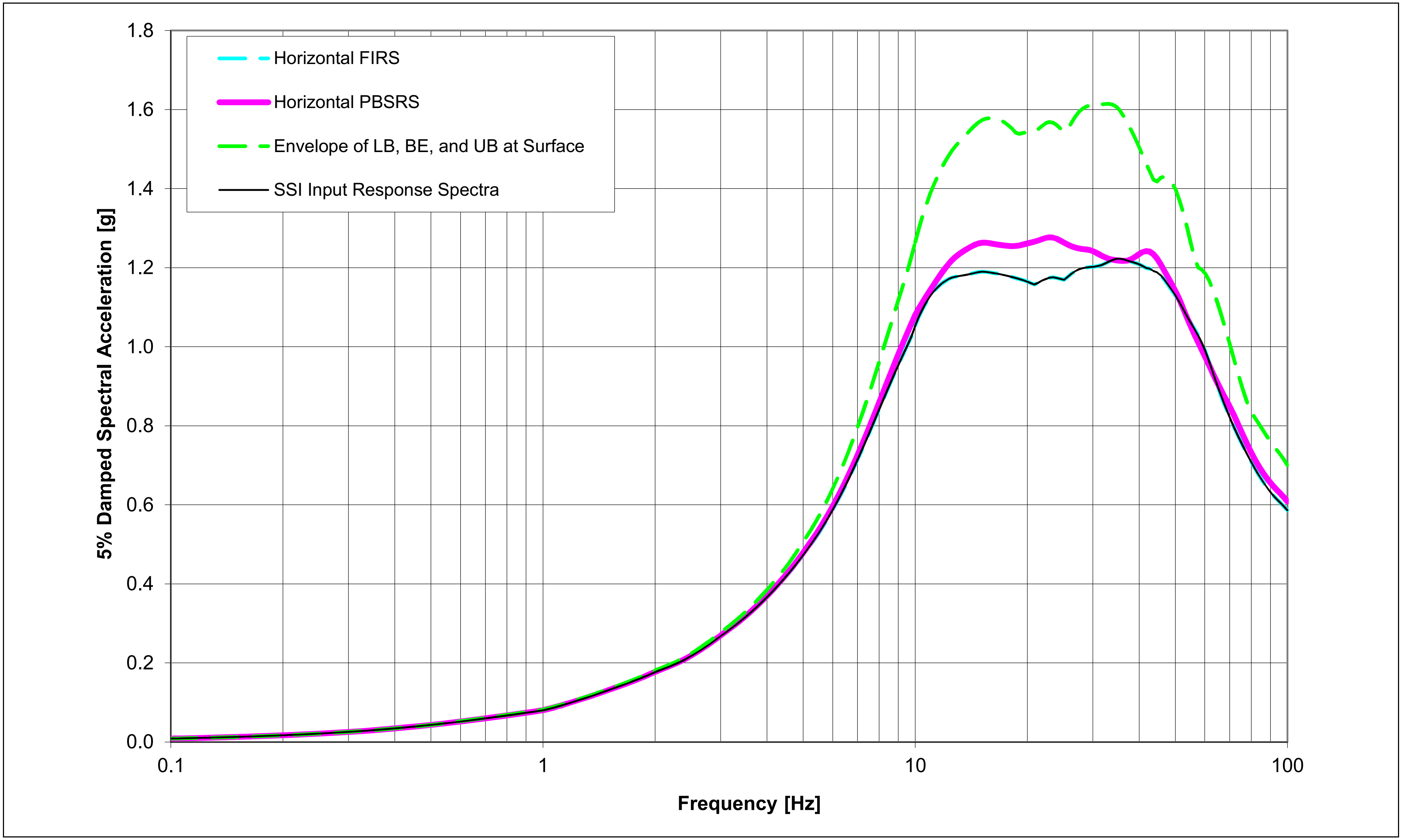


NAPS SUP 3.7-1

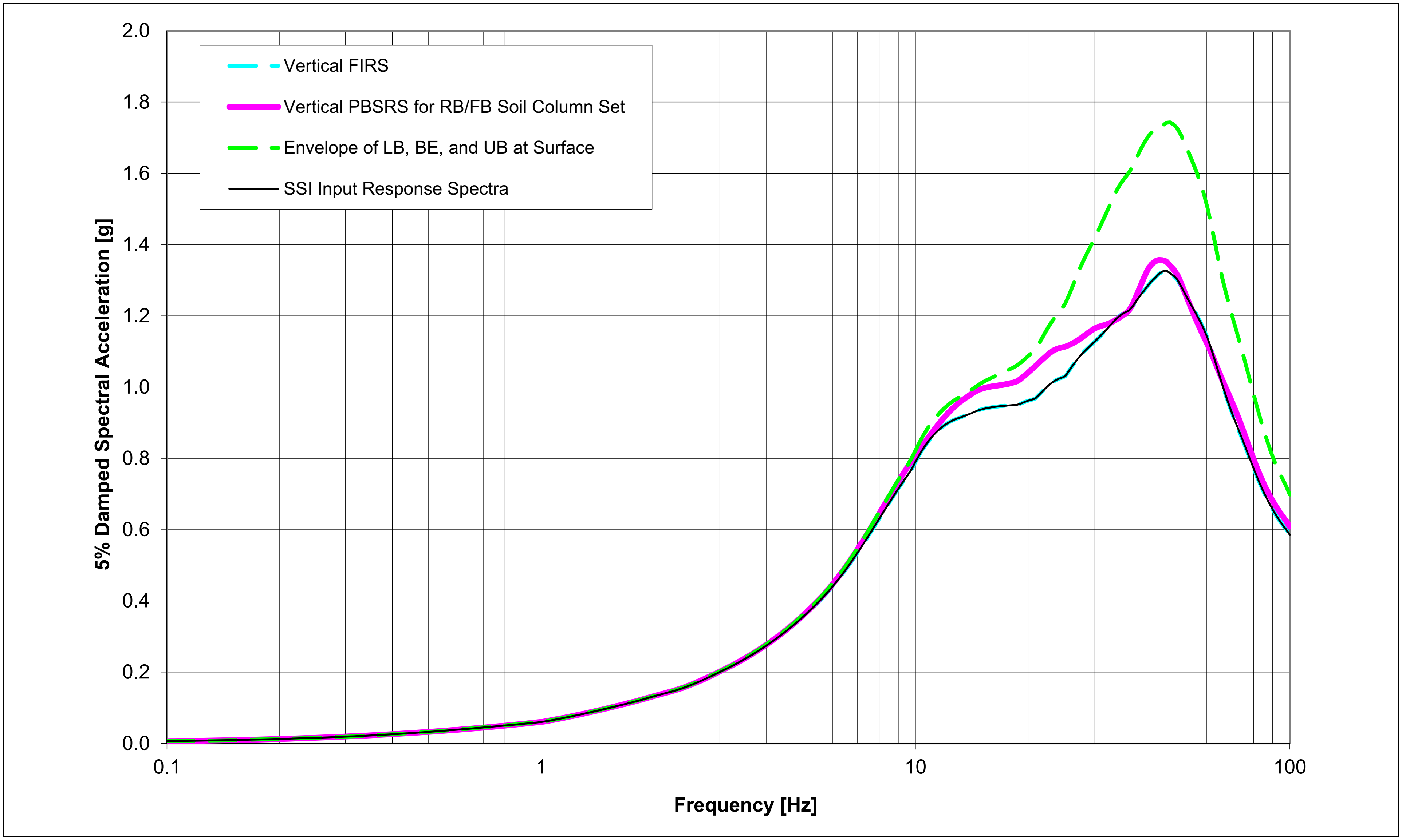
Figure 3.7.1-215 Envelope of Vertical FIRS Propagated to the Ground Surface through Partial Column SSI Input Profiles – RB/FB



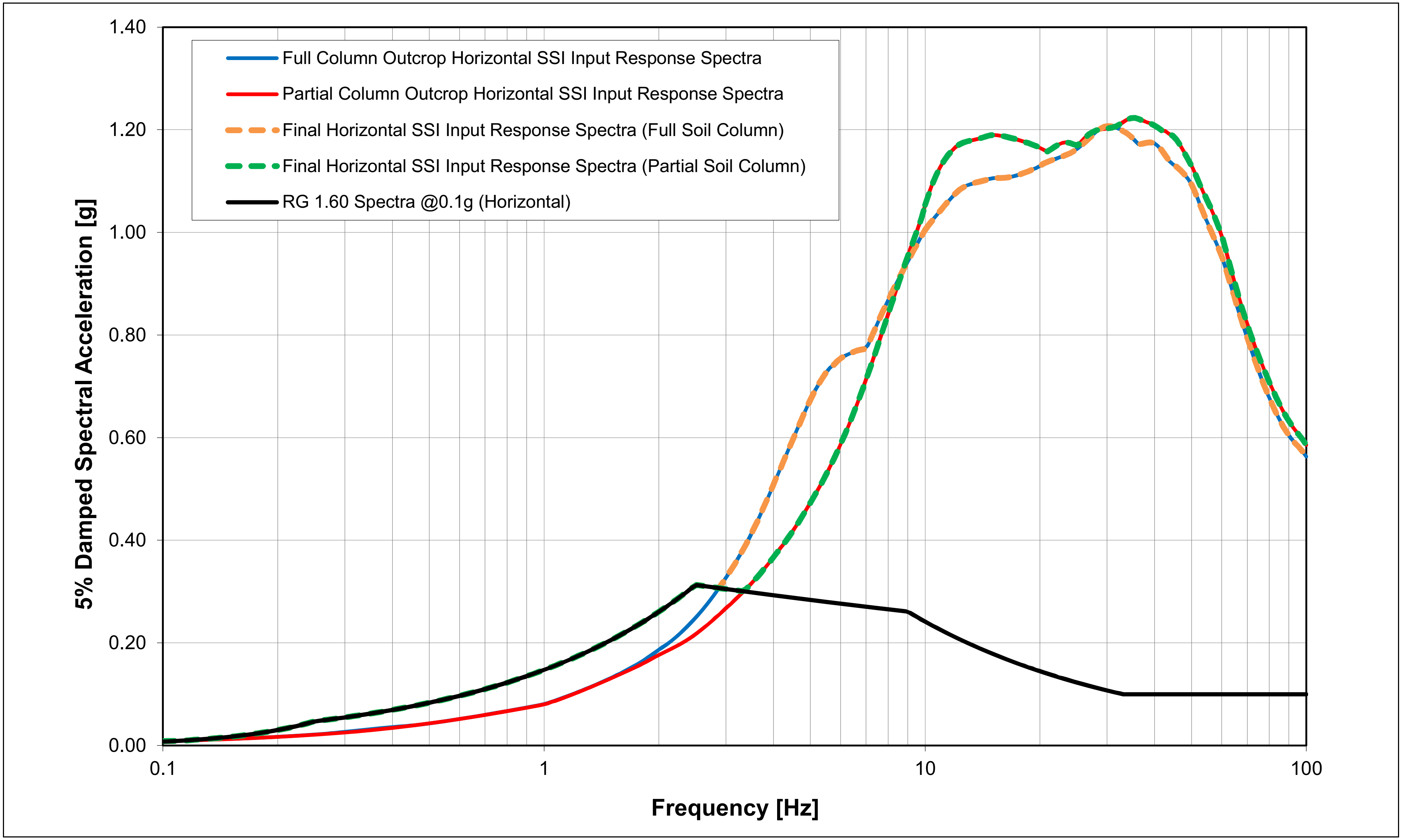
NAPS SUP 3.7-1 **Figure 3.7.1-216 NEI Check and SSI Input Response Spectra for Horizontal Partial Column FIRS – RB/FB**



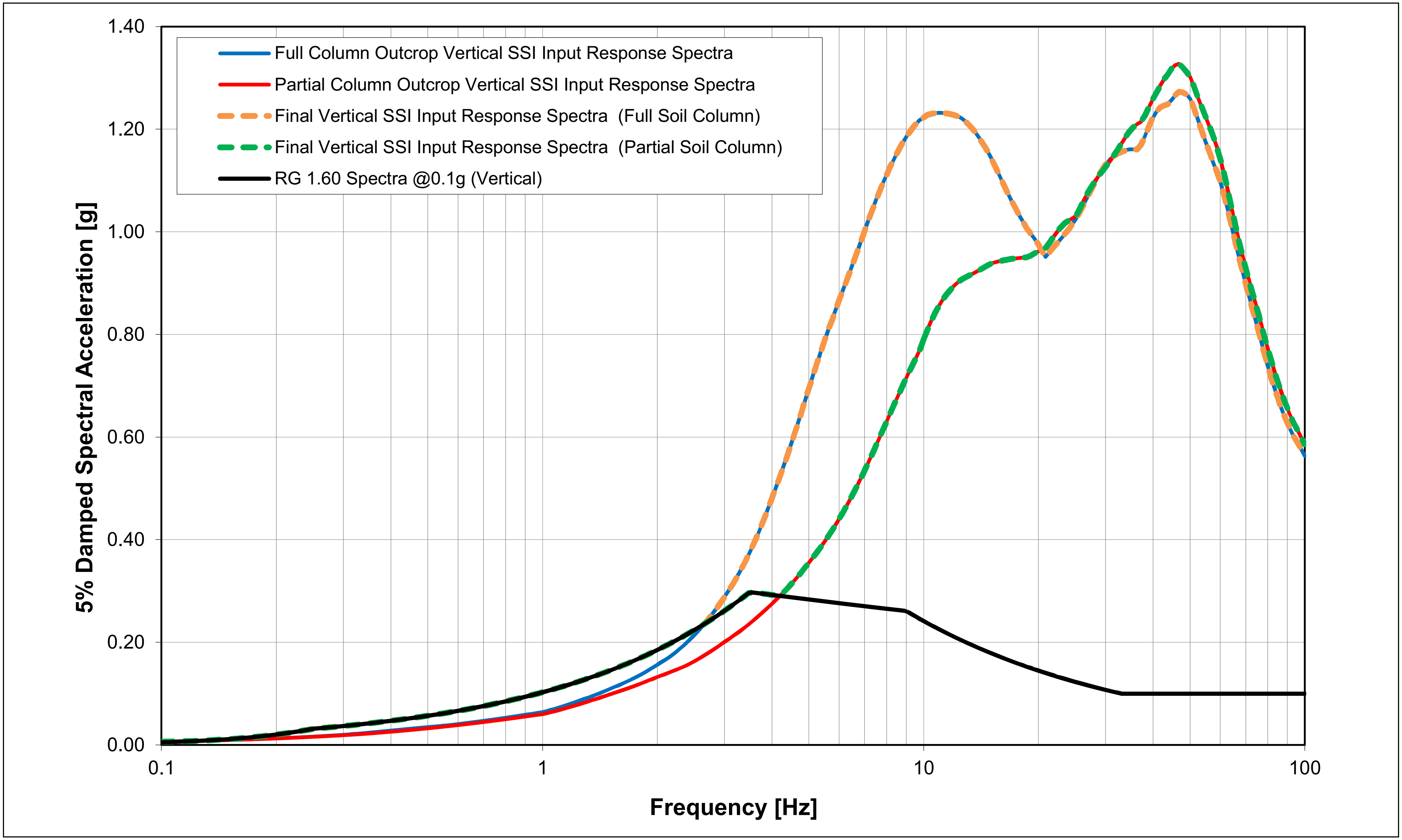
NAPS SUP 3.7-1 **Figure 3.7.1-217 NEI Check and SSI Input Response Spectra for Vertical Partial Column FIRS – RB/FB**



NAPS SUP 3.7-1 **Figure 3.7.1-218 Development of 5% Damped Final Horizontal SSI Input Response Spectra for RB/FB**



NAPS SUP 3.7-1 **Figure 3.7.1-219 Development of 5% Damped Final Vertical SSI Input Response Spectra for RB/FB**



NAPS SUP 3.7-1 **Figure 3.7.1-220 5% Damped Final SSI Input Response Spectra for RB/FB**

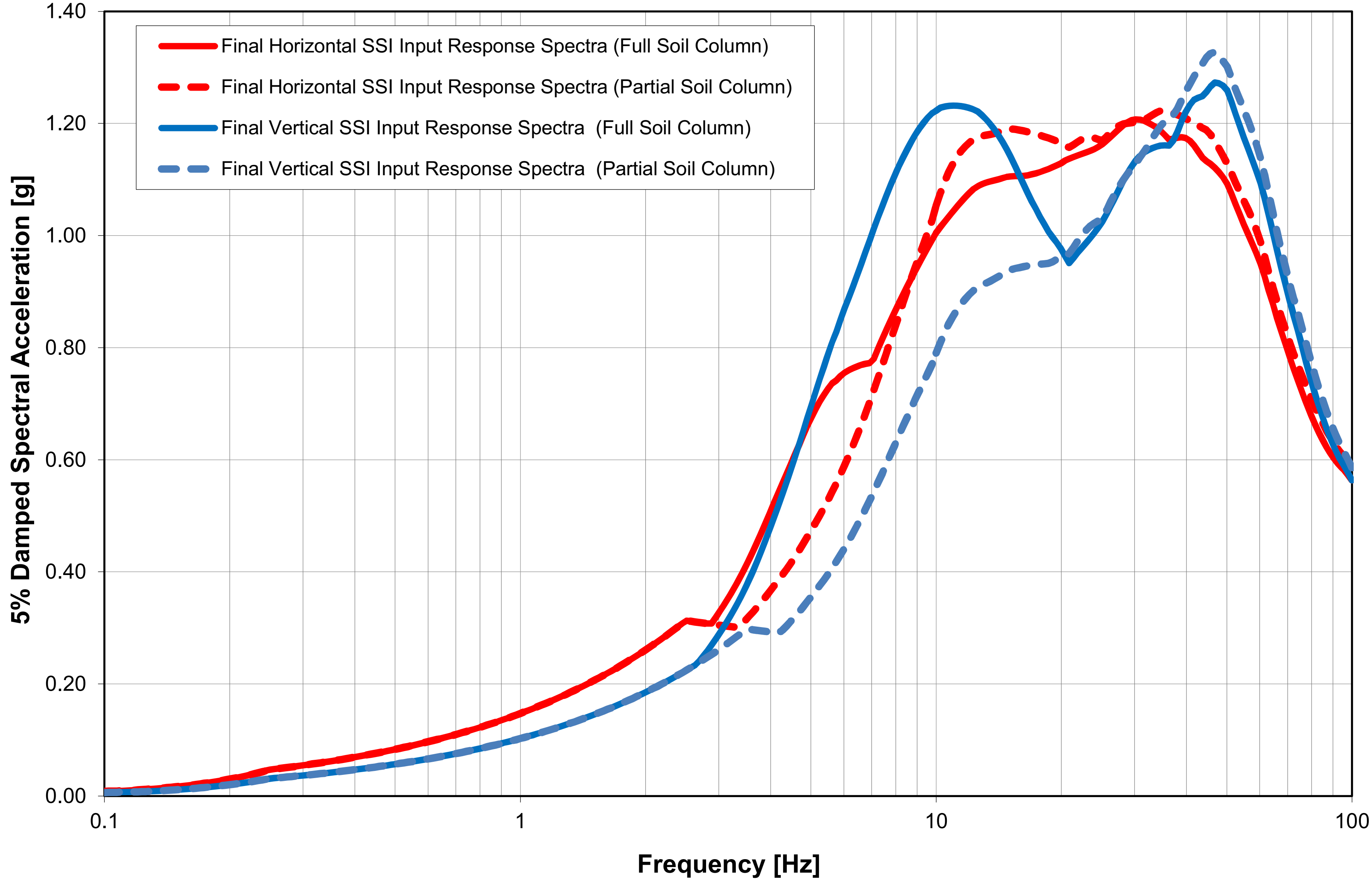
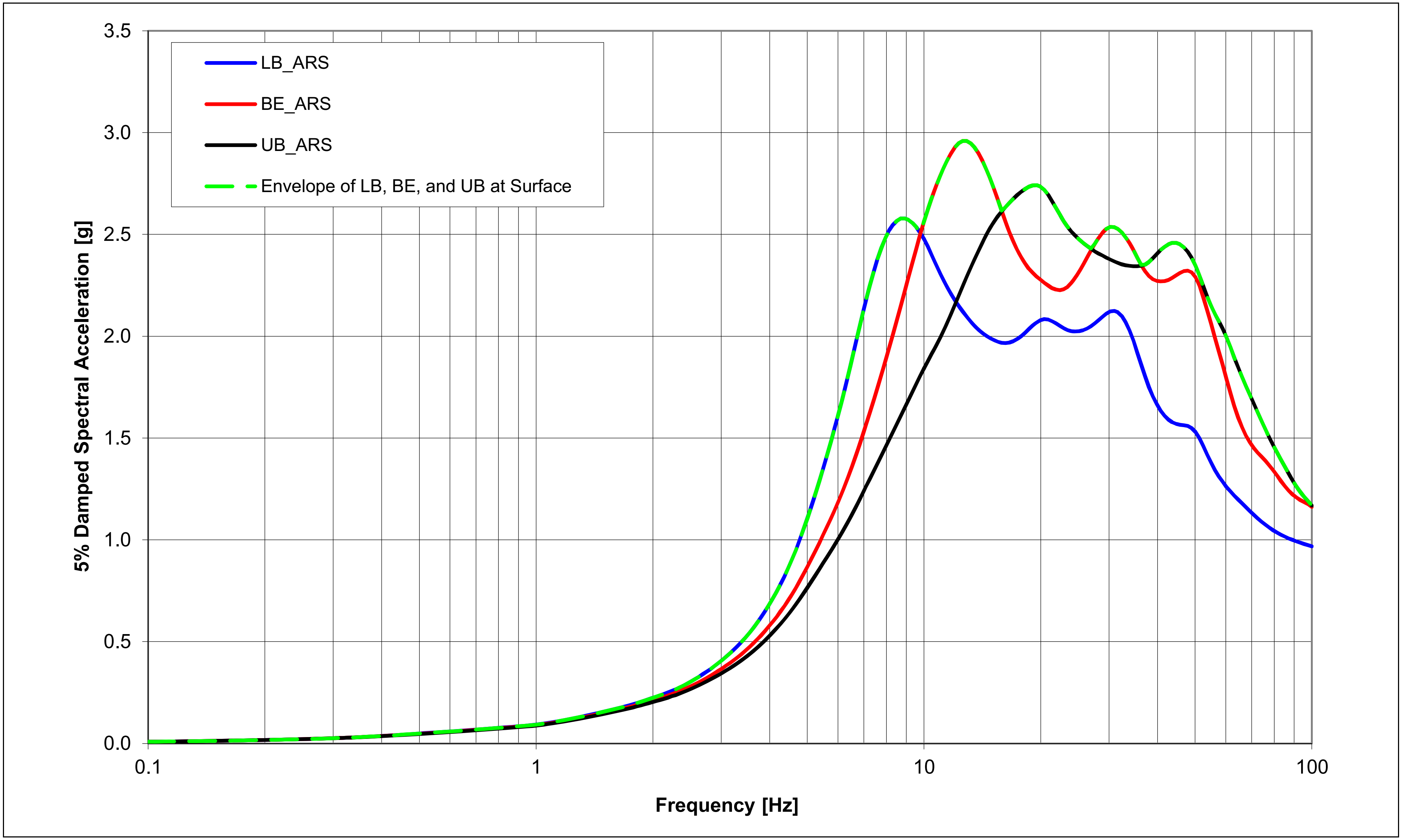
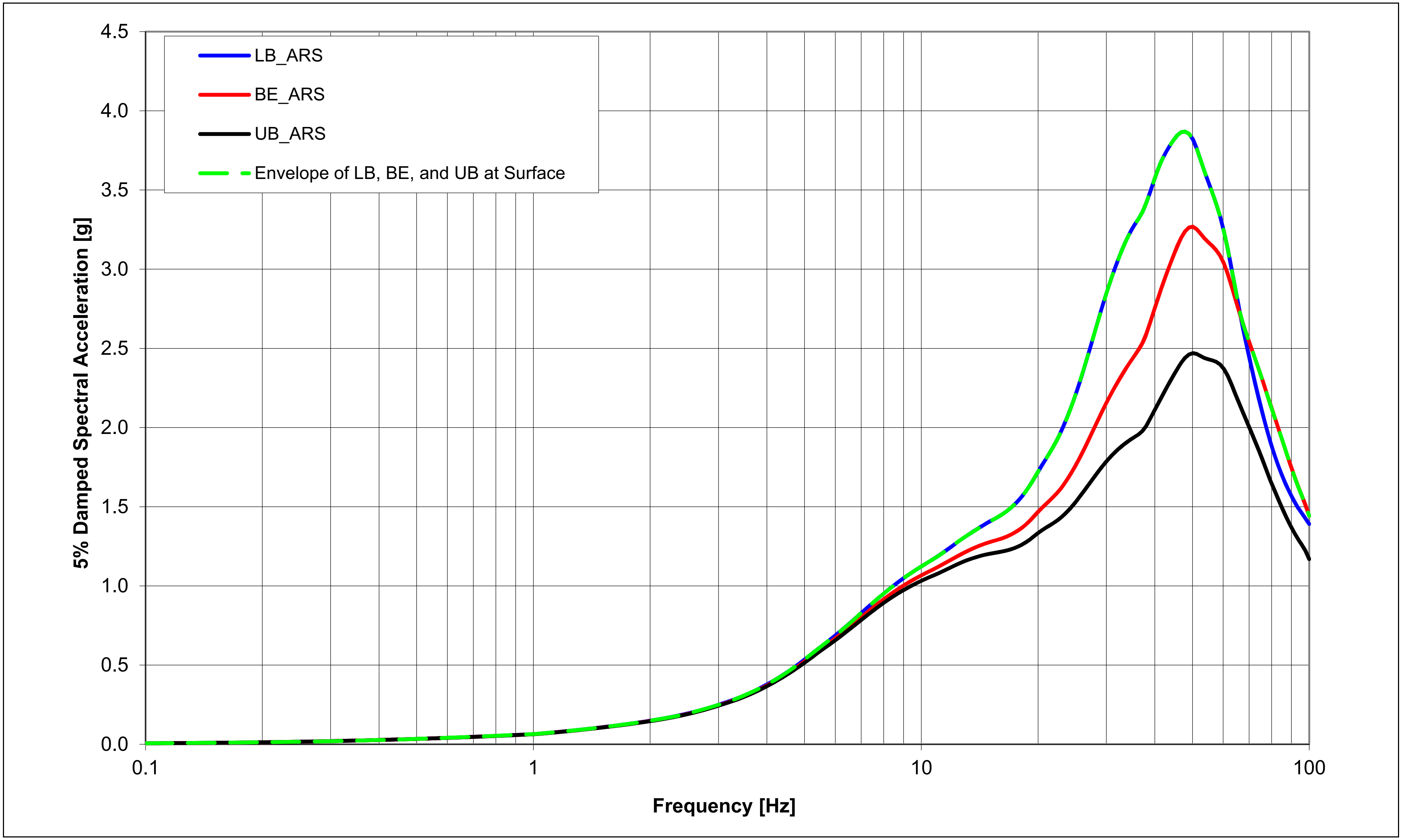


Figure 3.7.1-221 Envelope of Horizontal FIRS Propagated to the Ground Surface through Full Column SSI Input Profiles – CB

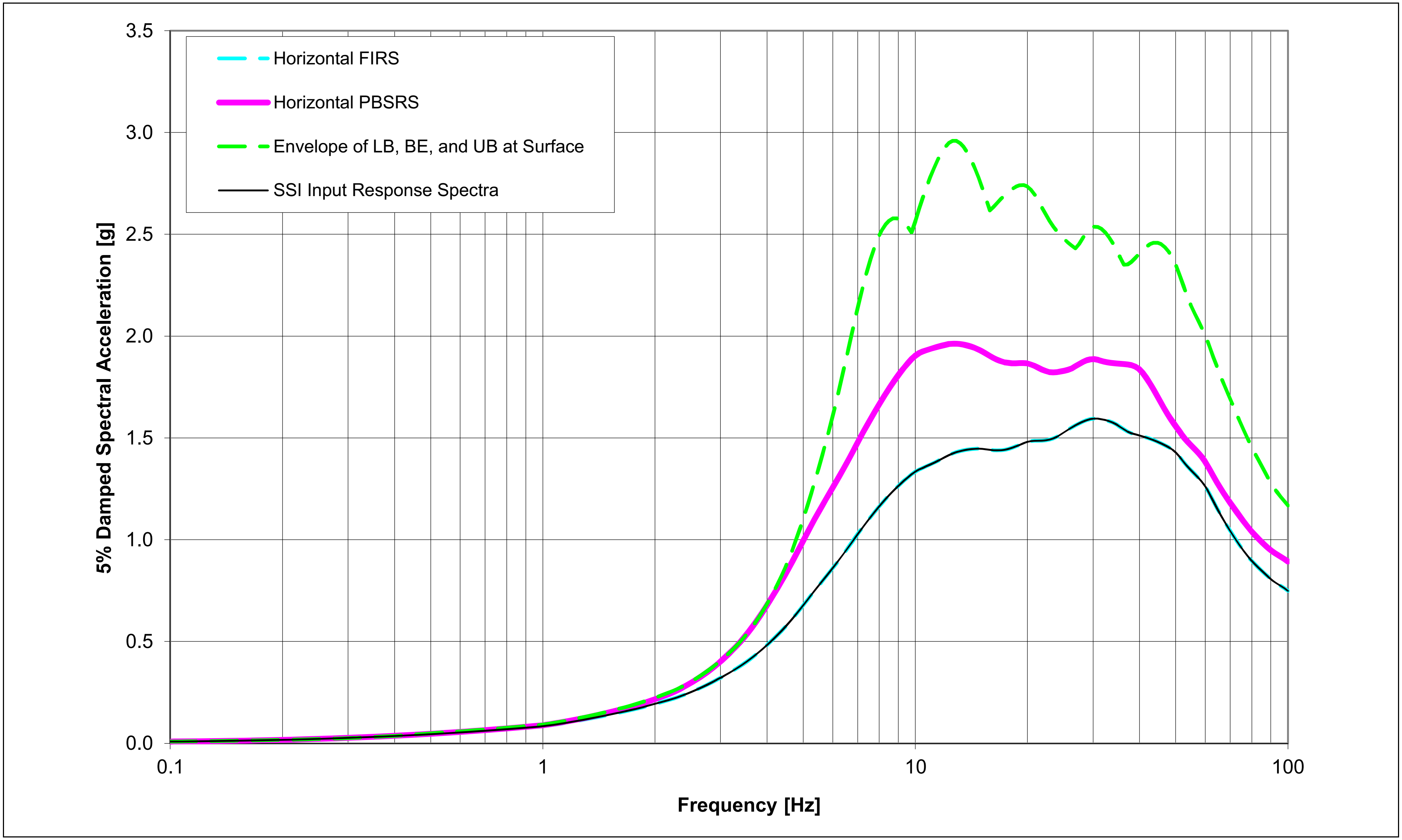


NAPS SUP 3.7-1

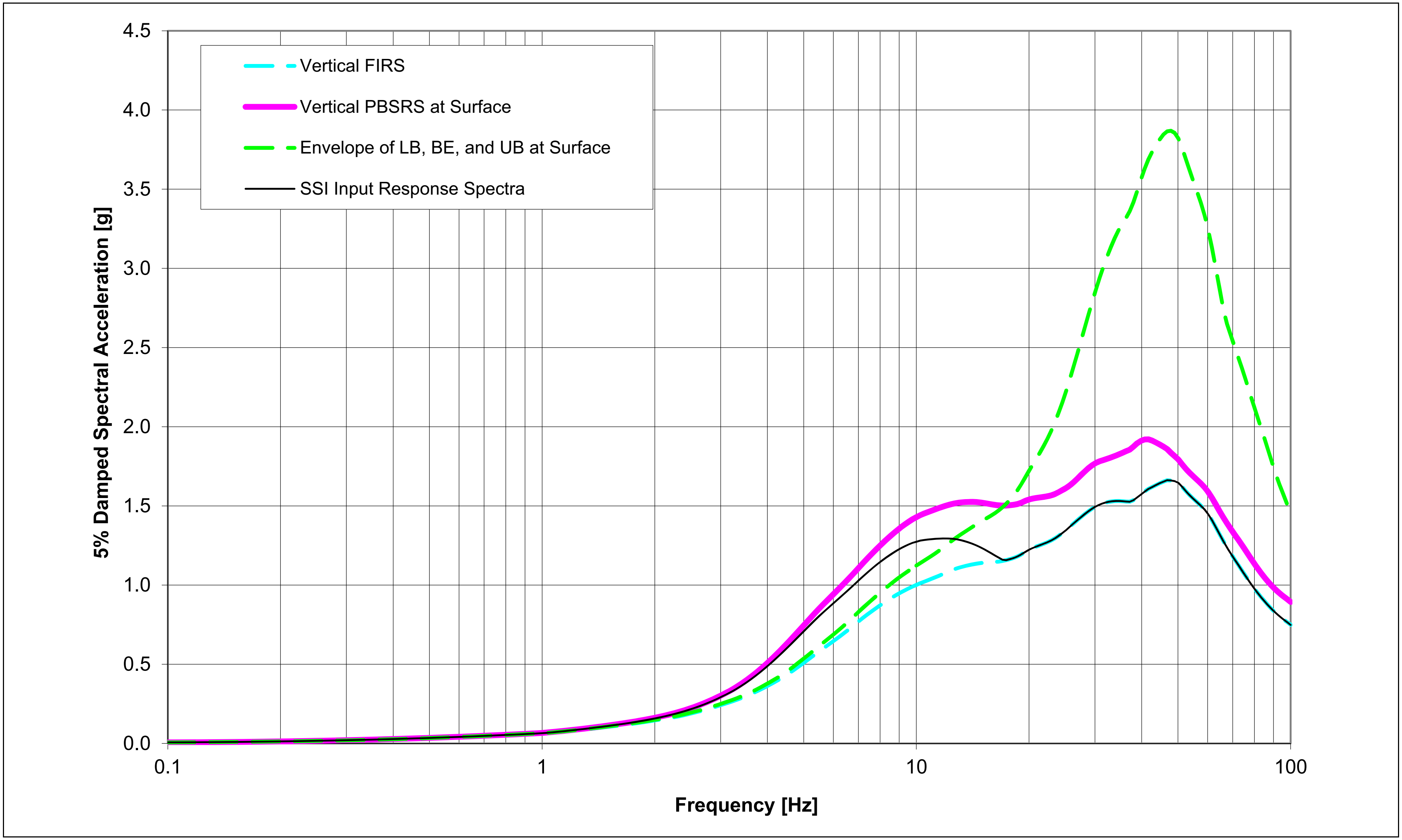
Figure 3.7.1-222 Envelope of Vertical FIRS Propagated to the Ground Surface through Full Column SSI Input Profiles – CB



NAPS SUP 3.7-1 **Figure 3.7.1-223 NEI Check and SSI Input Response Spectra for Horizontal Full Column FIRS – CB**

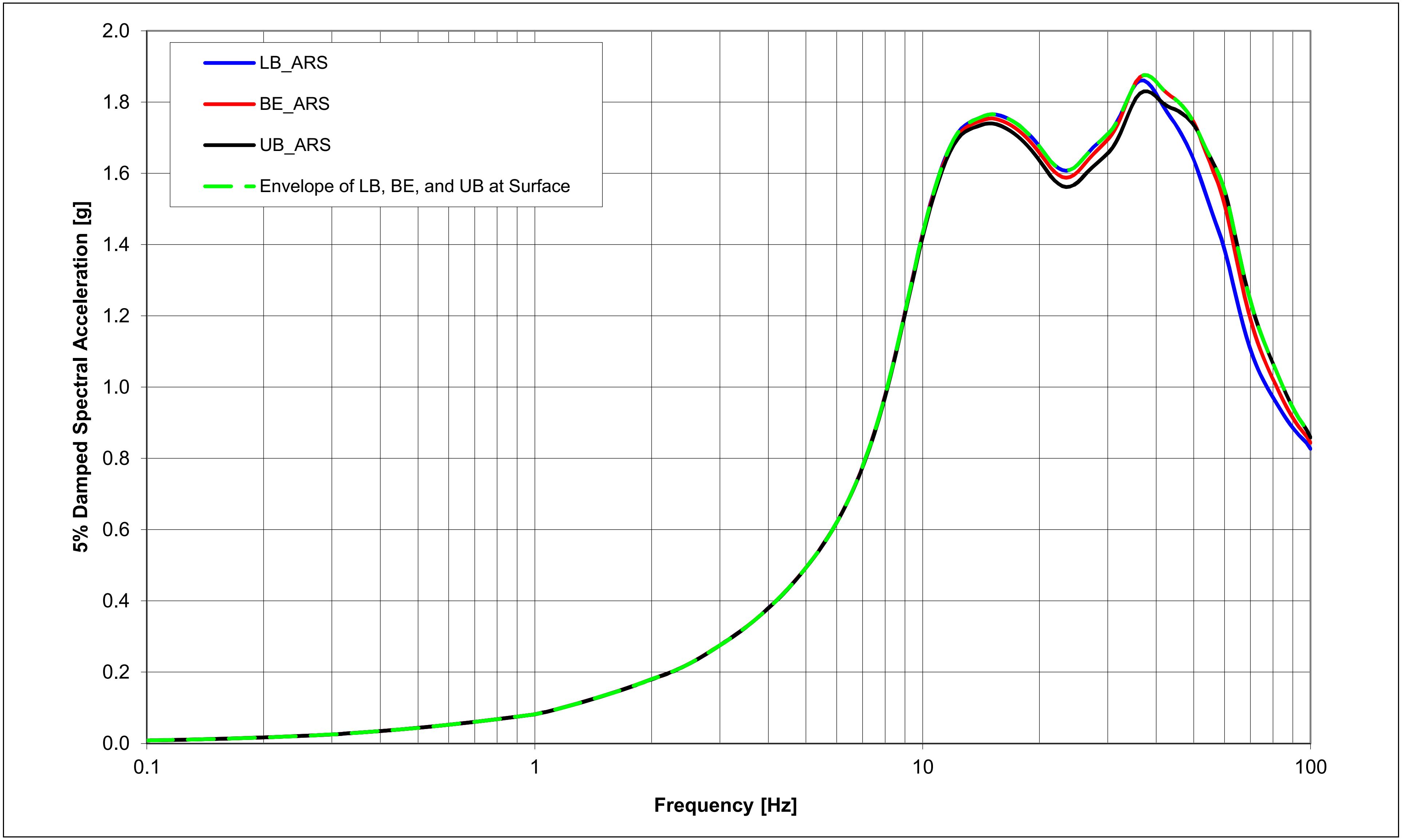


NAPS SUP 3.7-1 **Figure 3.7.1-224 NEI Check and SSI Input Response Spectra for Vertical Full Column FIRS – CB**



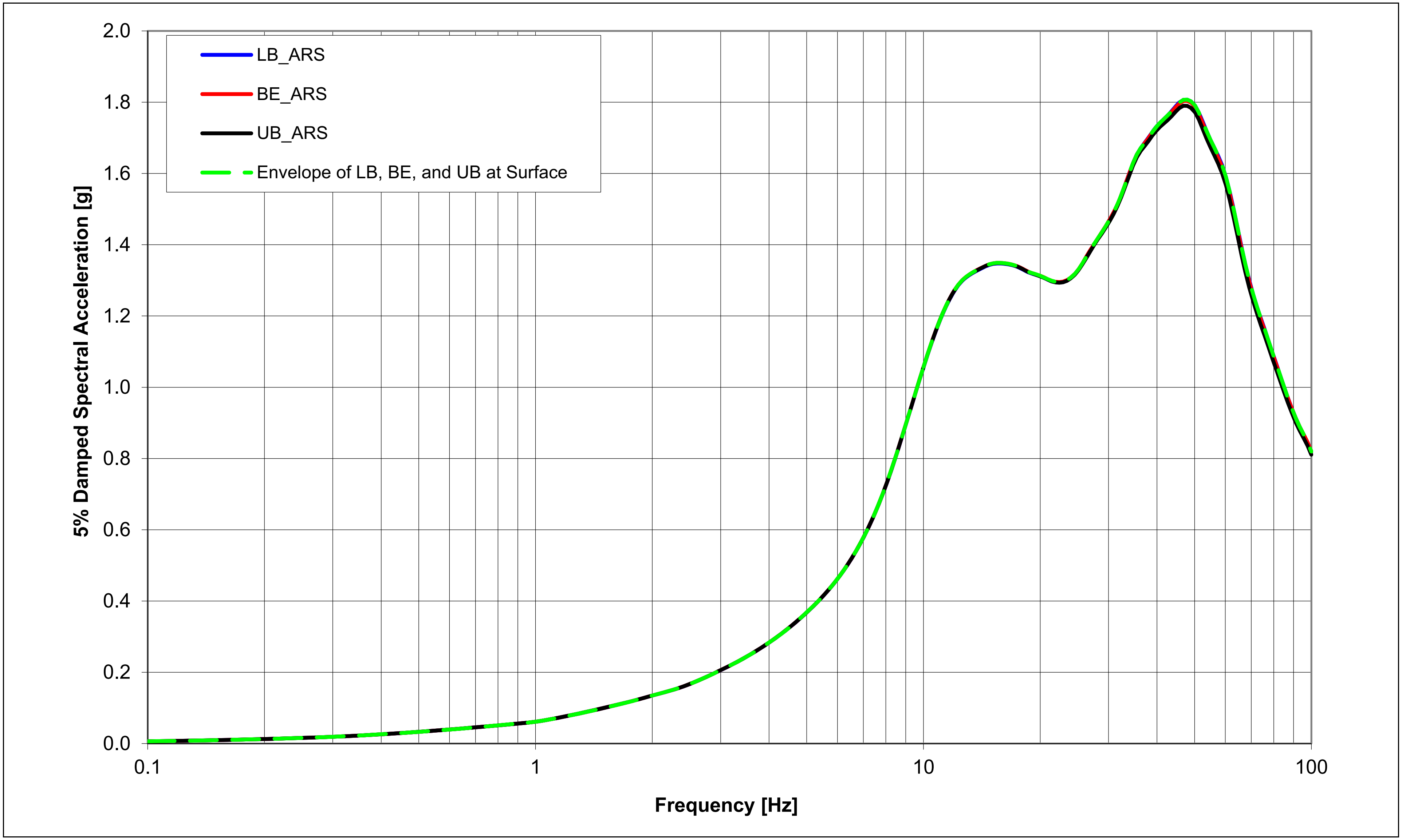
NAPS SUP 3.7-1

Figure 3.7.1-225 Envelope of Horizontal FIRS Propagated to the Ground Surface through Partial Column SSI Input Profiles – CB

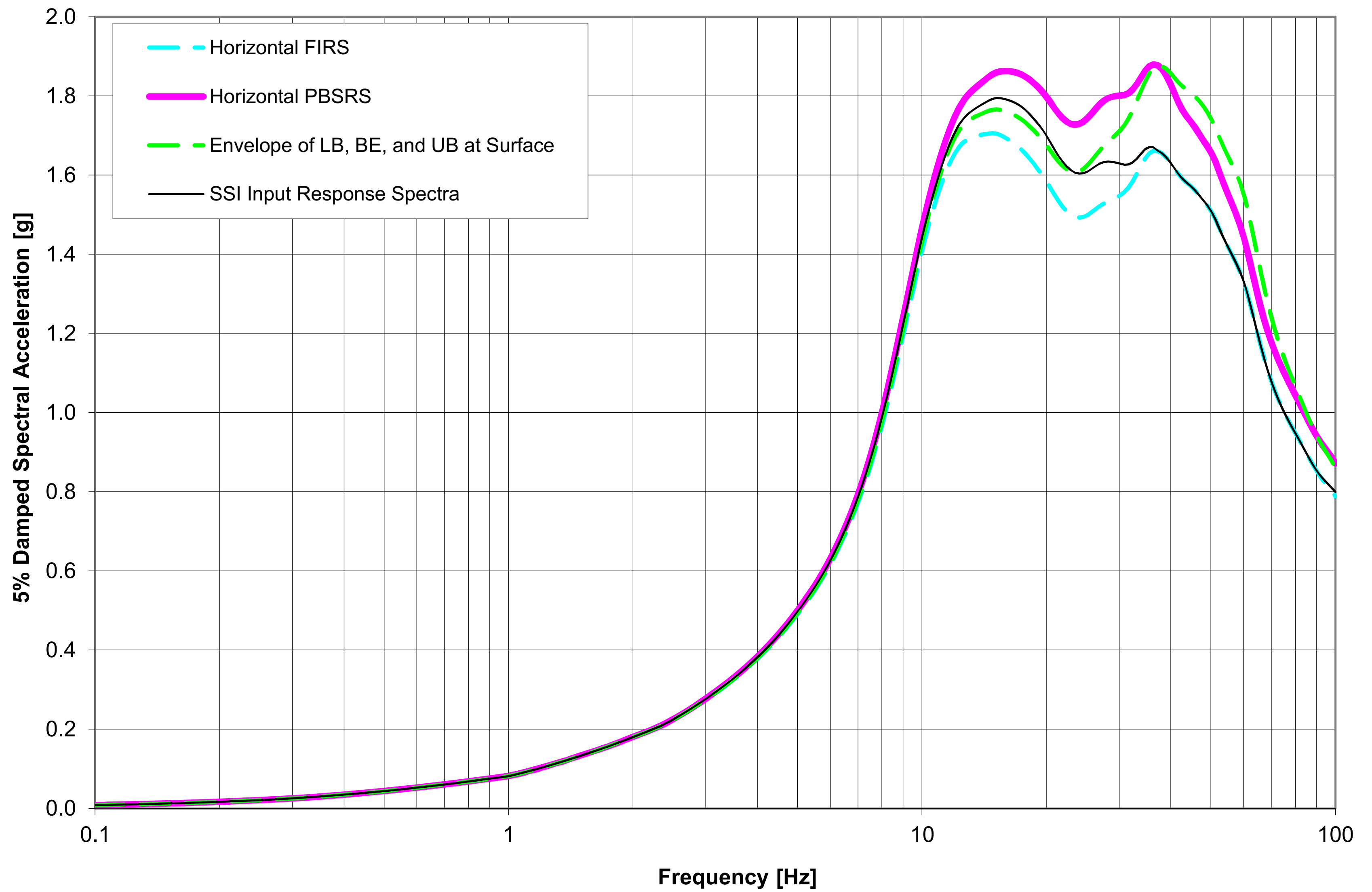


NAPS SUP 3.7-1

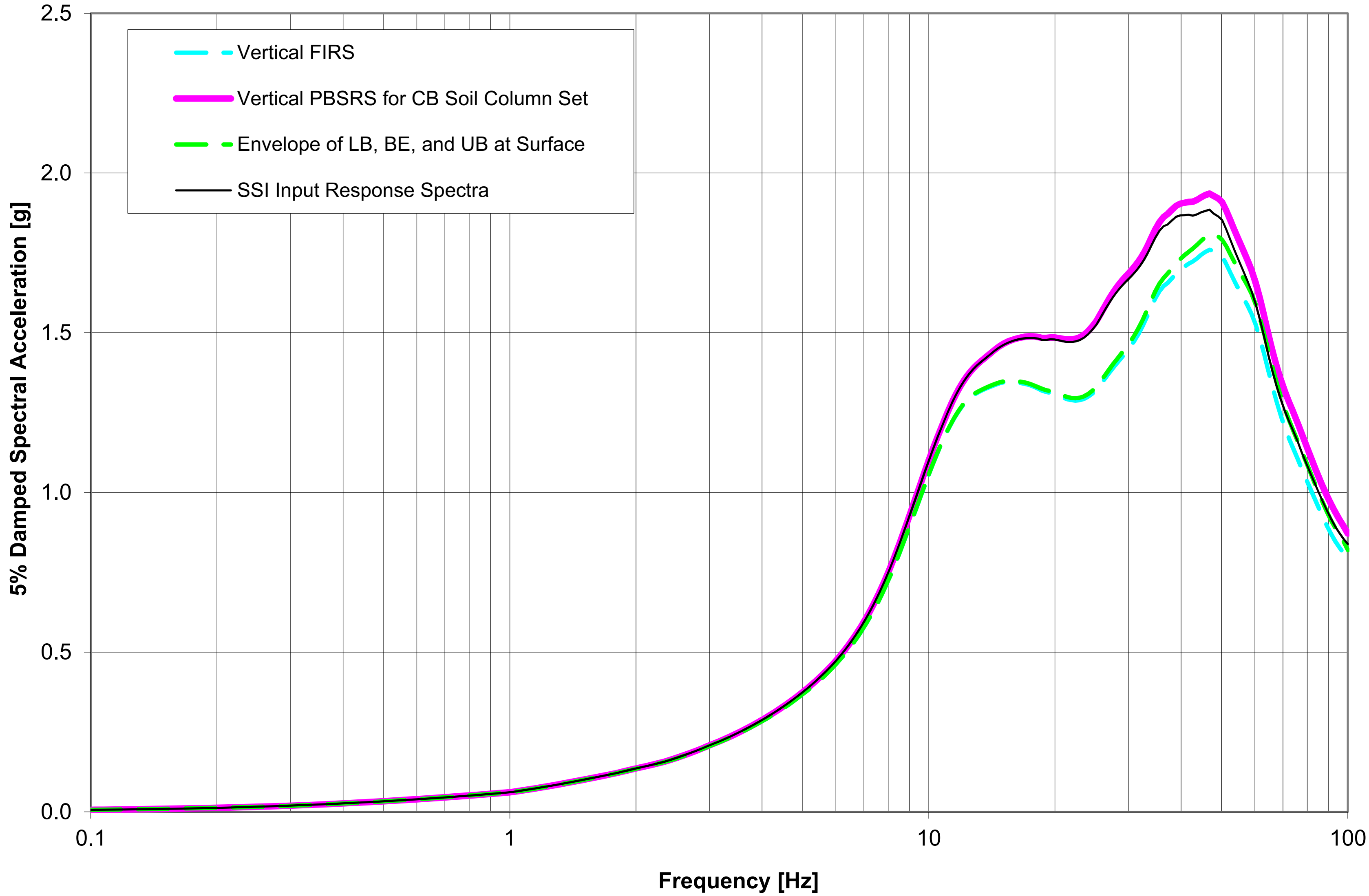
Figure 3.7.1-226 Envelope of Vertical FIRS Propagated to the Ground Surface through Partial Column SSI Input Profiles – CB



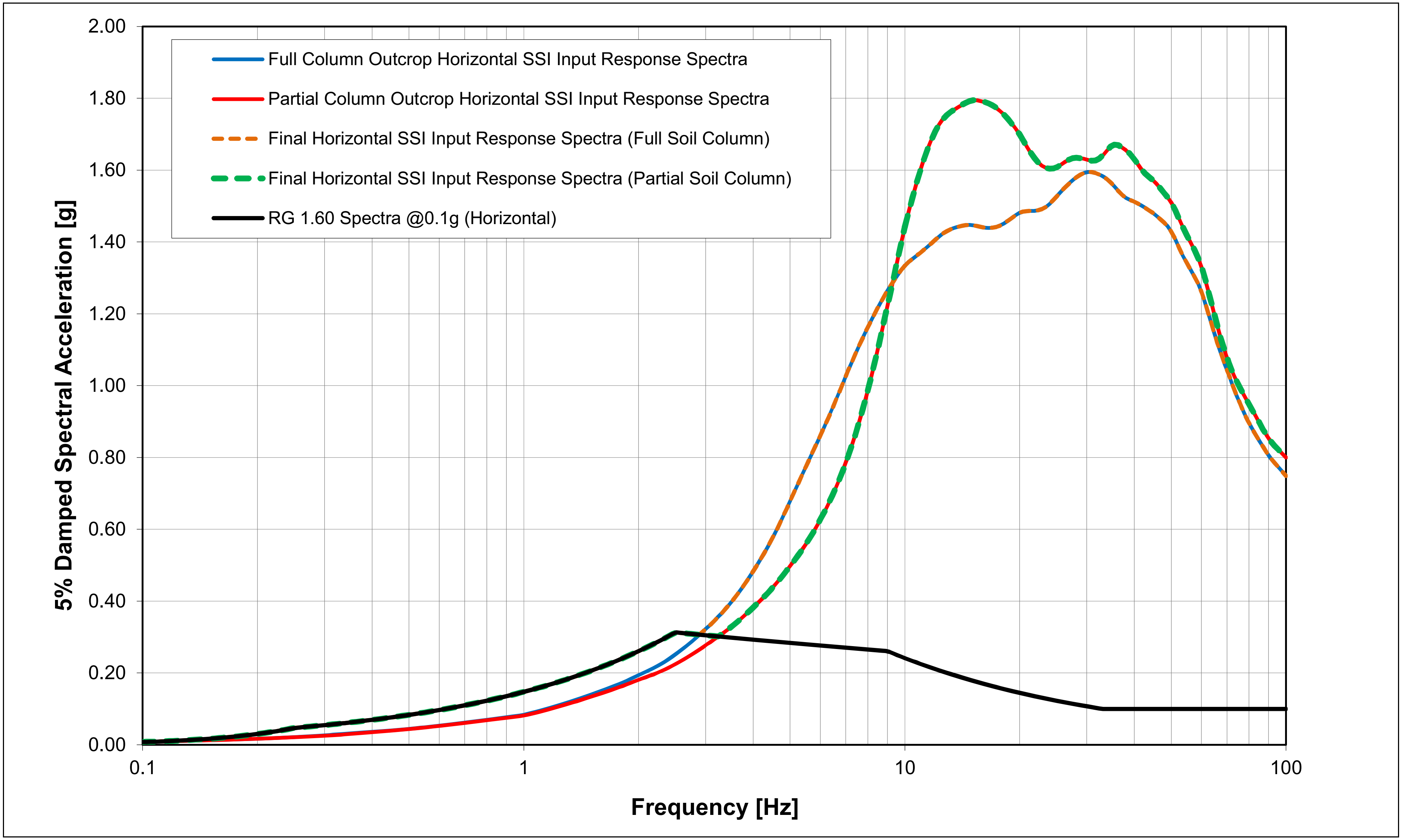
NAPS SUP 3.7-1 **Figure 3.7.1-227 NEI Check and SSI Input Response Spectra for Horizontal Partial Column FIRS – CB**



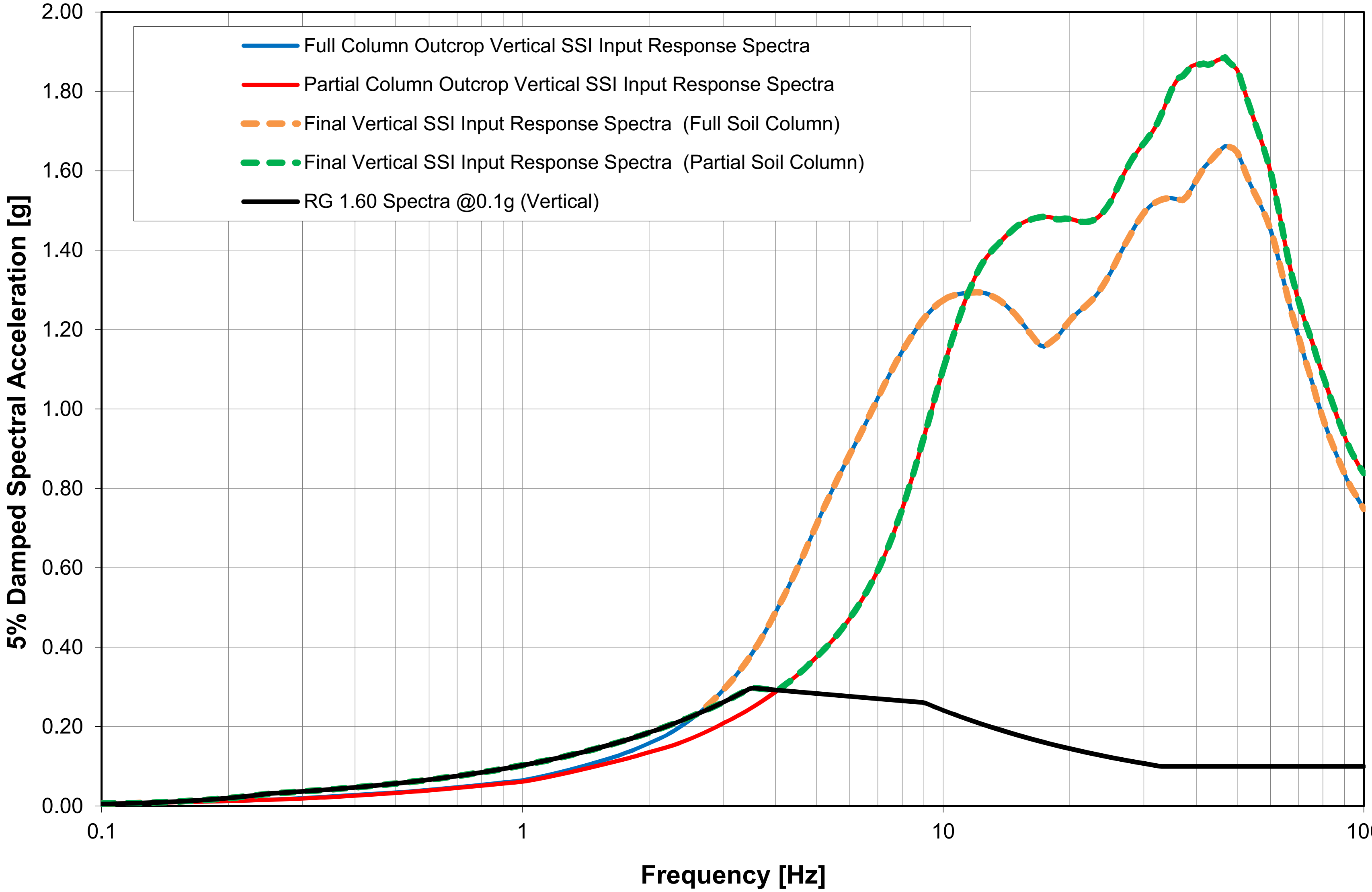
NAPS SUP 3.7-1 **Figure 3.7.1-228 NEI Check and SSI Input Response Spectra for Vertical Partial Column FIRS – CB**



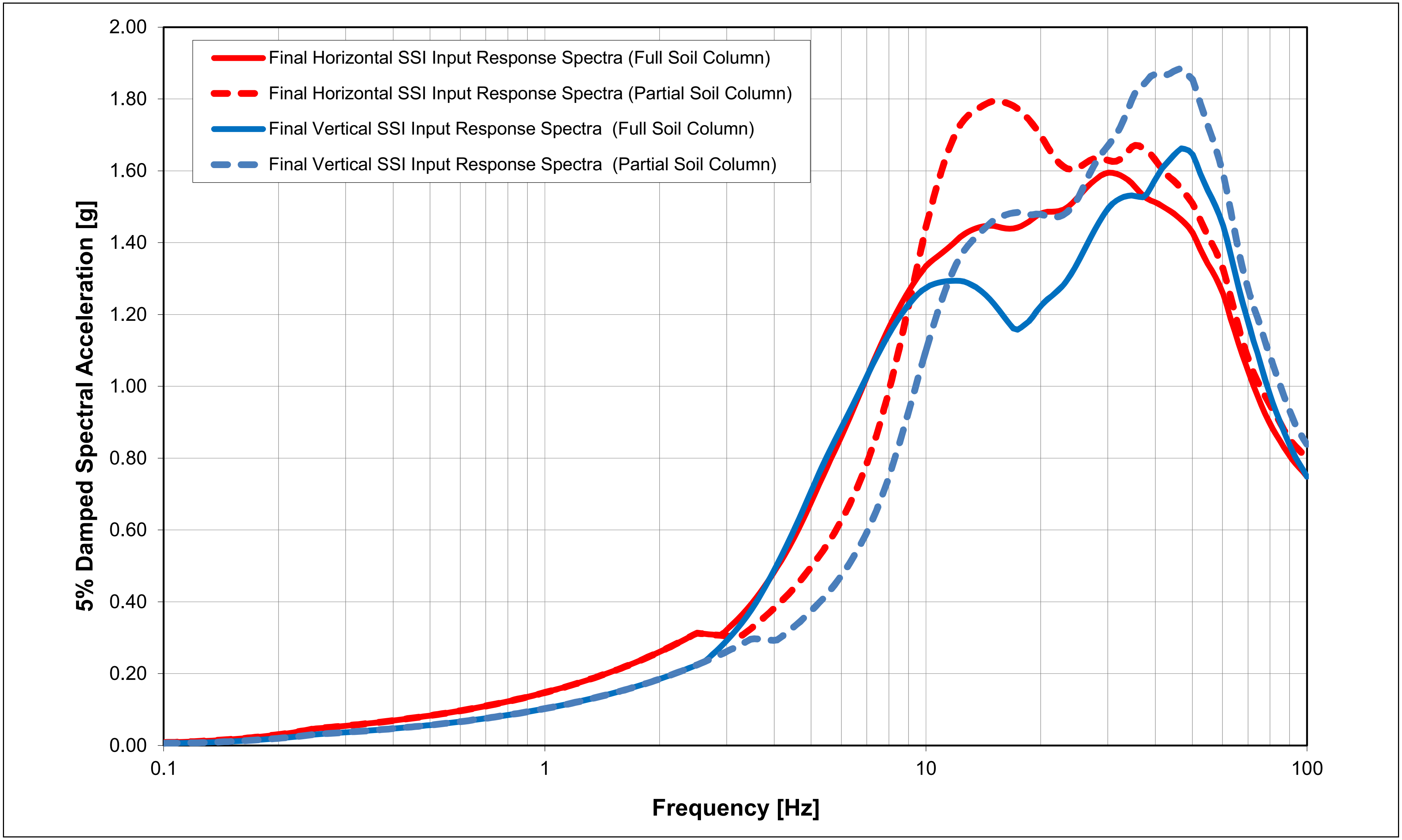
NAPS SUP 3.7-1 **Figure 3.7.1-229 Development of 5% Damped Final Horizontal SSI Input Response Spectra for CB**



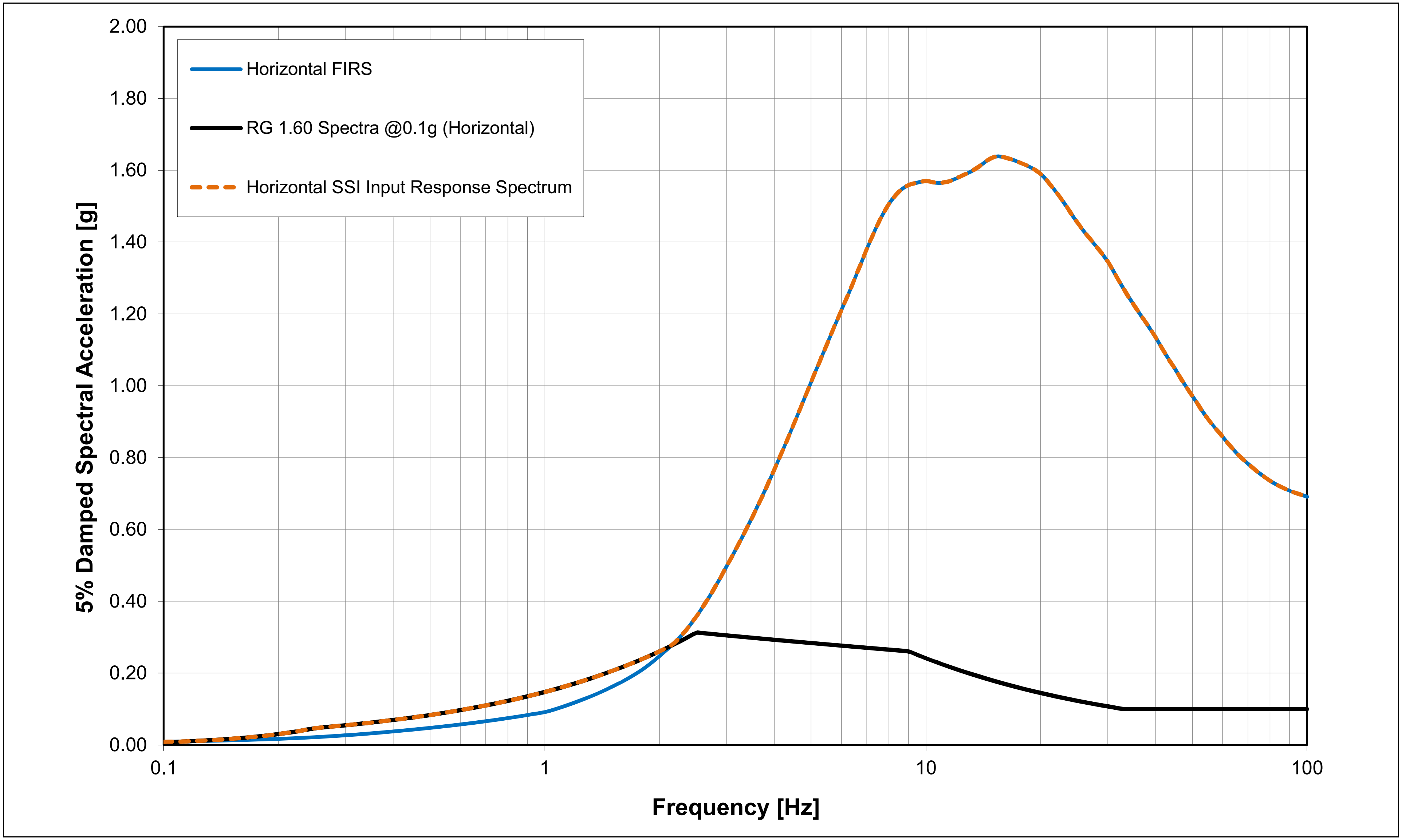
NAPS SUP 3.7-1 **Figure 3.7.1-230 Development of 5% Damped Final Vertical SSI Input Response Spectra for CB**



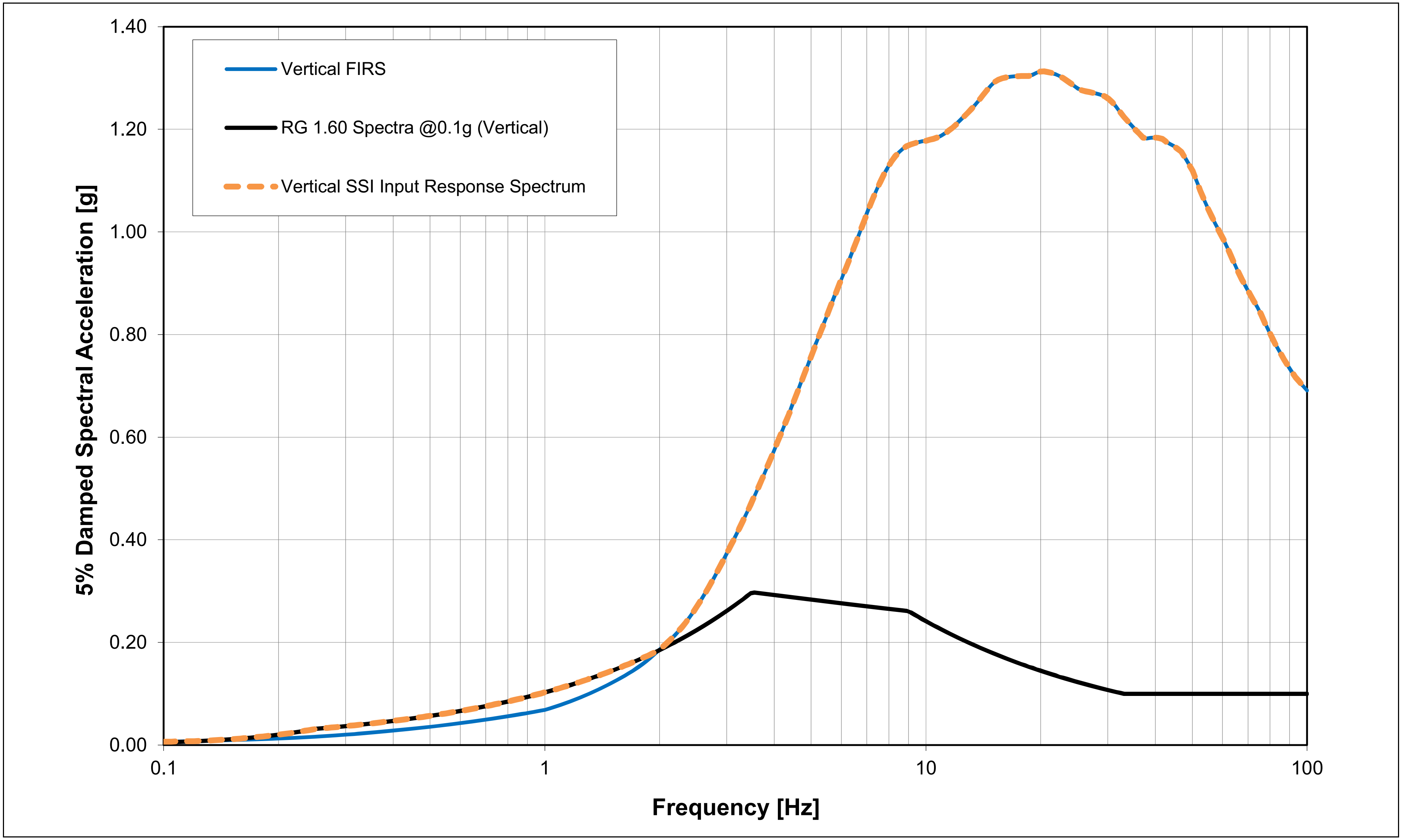
NAPS SUP 3.7-1 **Figure 3.7.1-231 5% Damped Final SSI Input Response Spectra for CB**



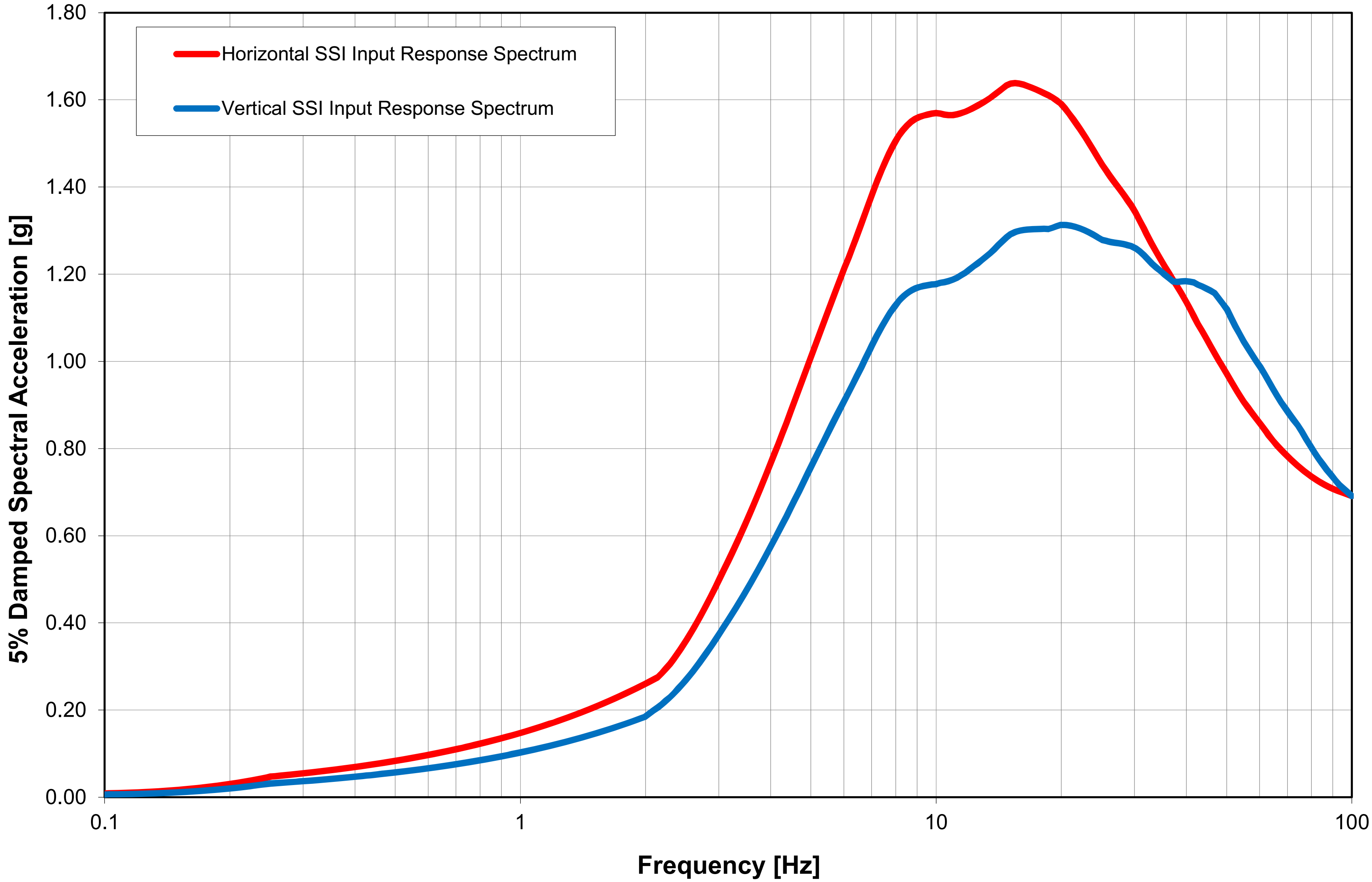
NAPS SUP 3.7-1 **Figure 3.7.1-232 Development of 5% Damped Final Horizontal SSI Input Response Spectrum at Elevation 282 ft for FWSC**



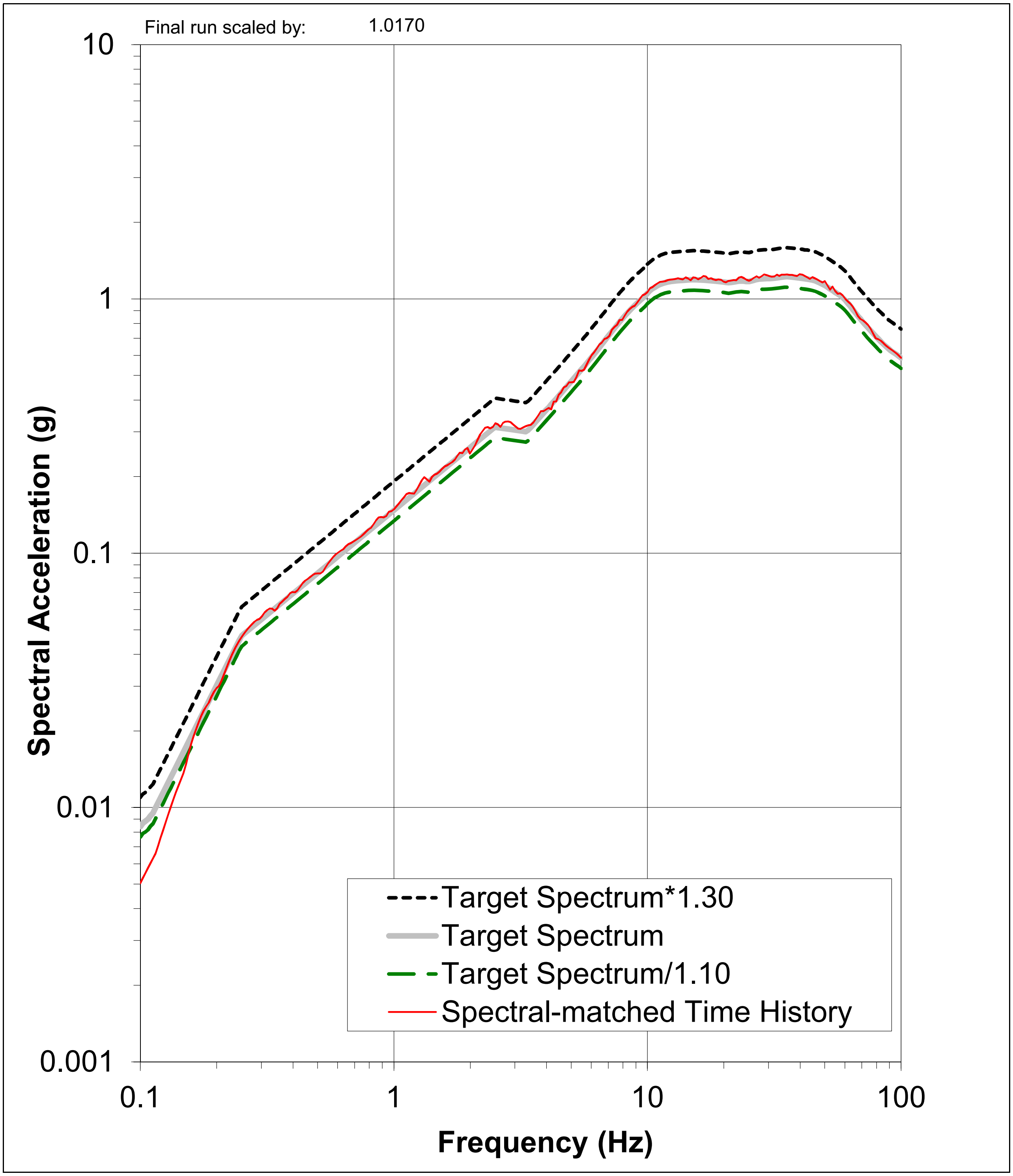
NAPS SUP 3.7-1 **Figure 3.7.1-233 Development of 5% Damped Final Vertical SSI Input Response Spectrum at Elevation 282 ft for FWSC**



NAPS SUP 3.7-1 **Figure 3.7.1-234 5% Damped Final SSI Input Response Spectra at Elevation 282 ft for FWSC**

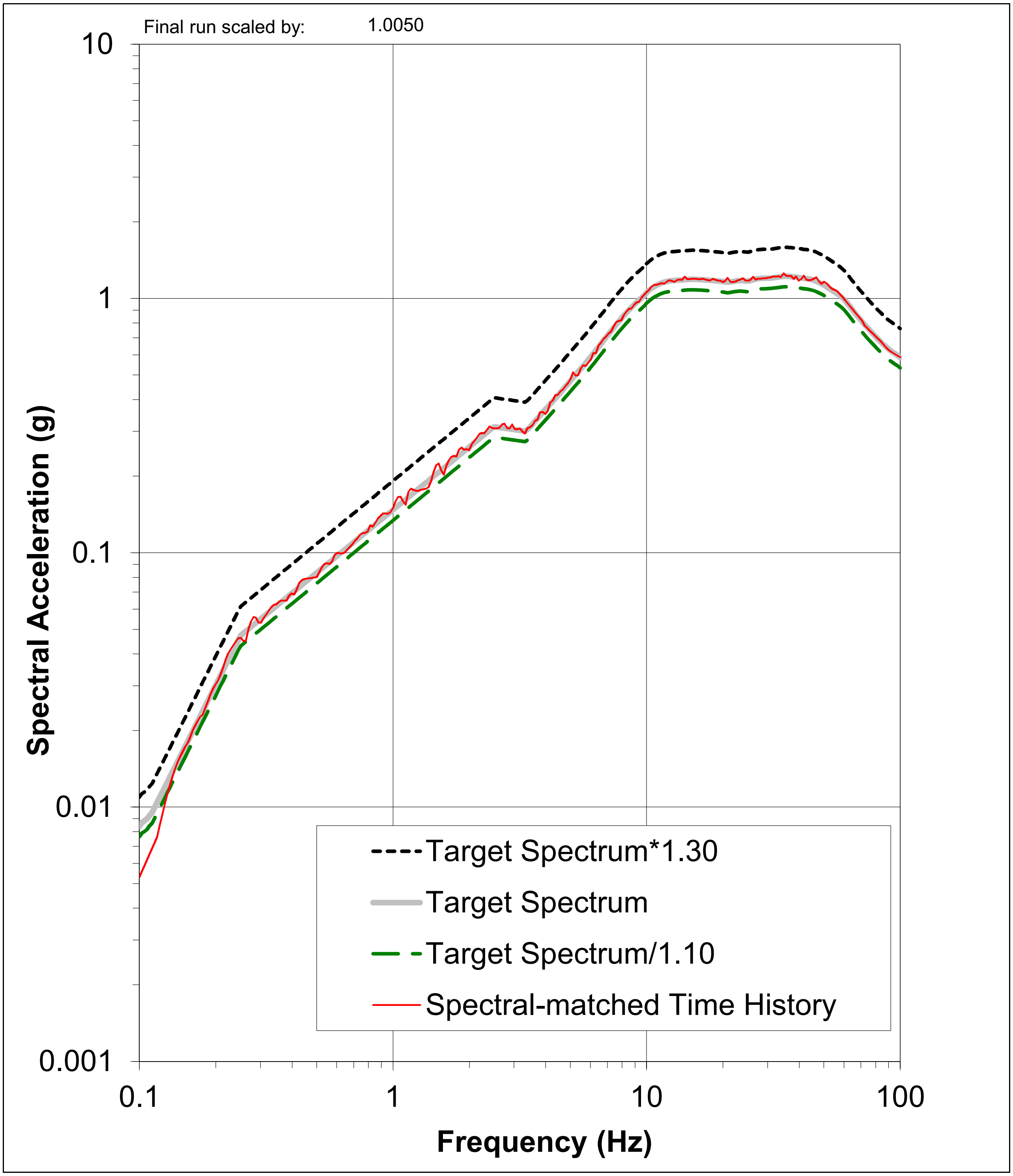


NAPS SUP 3.7-2 **Figure 3.7.1-235 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column RB/FB case, H1 Component**

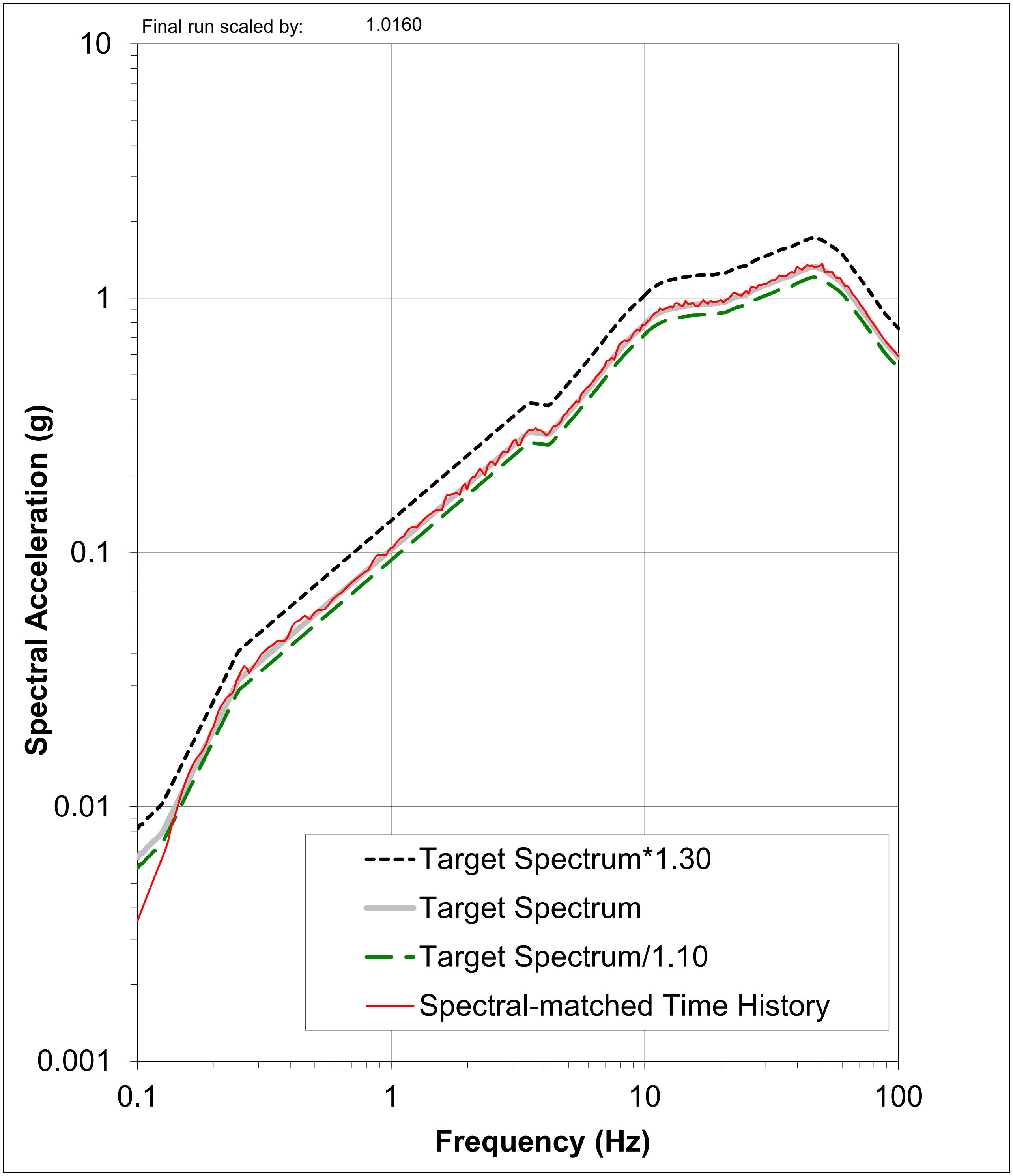


NAPS SUP 3.7-2

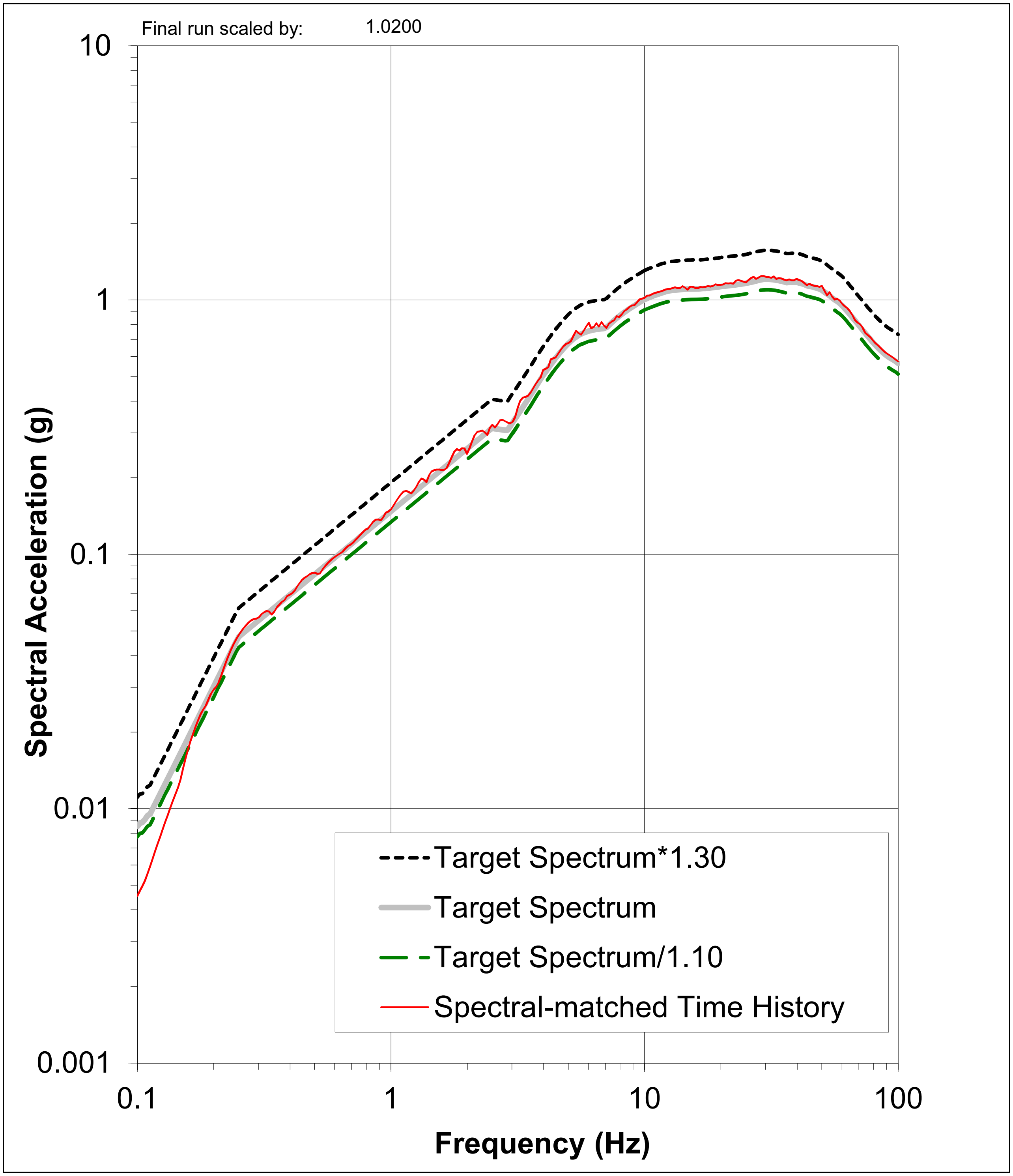
Figure 3.7.1-236 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column RB/FB case, H2 Component



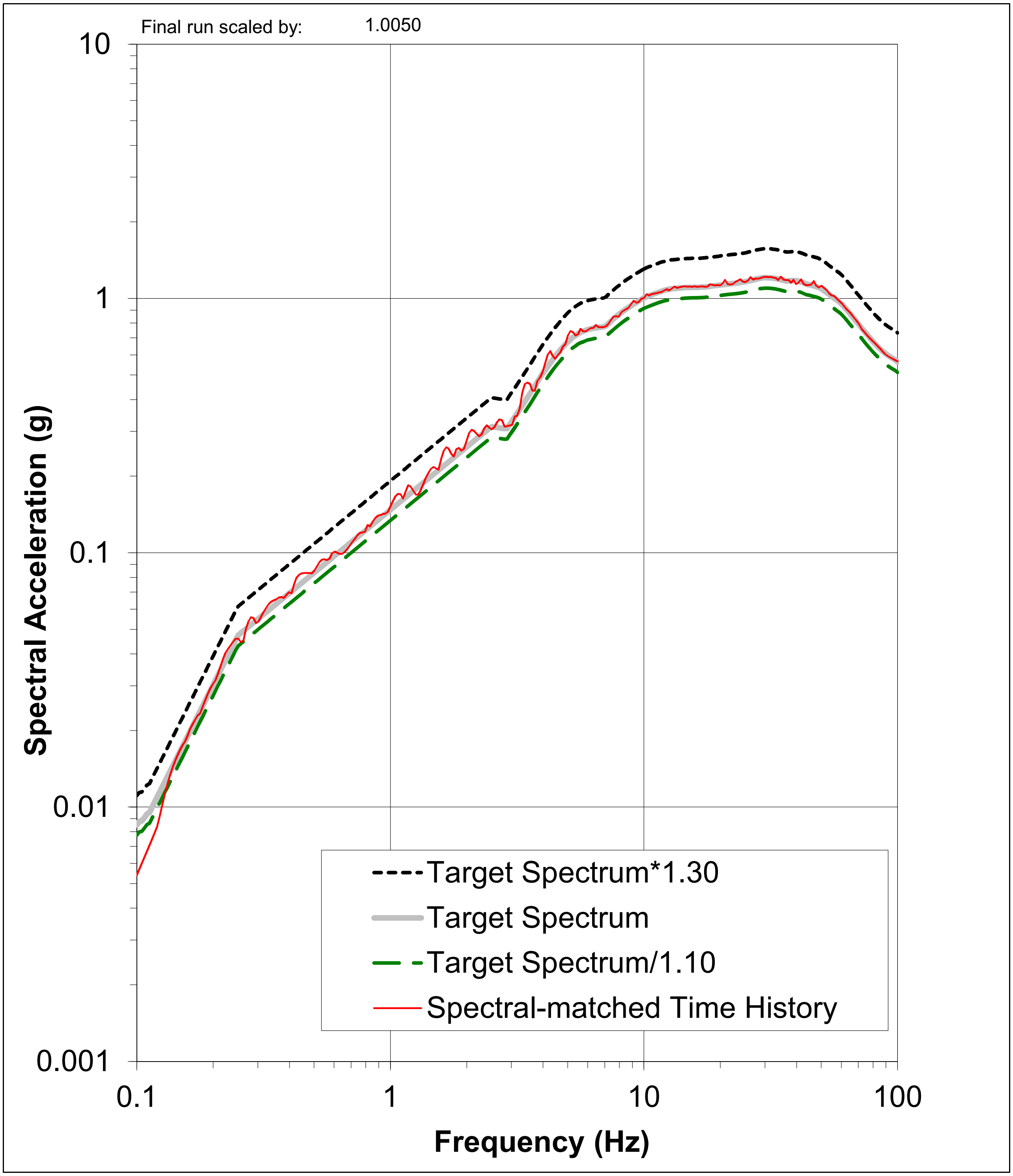
NAPS SUP 3.7-2 **Figure 3.7.1-237 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column RB/FB case, UP Component**



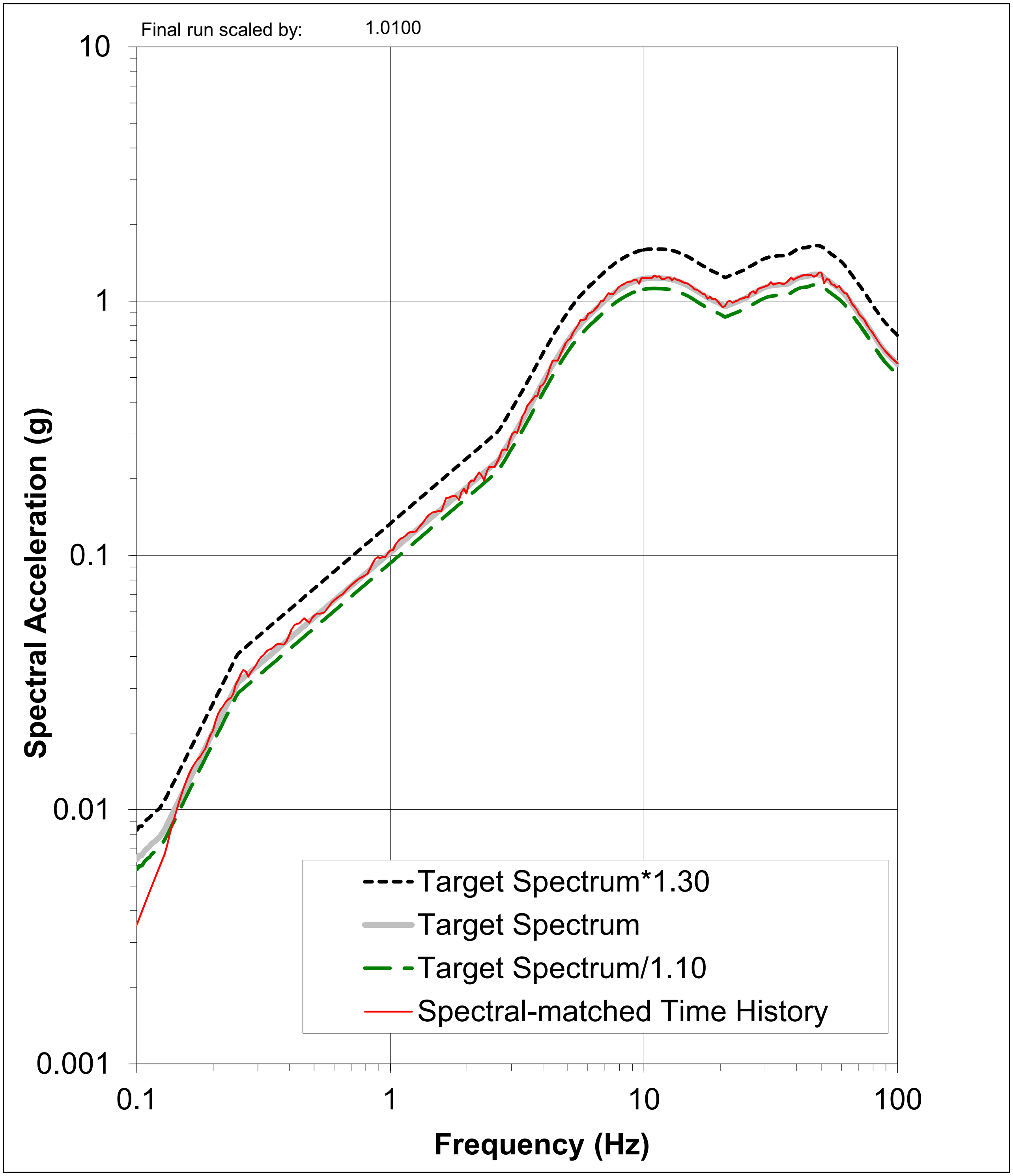
NAPS SUP 3.7-2 **Figure 3.7.1-238 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column RB/FB case, H1 Component**



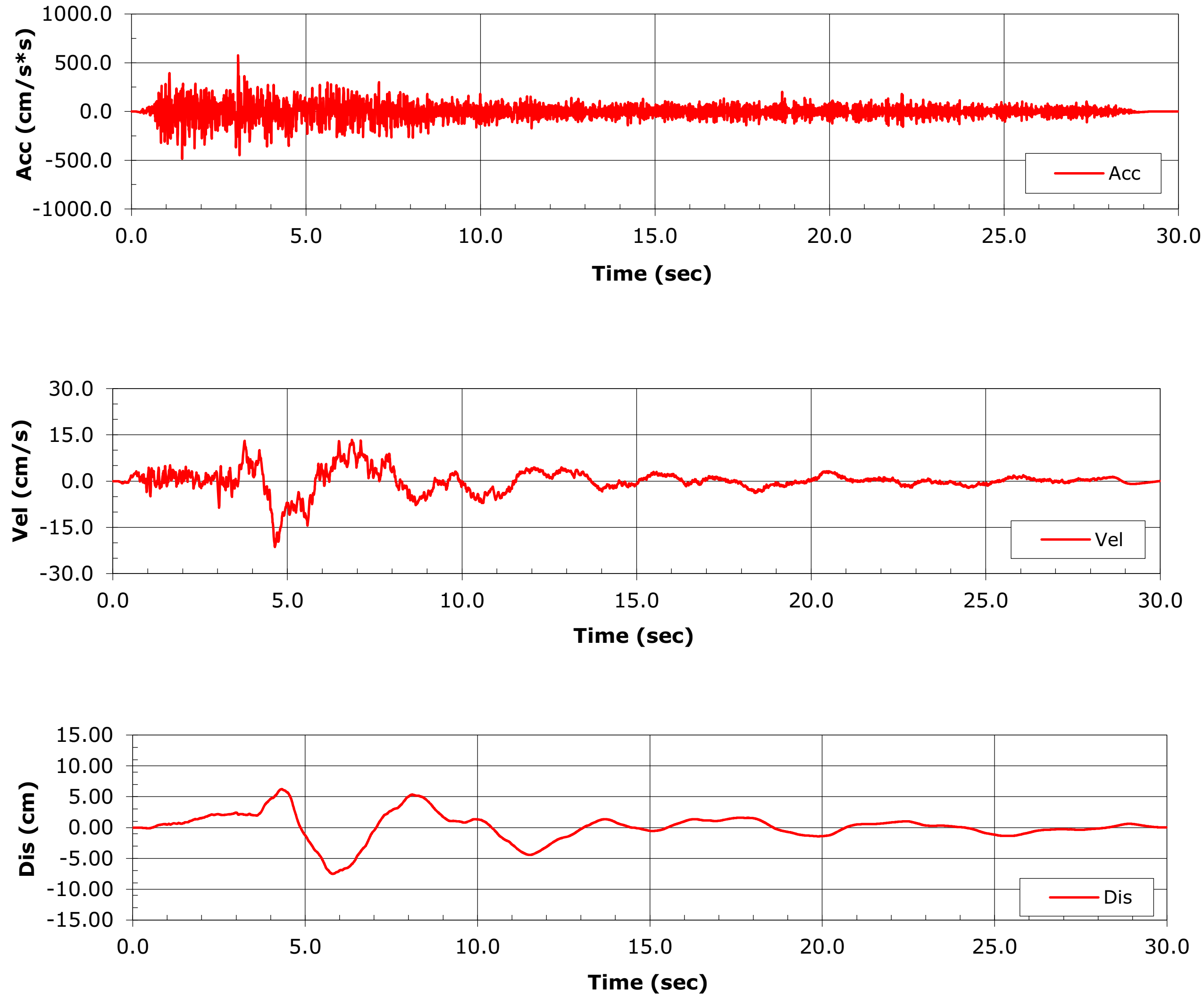
NAPS SUP 3.7-2 **Figure 3.7.1-239 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column RB/FB case, H2 Component**



NAPS SUP 3.7-2 **Figure 3.7.1-240 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column RB/FB case, UP Component**

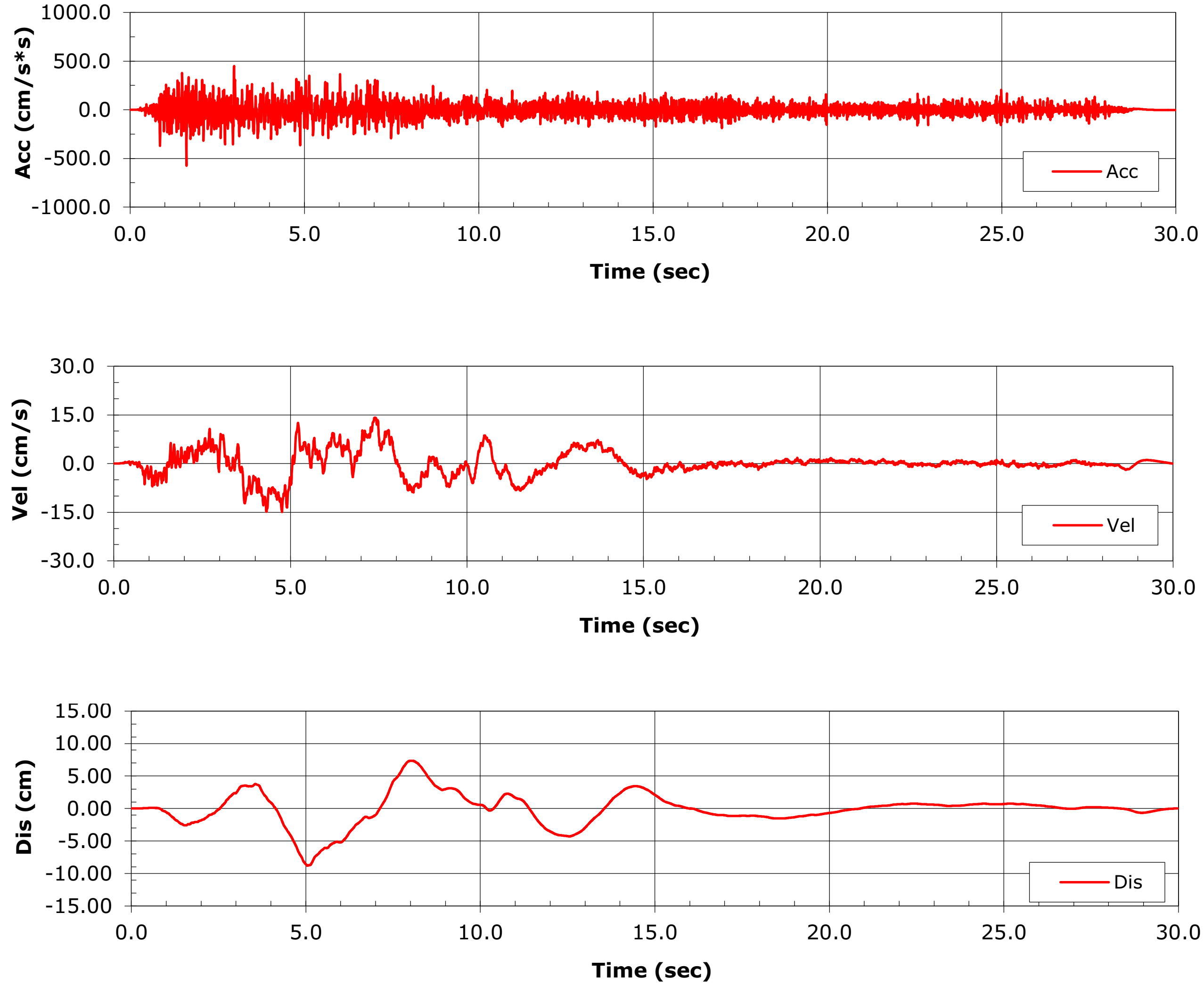


NAPS SUP 3.7-2 **Figure 3.7.1-241 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for RB/FB, H1 Component**

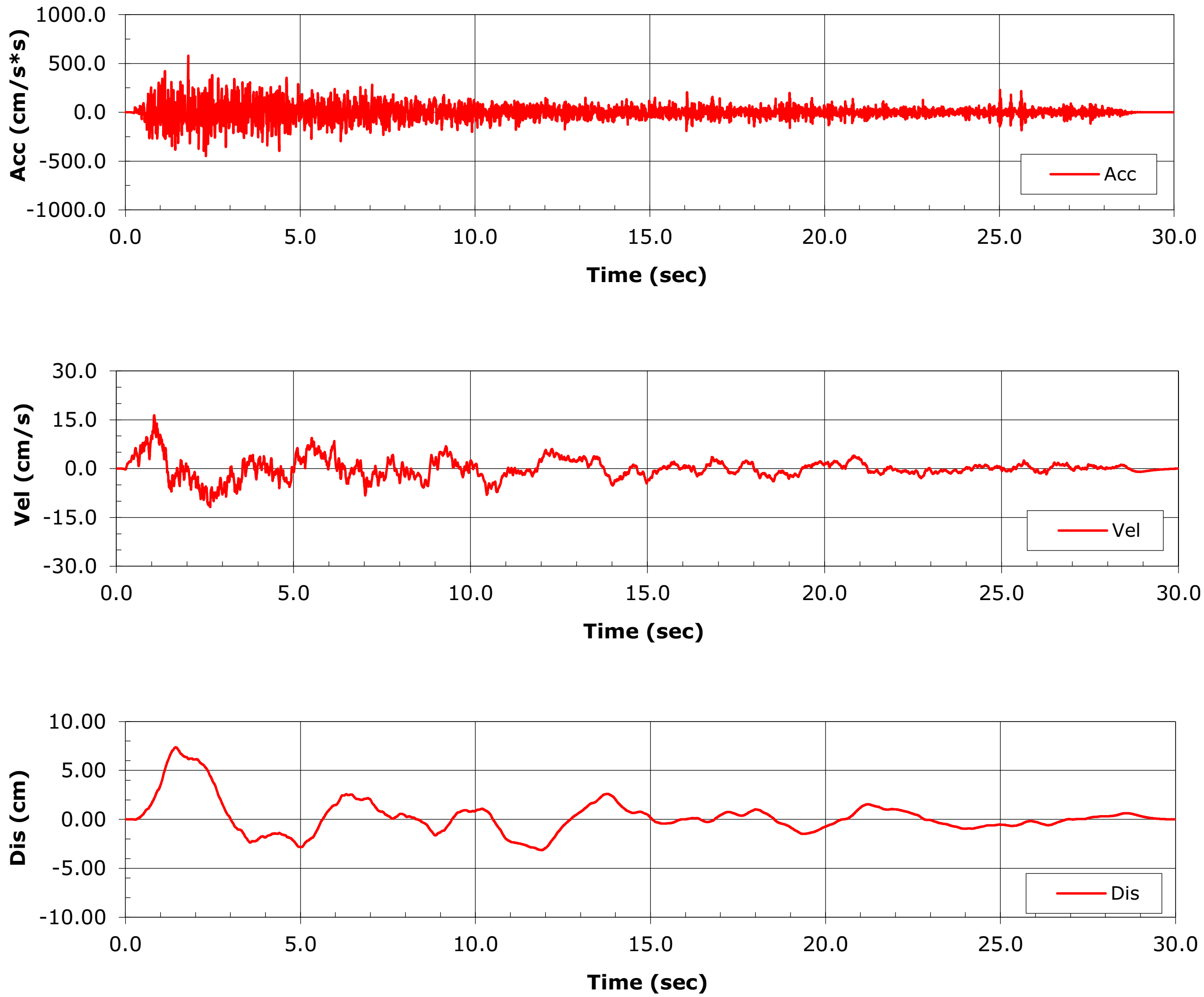


NAPS SUP 3.7-2

Figure 3.7.1-242 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for RB/FB, H2 Component

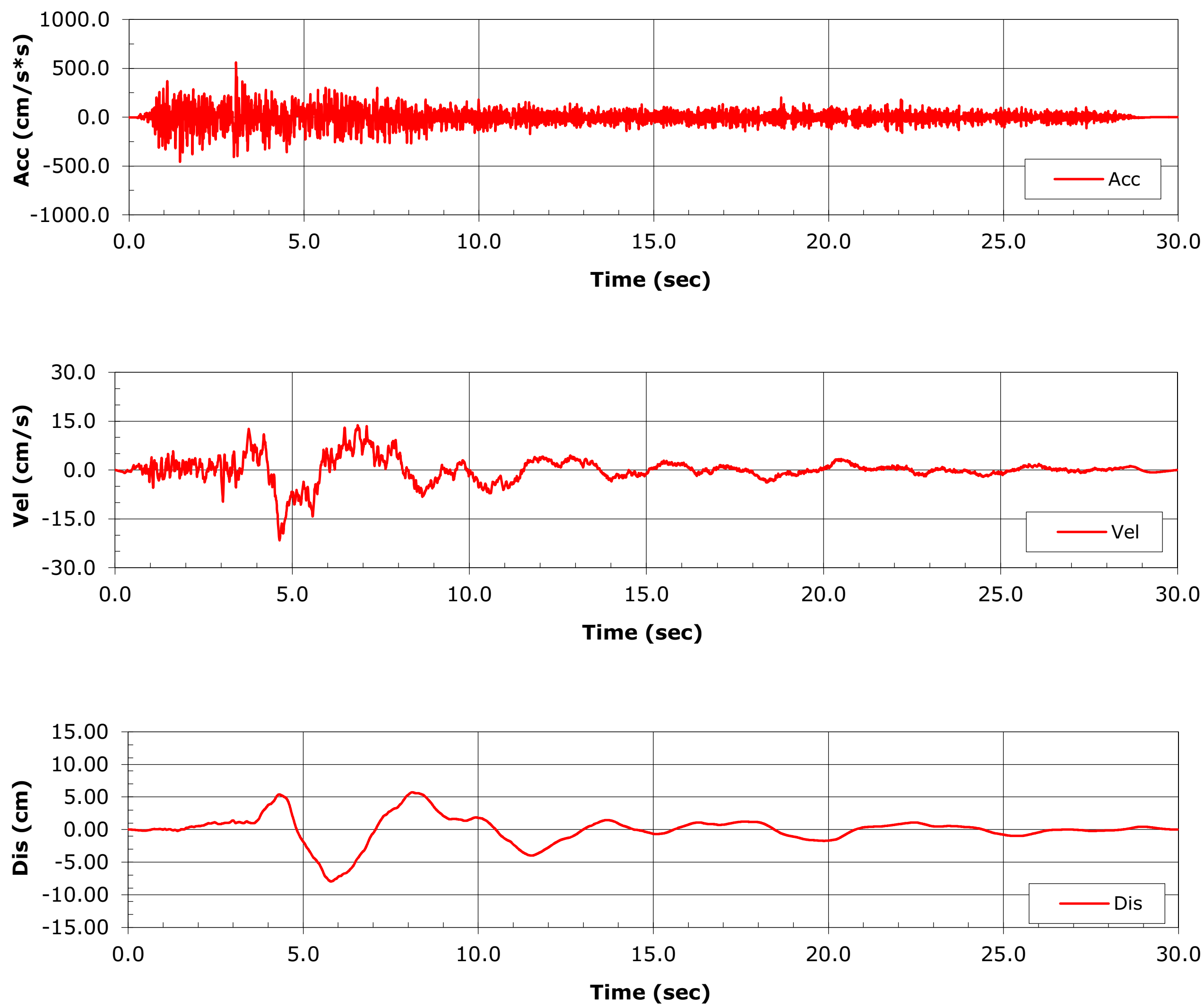


NAPS SUP 3.7-2 **Figure 3.7.1-243 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for RB/FB, UP Component**



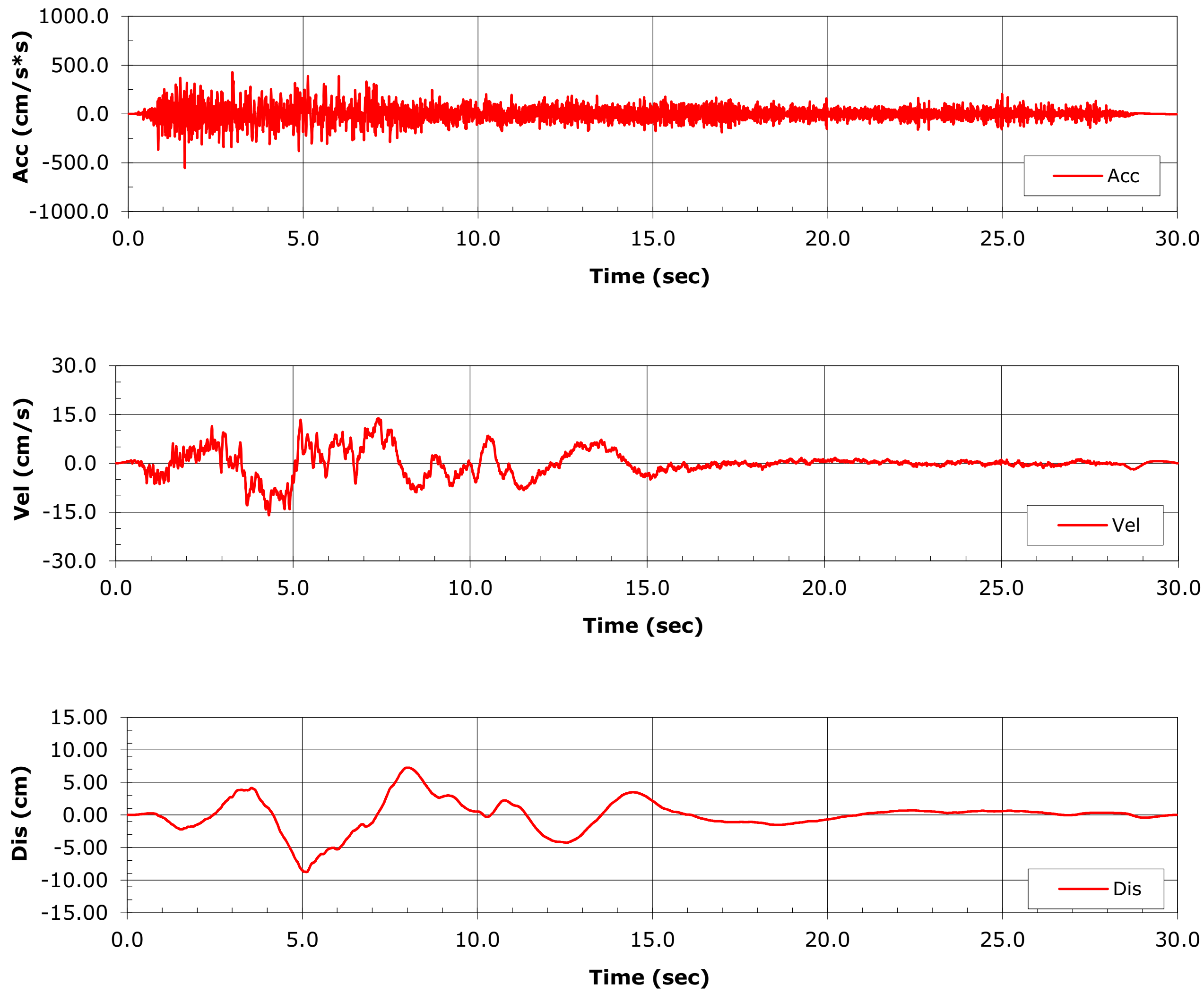
NAPS SUP 3.7-2

Figure 3.7.1-244 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for RB/FB, H1 Component

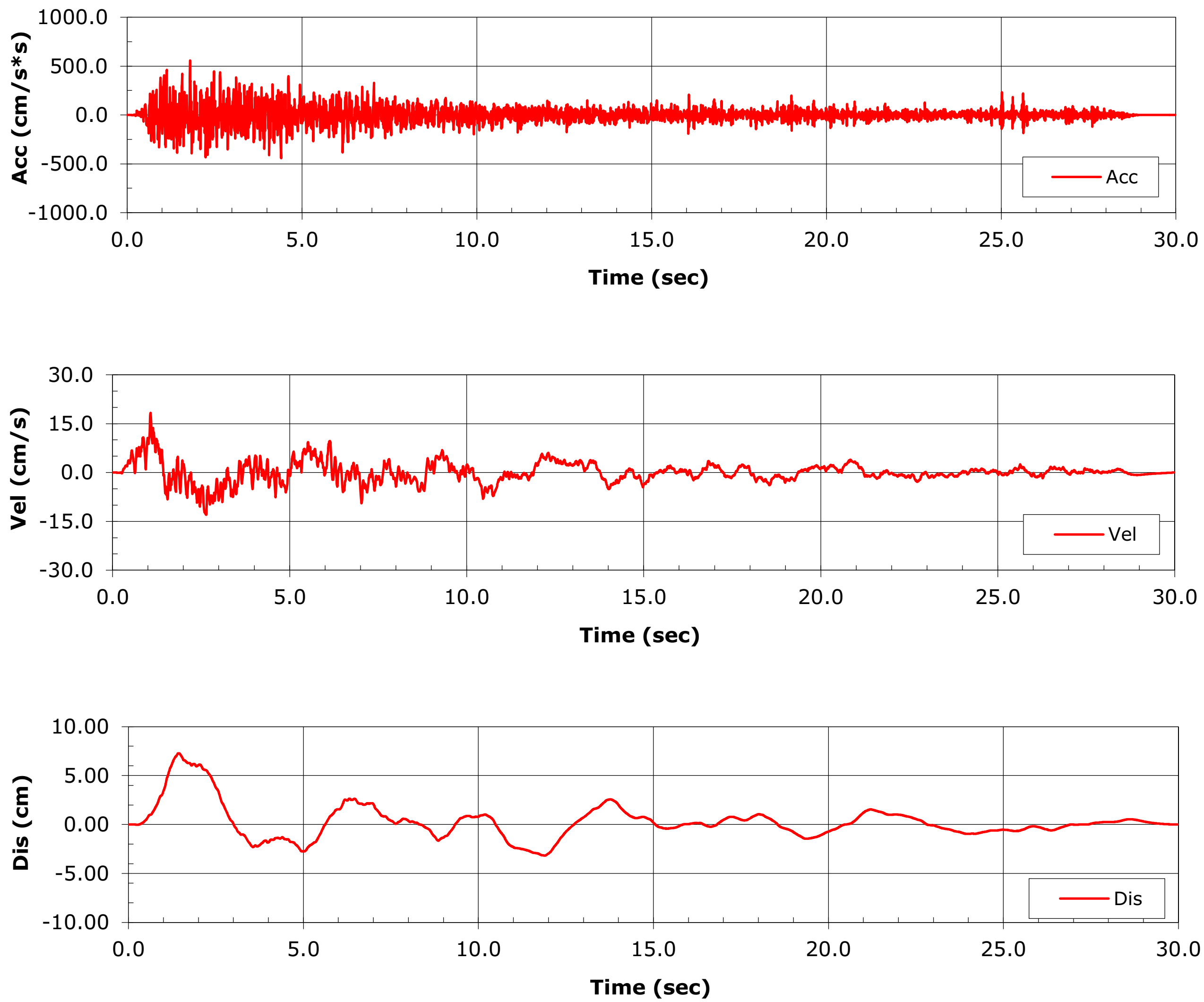


NAPS SUP 3.7-2

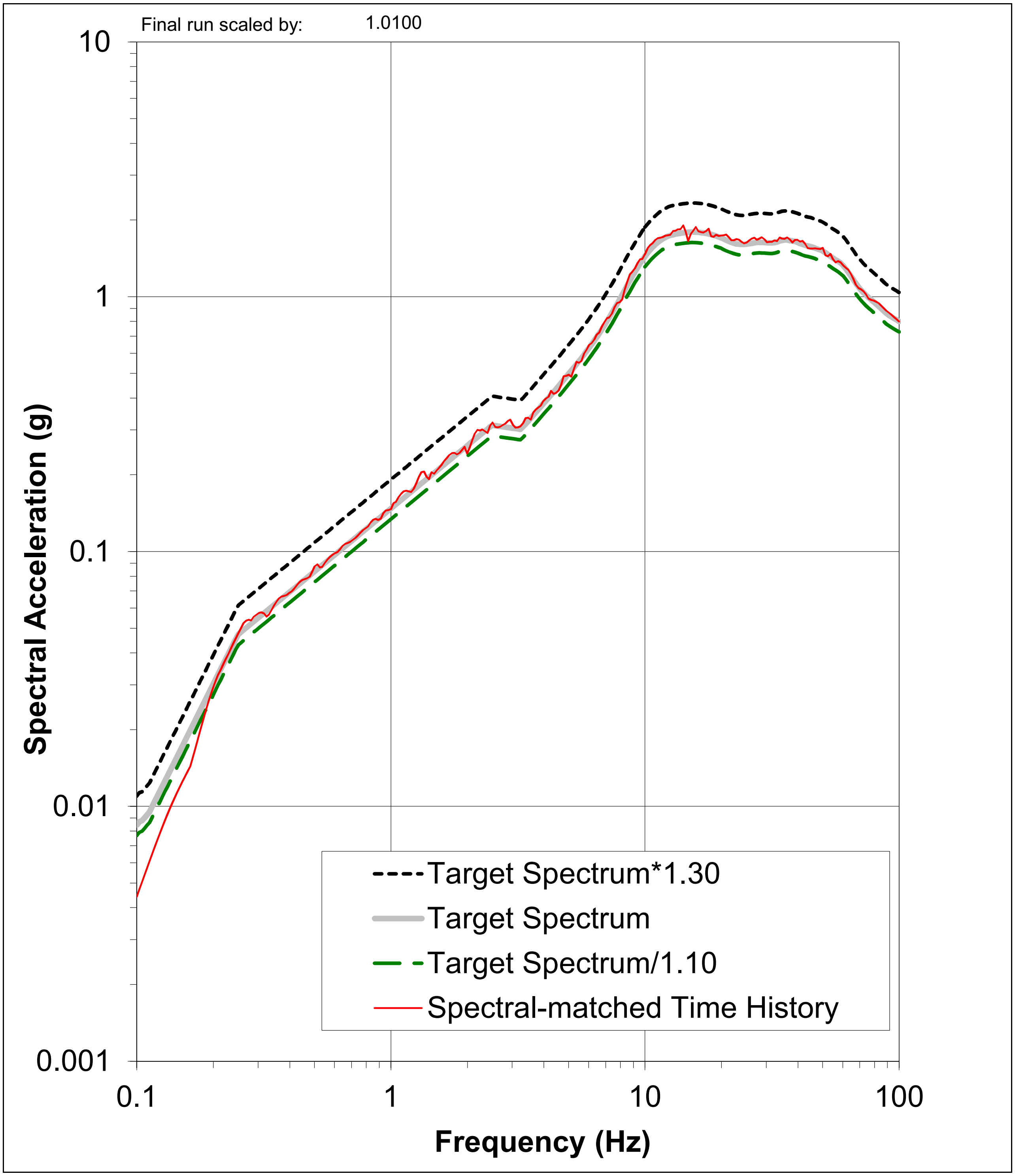
Figure 3.7.1-245 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for RB/FB, H2 Component



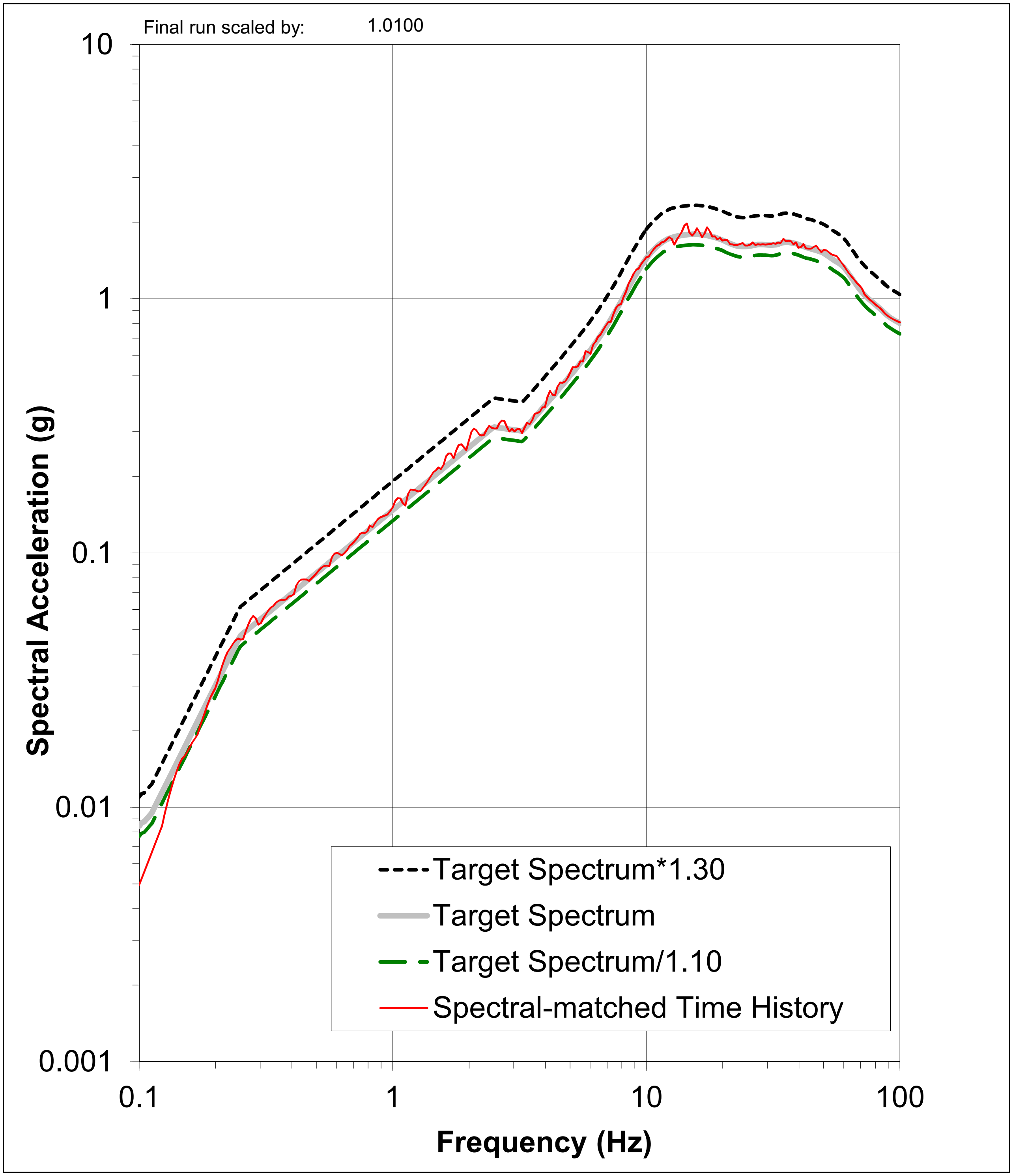
NAPS SUP 3.7-2 **Figure 3.7.1-246 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for RB/FB, UP Component**



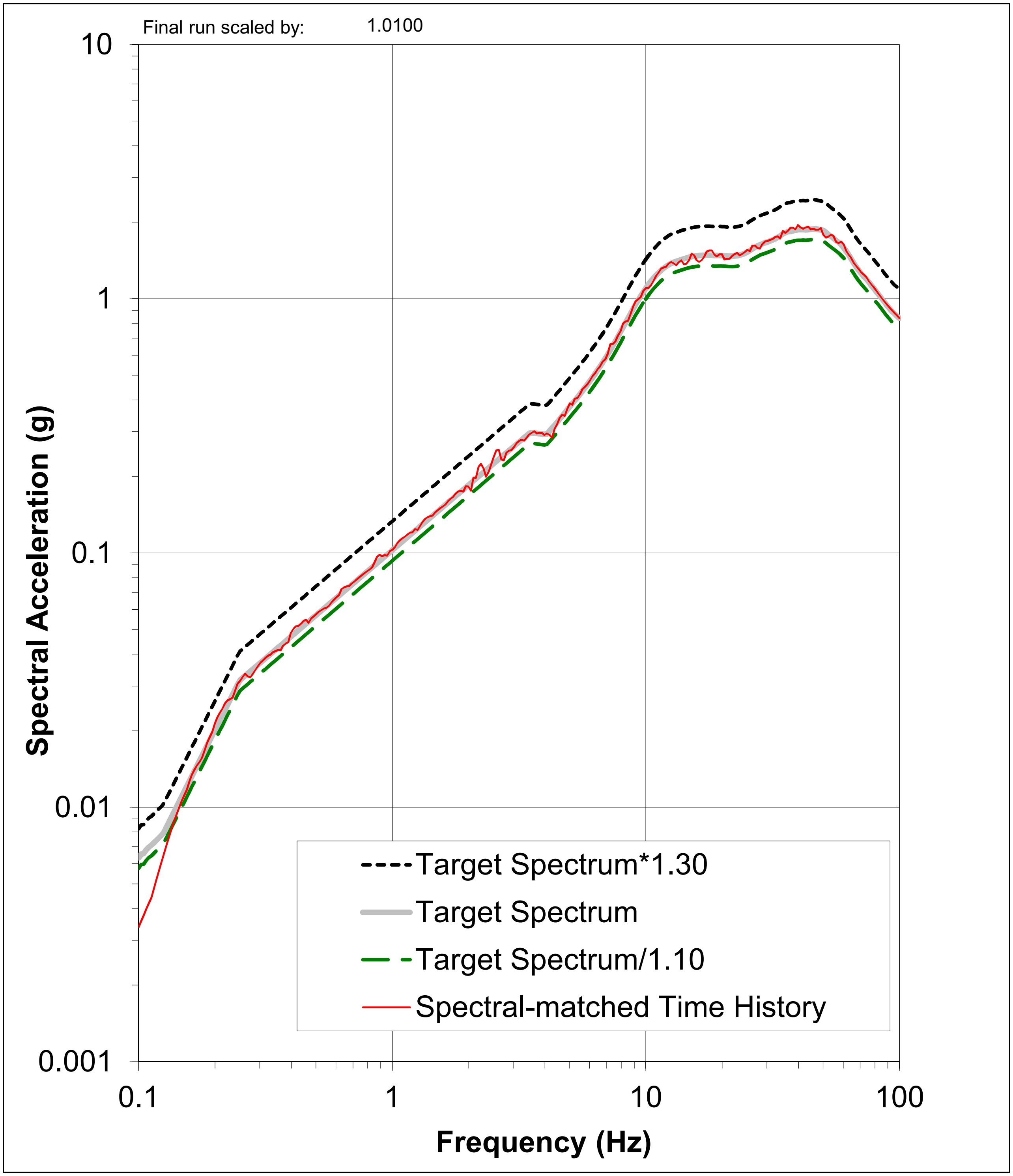
NAPS SUP 3.7-2 **Figure 3.7.1-247 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column CB case, H1 Component**



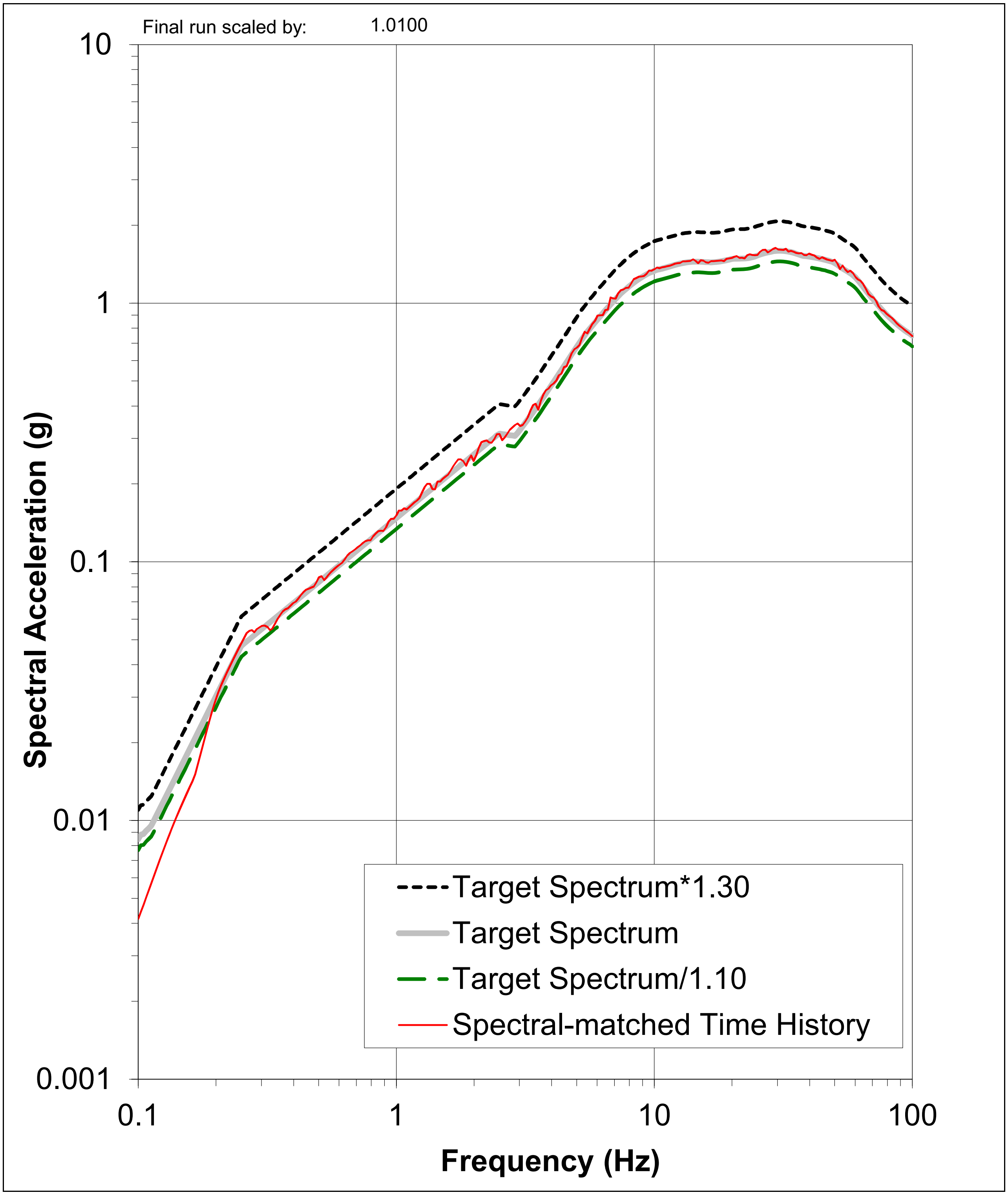
NAPS SUP 3.7-2 **Figure 3.7.1-248 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column CB case, H2 Component**



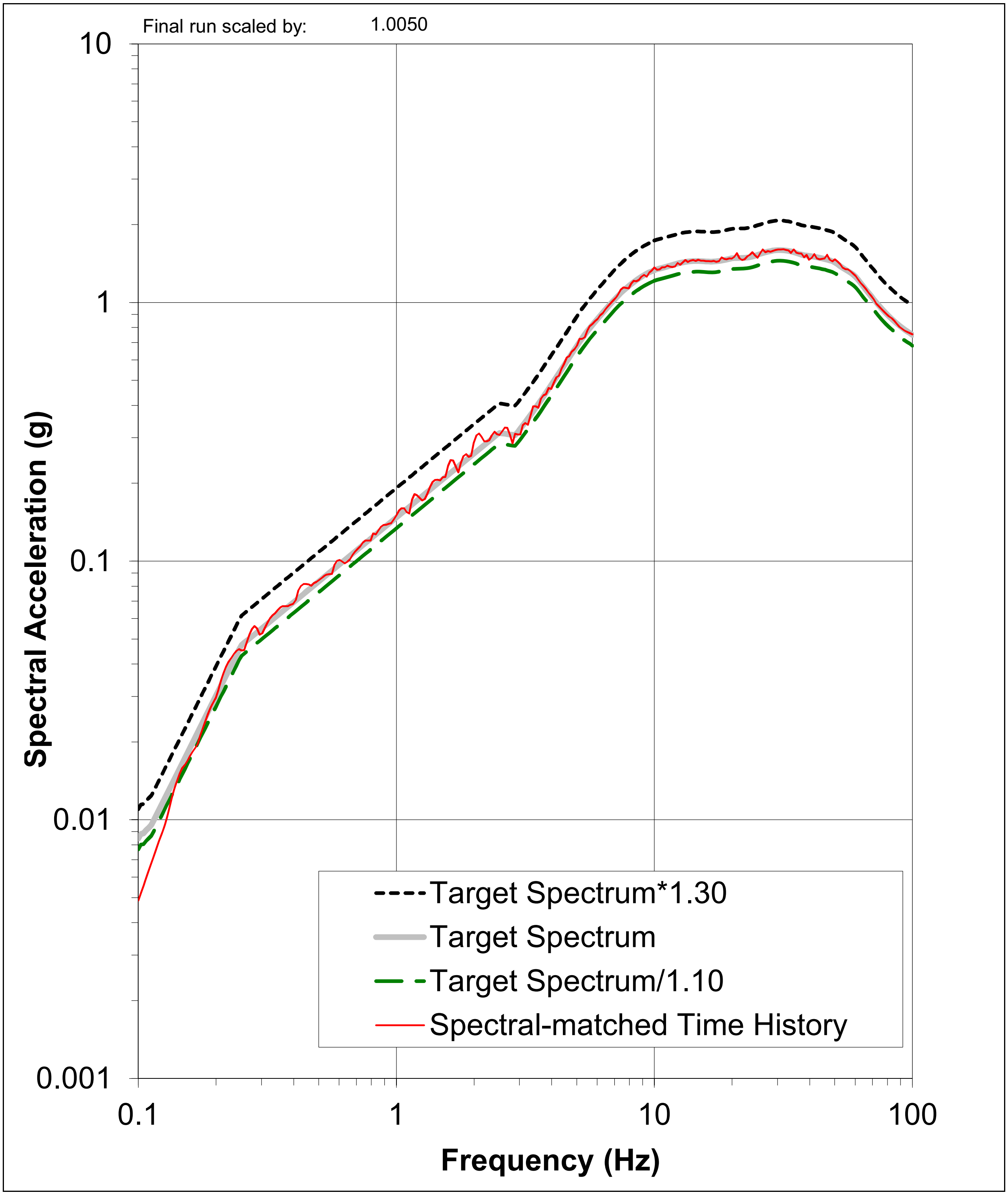
NAPS SUP 3.7-2 **Figure 3.7.1-249 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Partial Column CB case, UP Component**



NAPS SUP 3.7-2 **Figure 3.7.1-250 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column CB case, H1 Component**

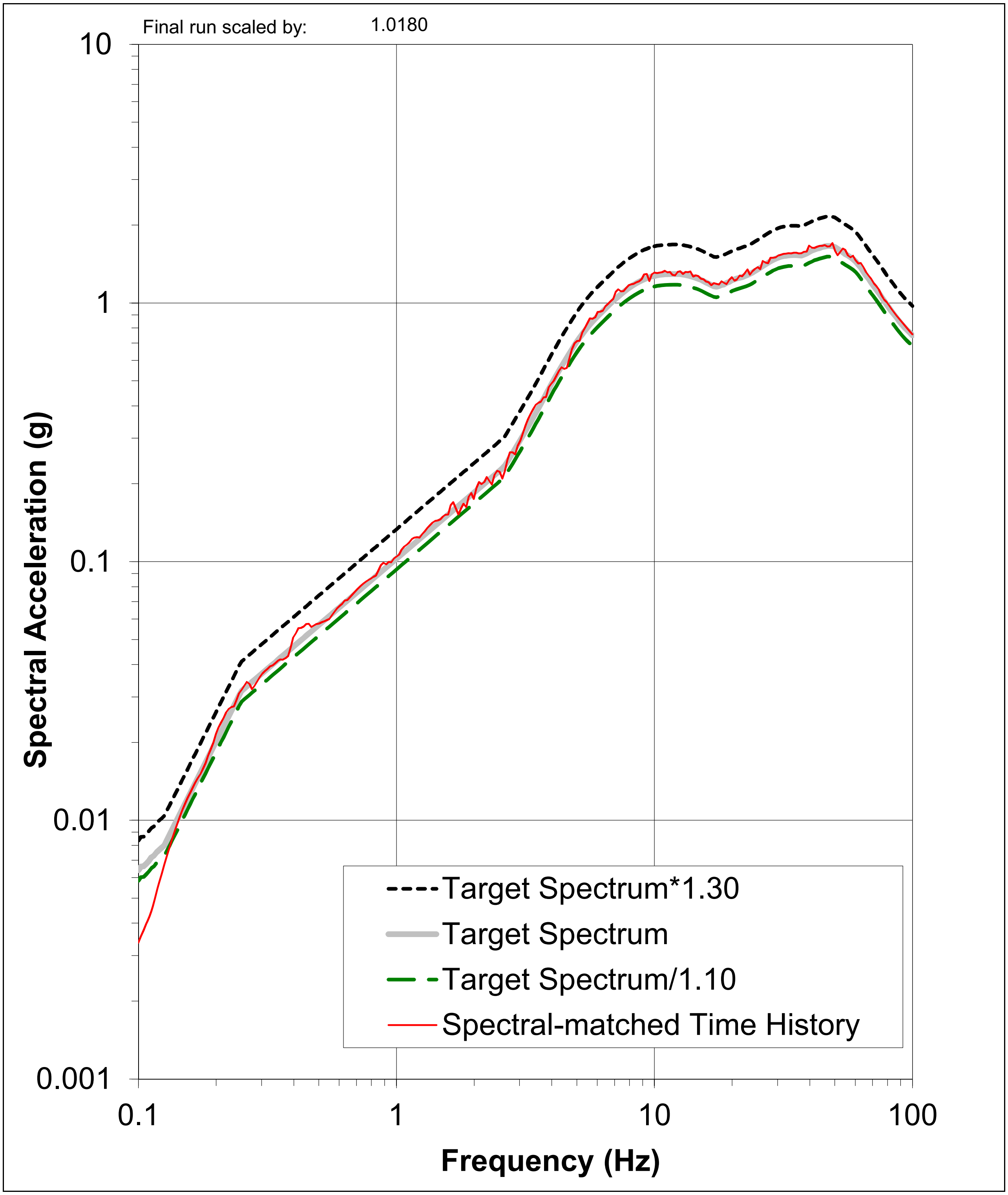


NAPS SUP 3.7-2 **Figure 3.7.1-251 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column CB case, H2 Component**

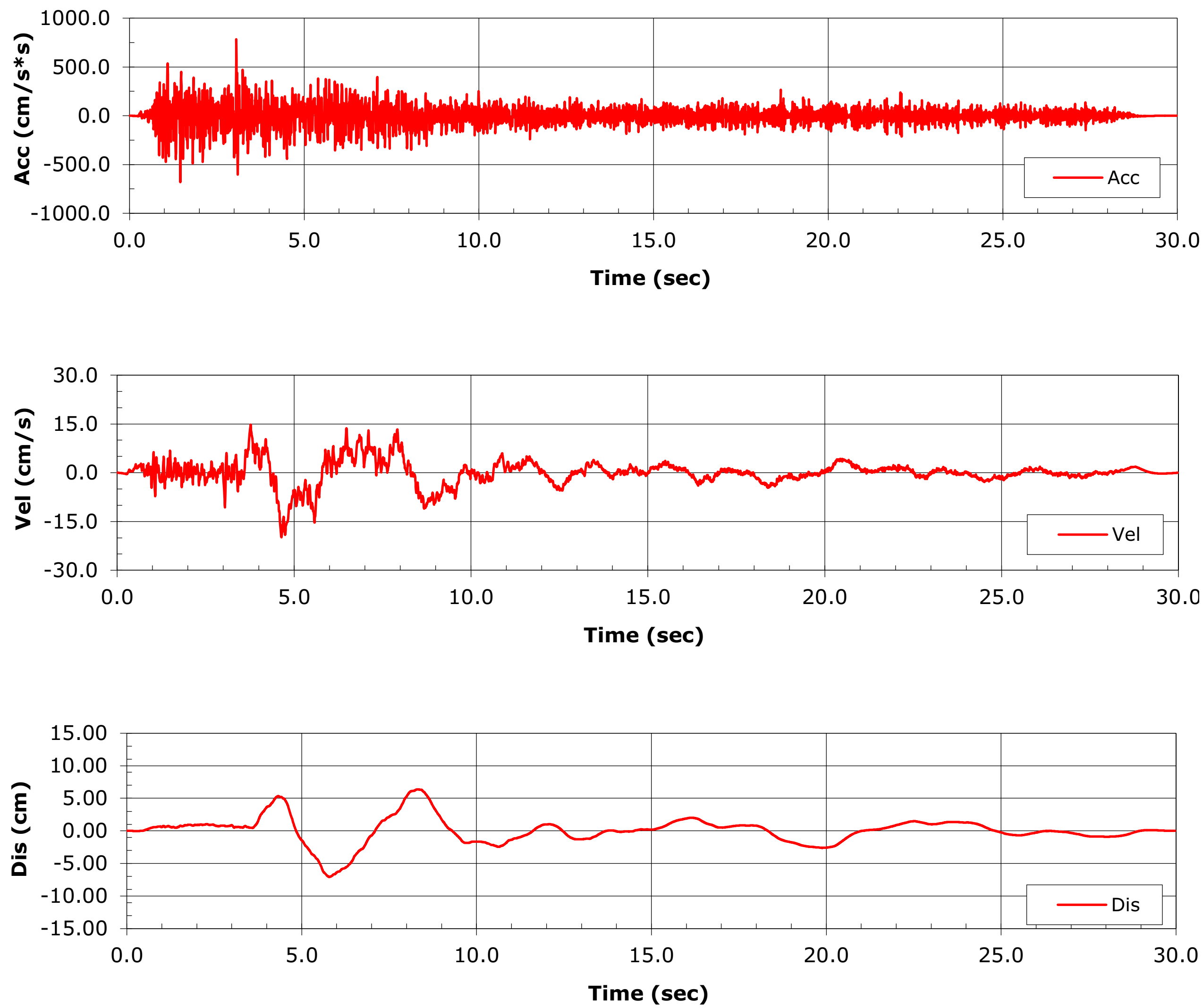


NAPS SUP 3.7-2

Figure 3.7.1-252 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the Full Column CB case, UP Component

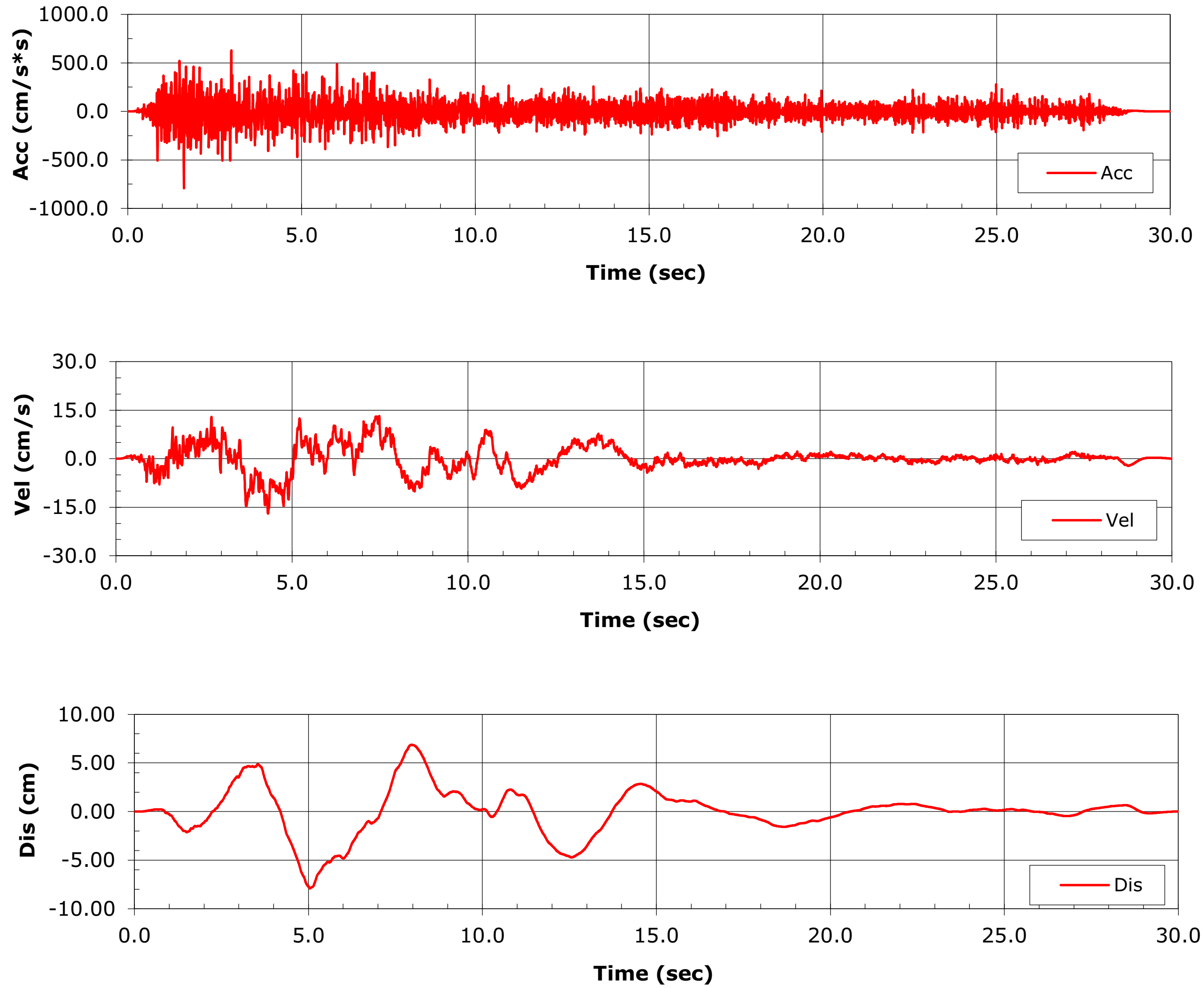


NAPS SUP 3.7-2 **Figure 3.7.1-253 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for CB, H1 Component**



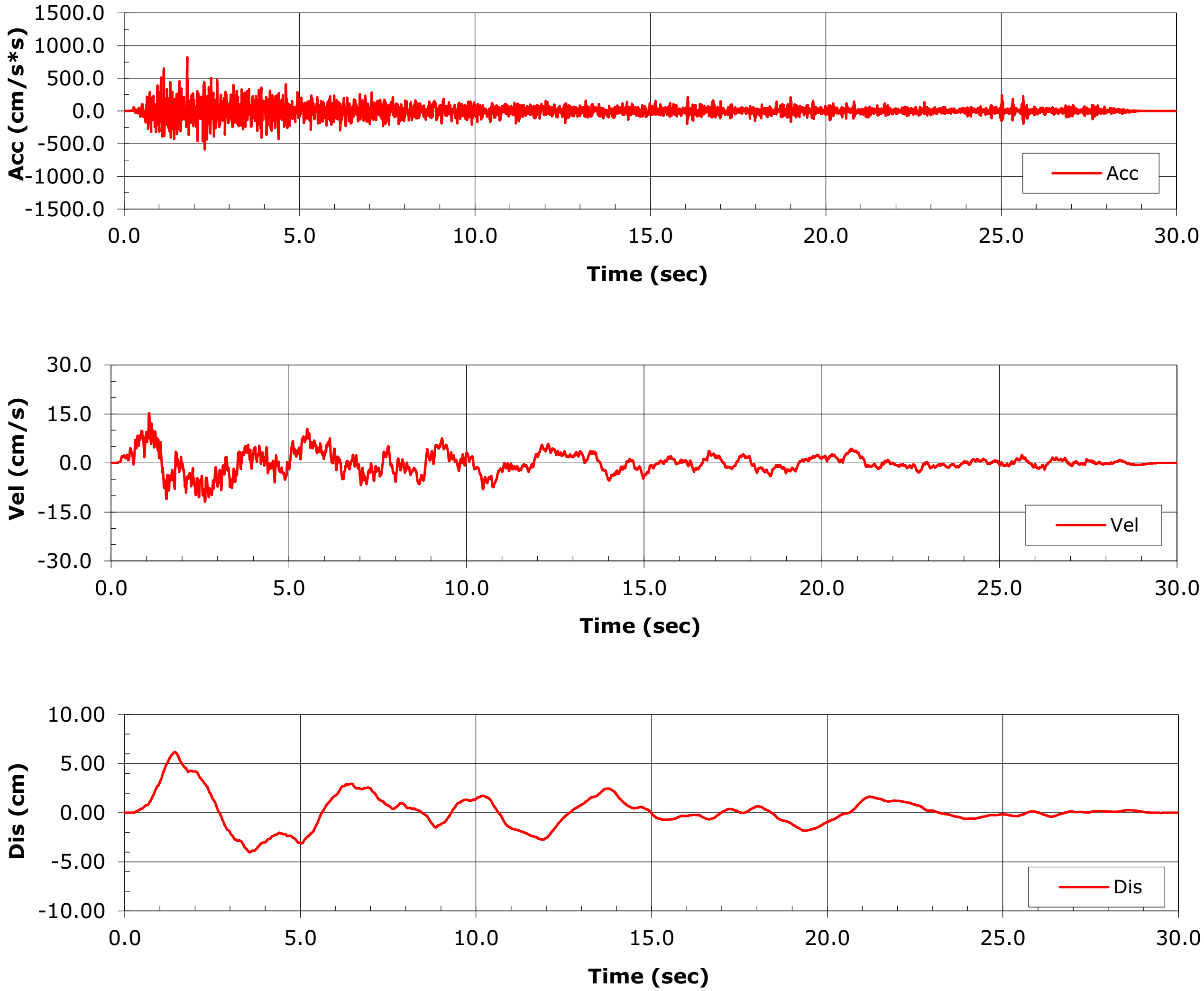
NAPS SUP 3.7-2

Figure 3.7.1-254 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for CB, H2 Component

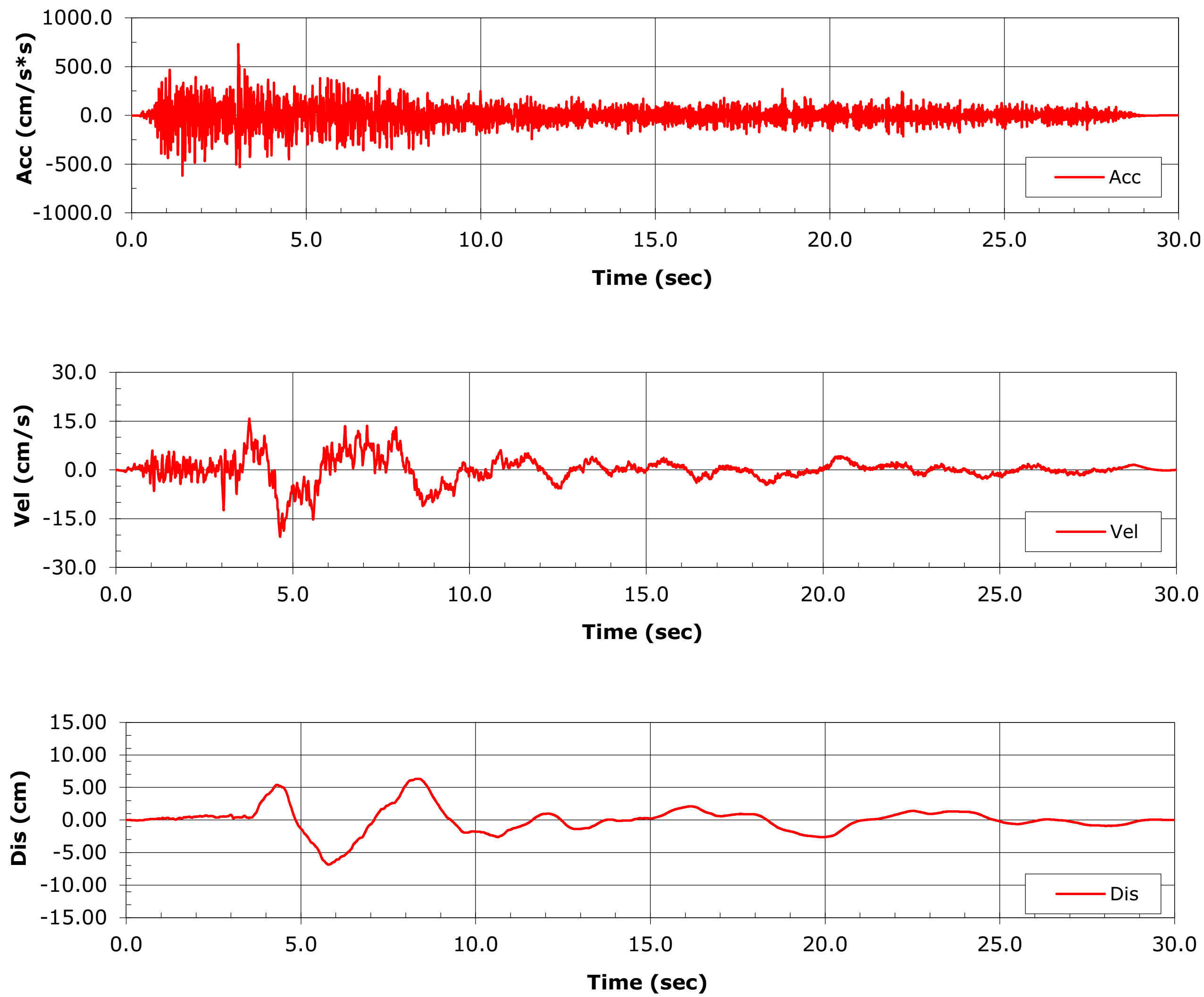


NAPS SUP 3.7-2

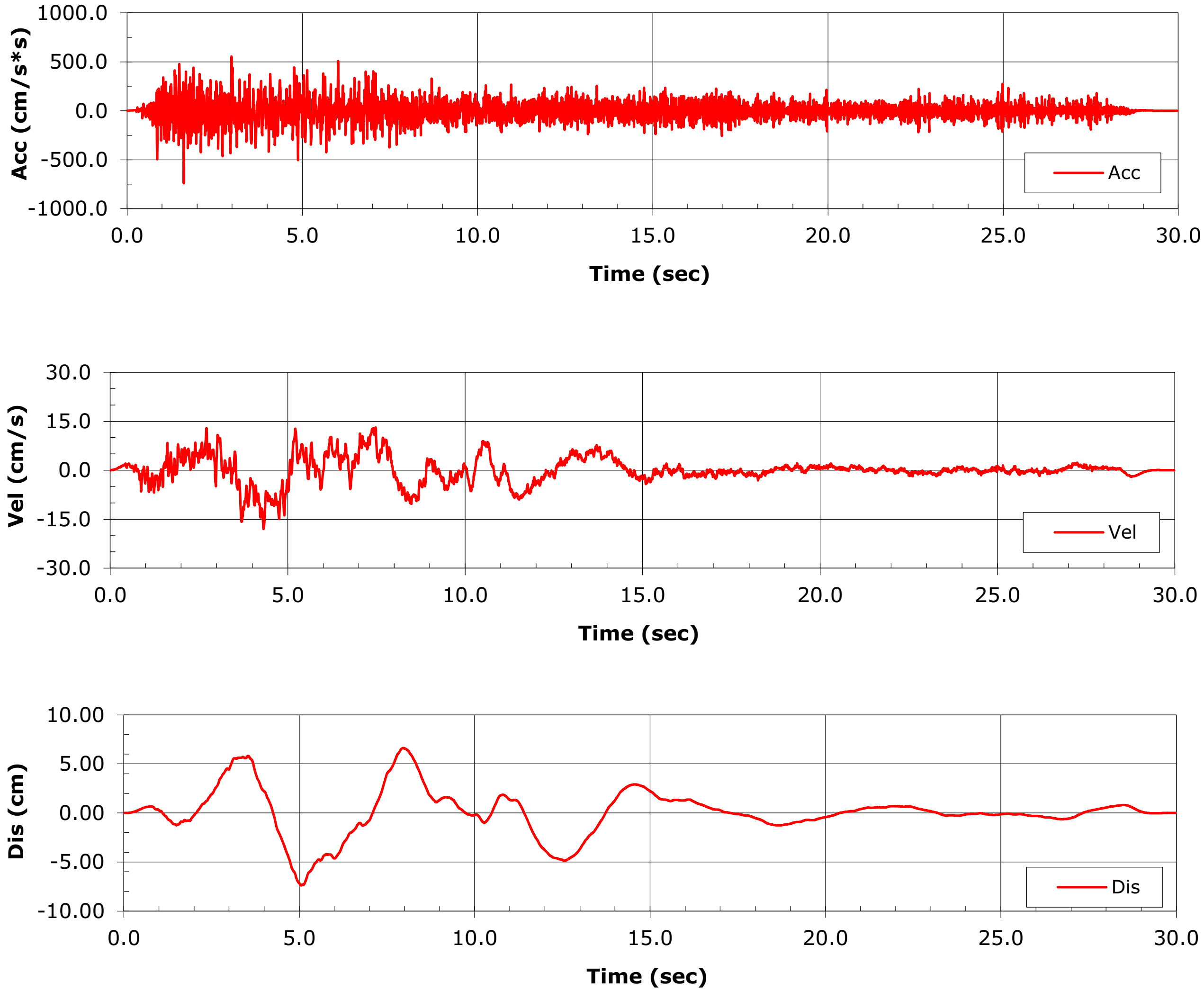
Figure 3.7.1-255 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time-Histories for CB, UP Component



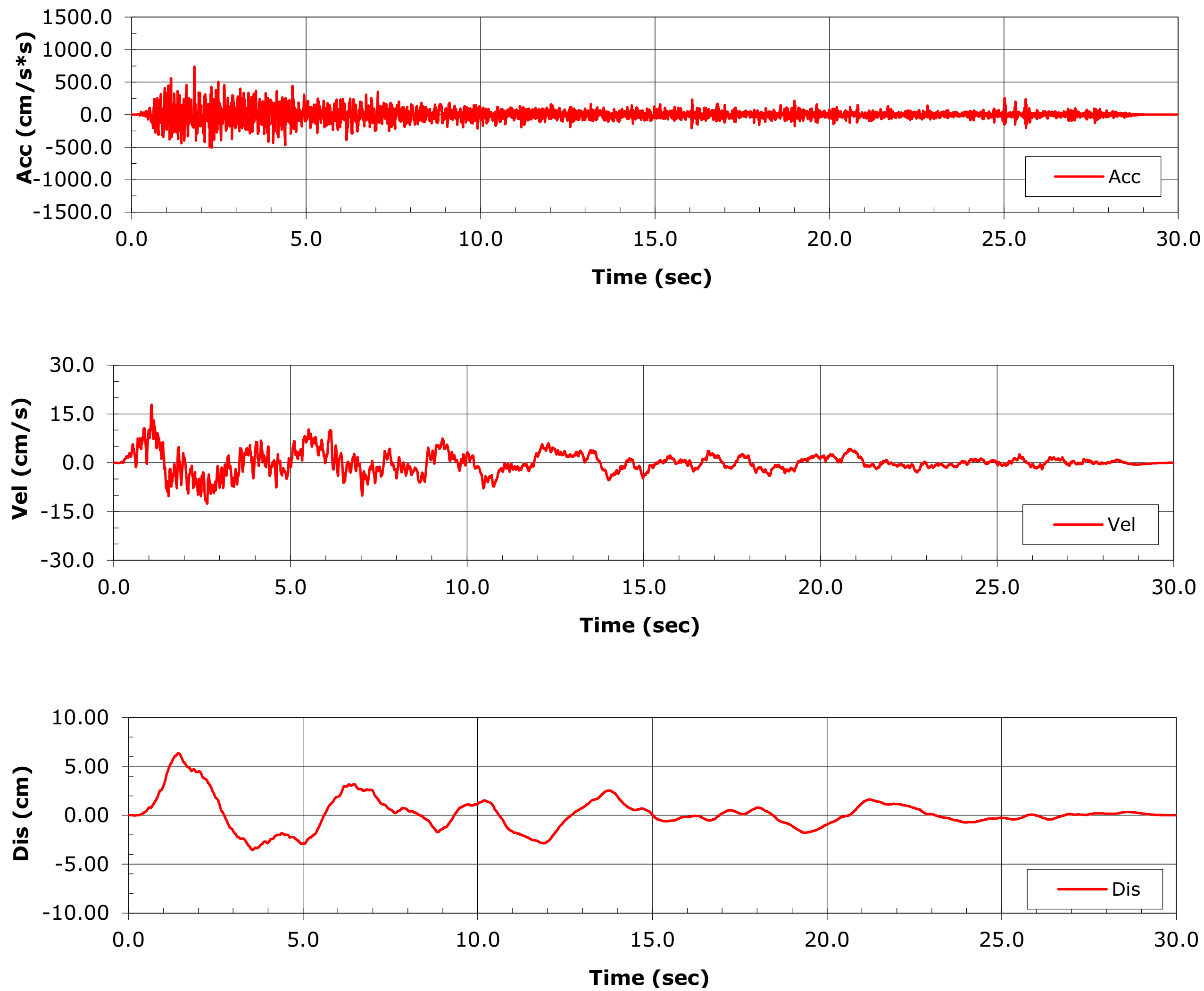
NAPS SUP 3.7-2 **Figure 3.7.1-256 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for CB, H1 Component**



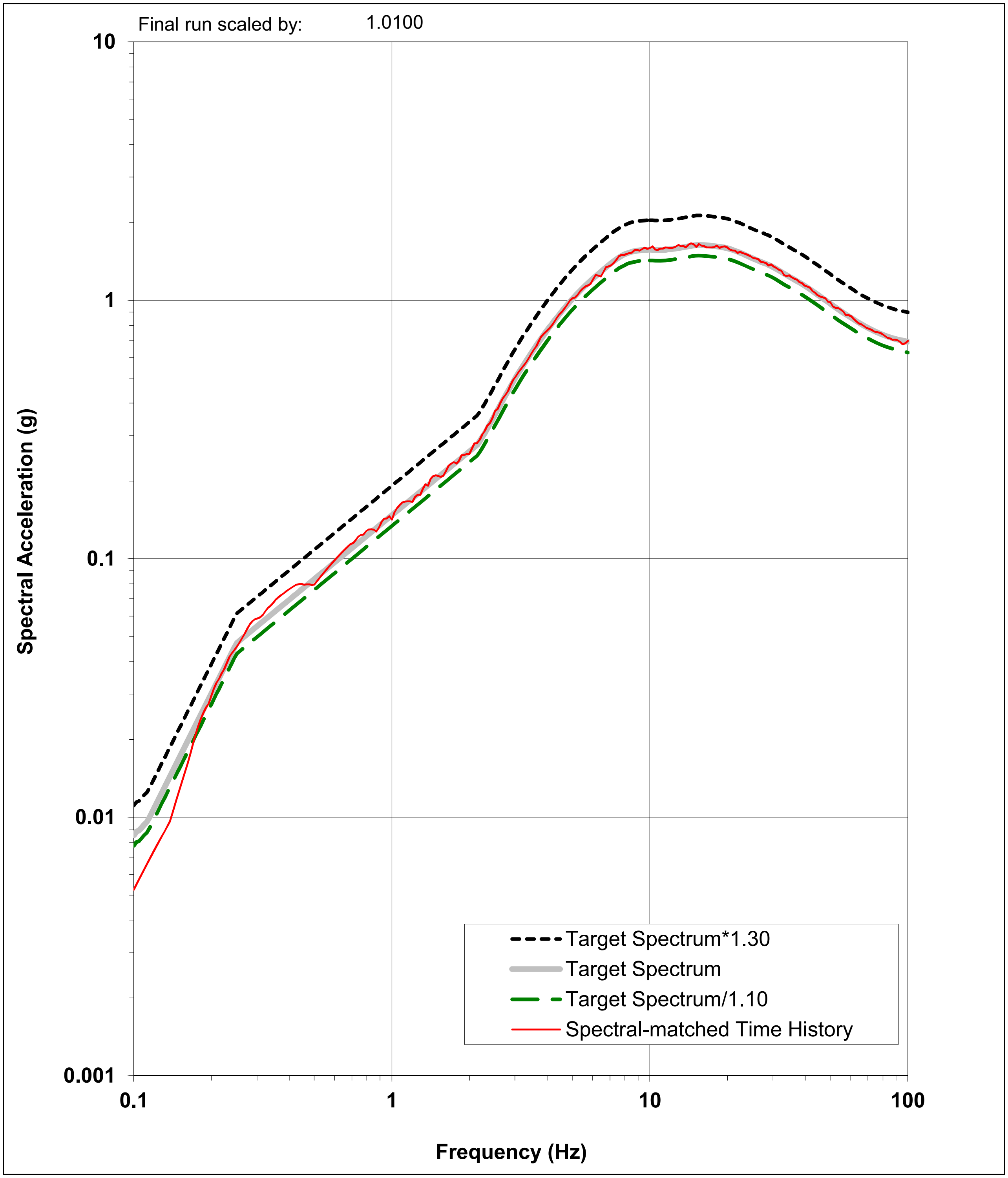
NAPS SUP 3.7-2 **Figure 3.7.1-257 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for CB, H2 Component**



NAPS SUP 3.7-2 **Figure 3.7.1-258 Acceleration, Velocity, and Displacement Spectrally Matched Full Column Outcrop Time-Histories for CB, UP Component**

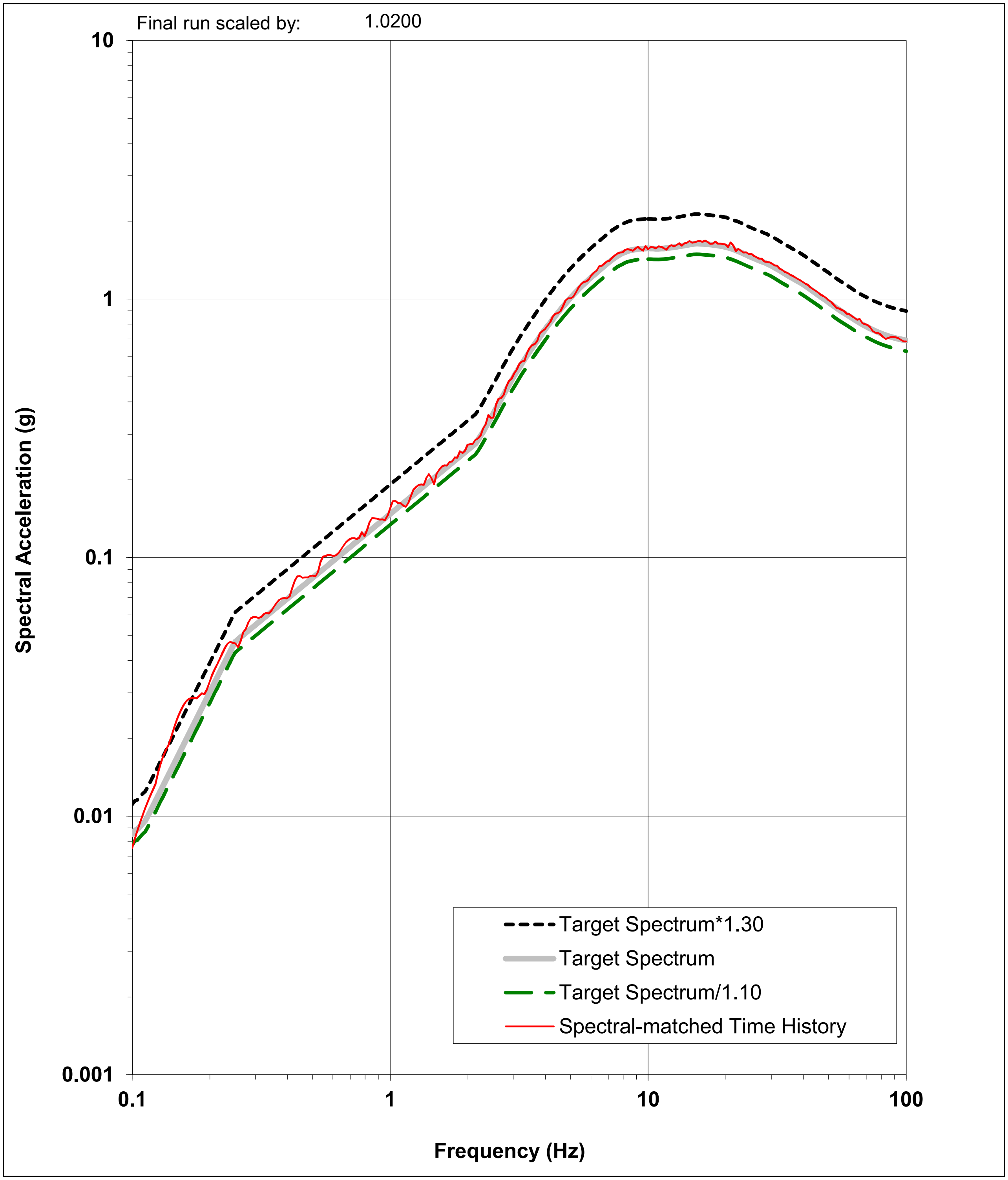


NAPS SUP 3.7-2 **Figure 3.7.1-259 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, H1 Component at Elevation 282 ft**



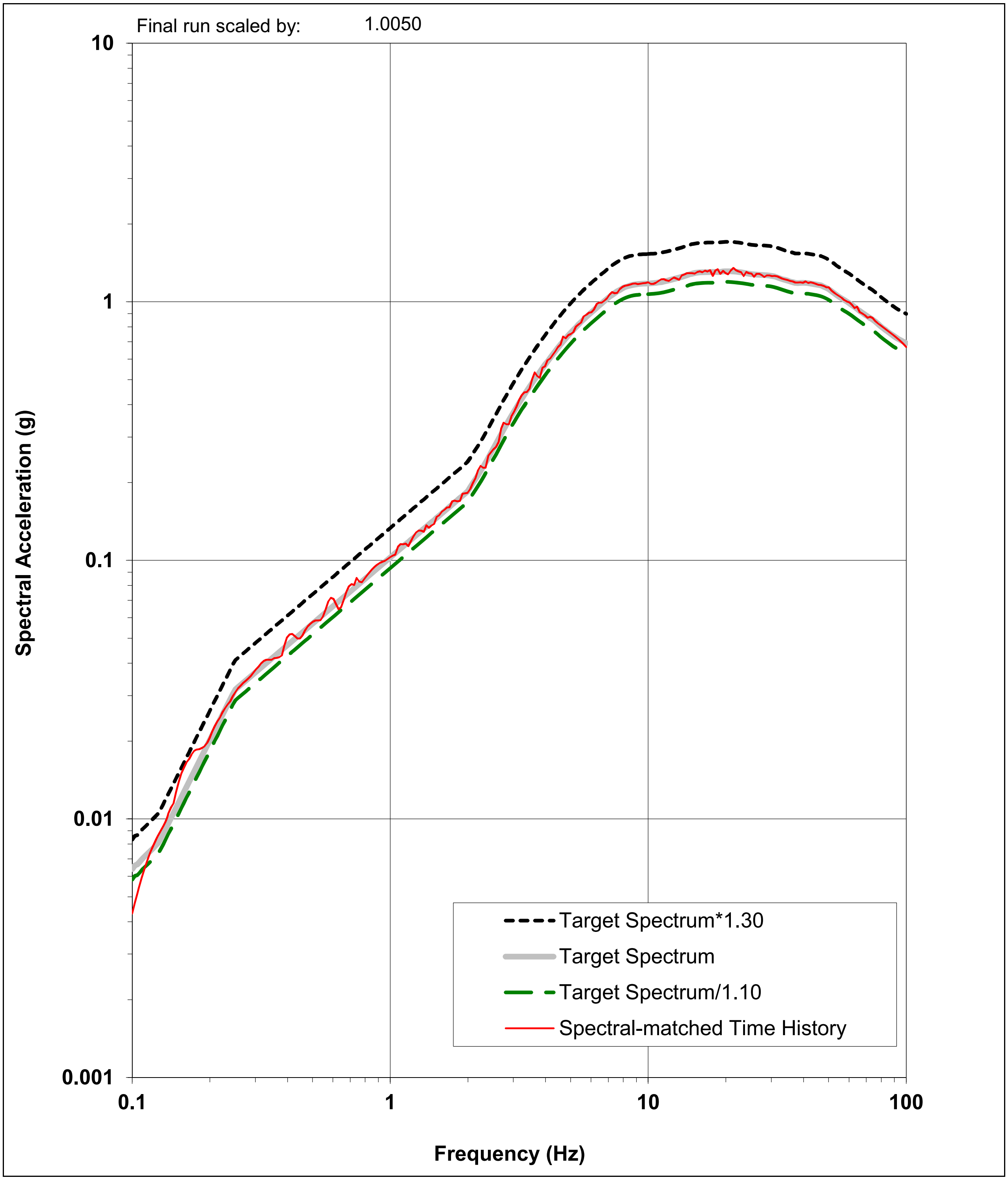
NAPS SUP 3.7-2

Figure 3.7.1-260 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, H2 Component at Elevation 282 ft



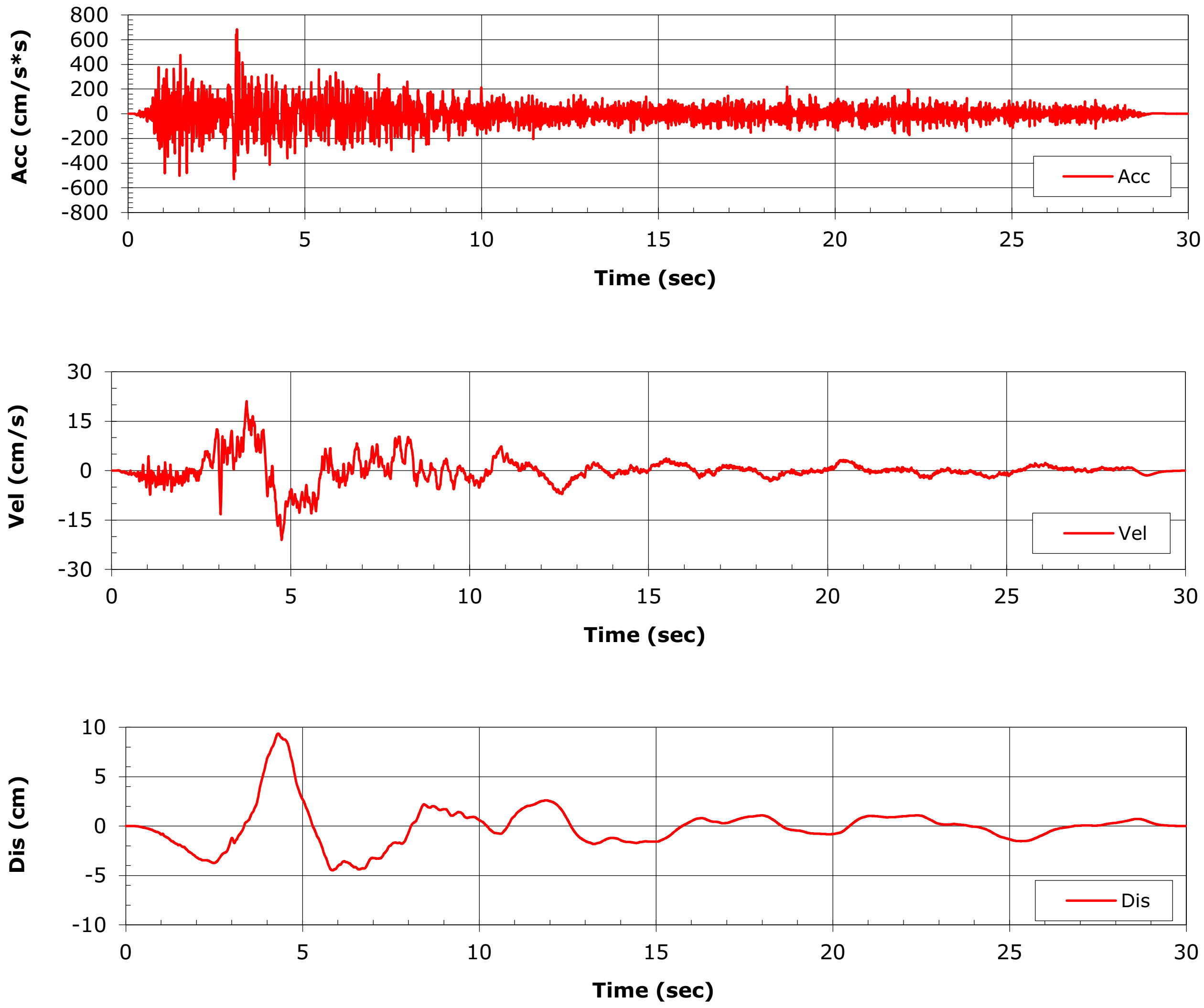
NAPS SUP 3.7-2

Figure 3.7.1-261 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, UP Component at Elevation 282 ft



NAPS SUP 3.7-2

Figure 3.7.1-262 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, H1 Component at Elevation 282 ft



NAPS SUP 3.7-2 **Figure 3.7.1-263 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, H2 Component at Elevation 282 ft**

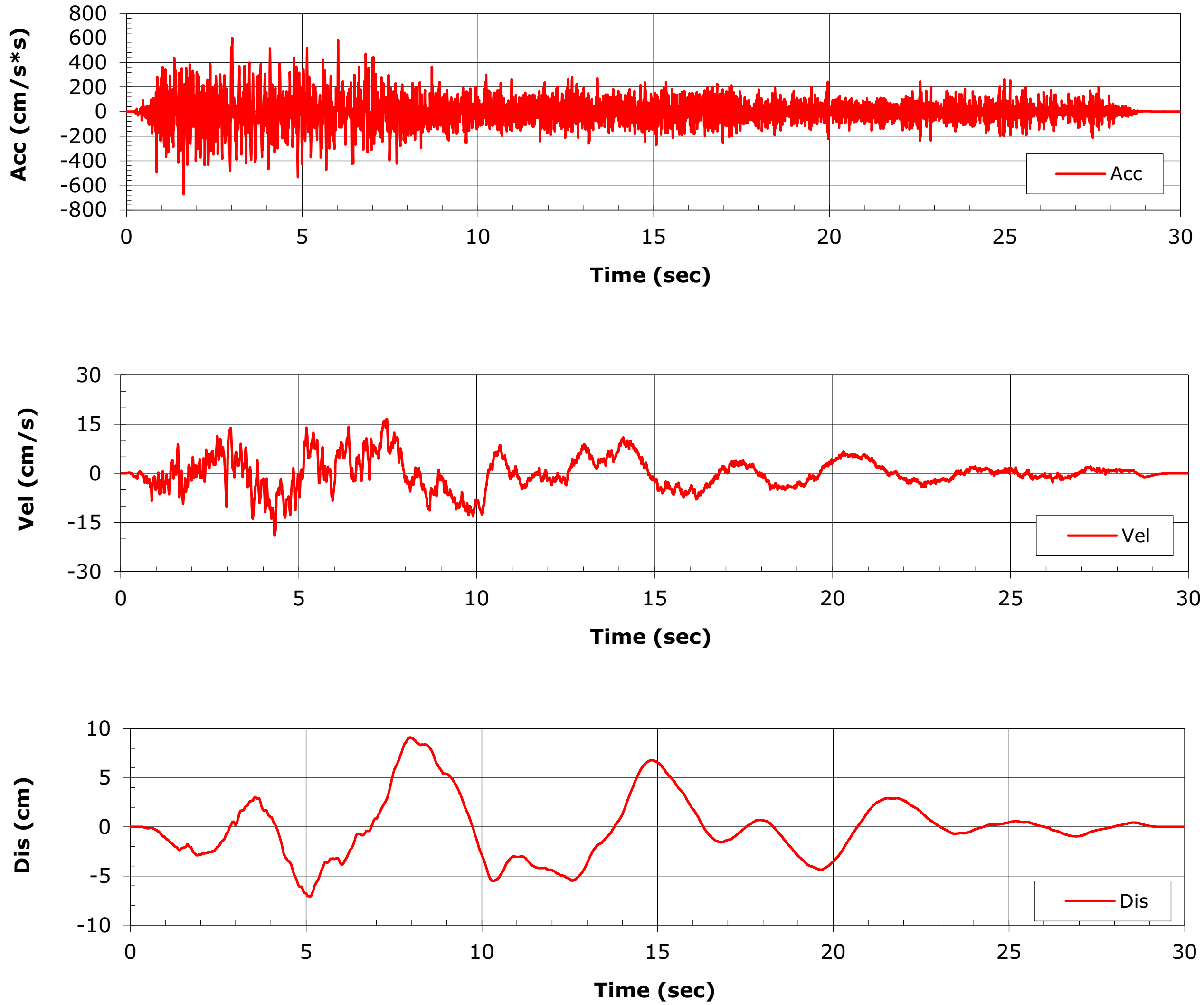
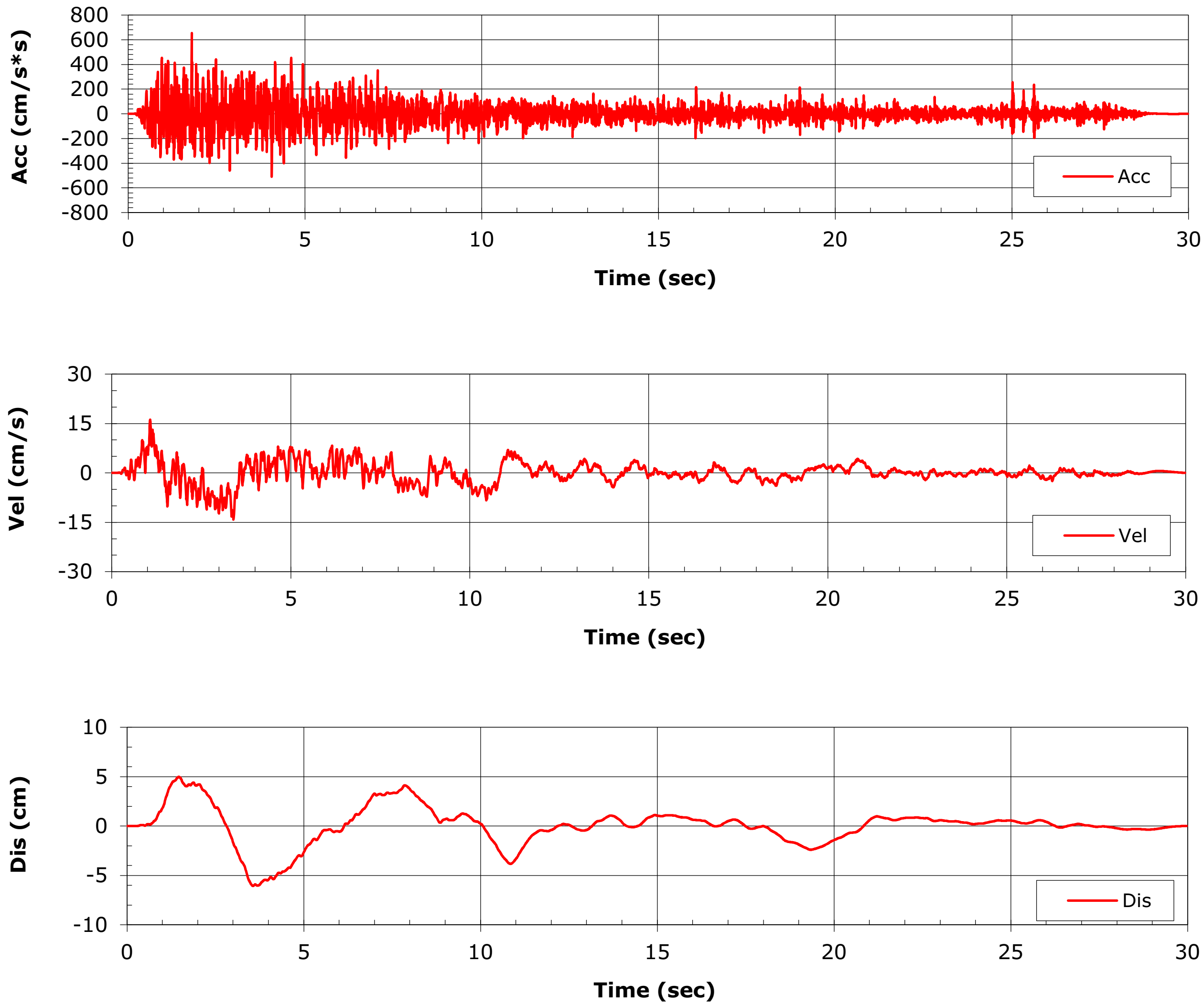
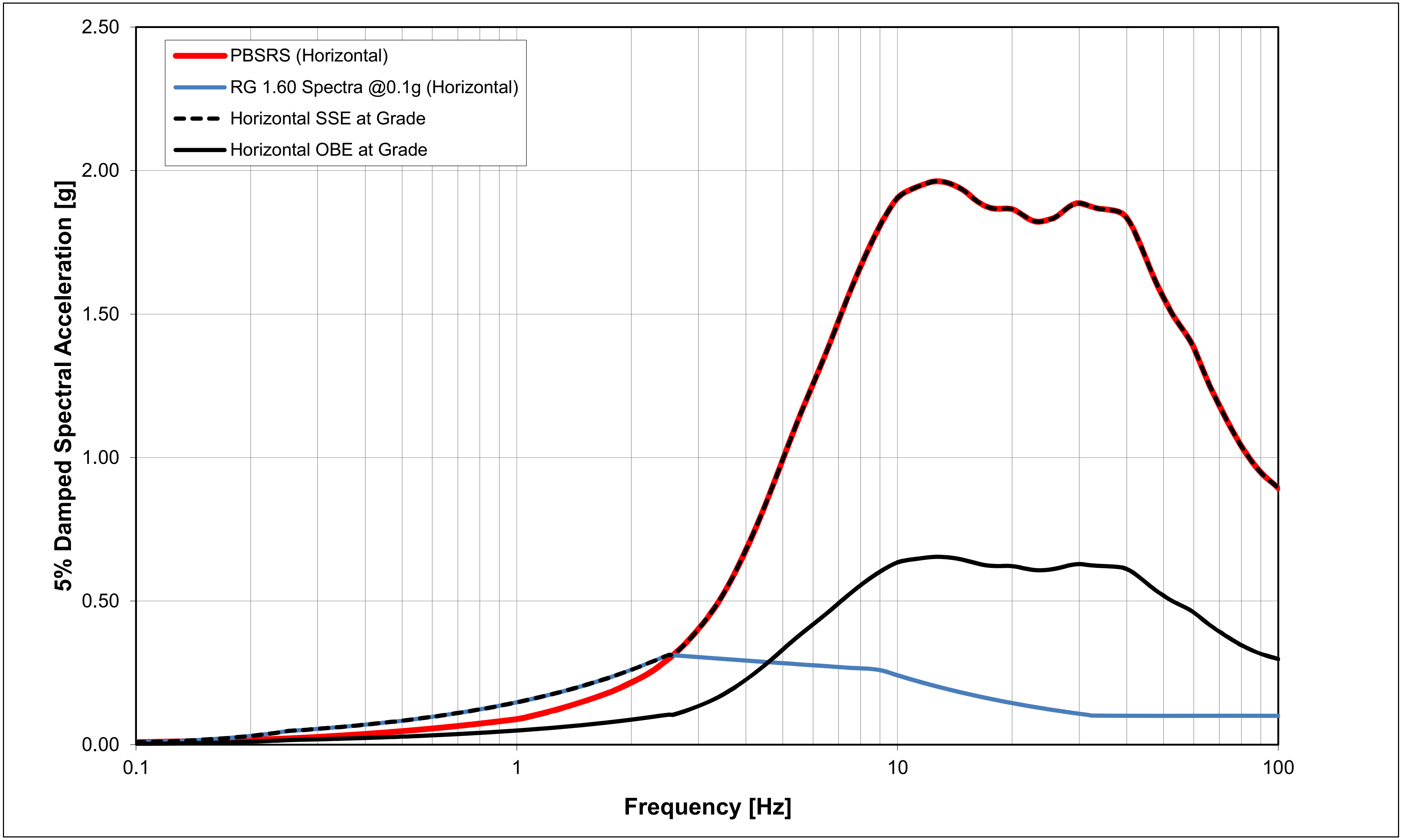


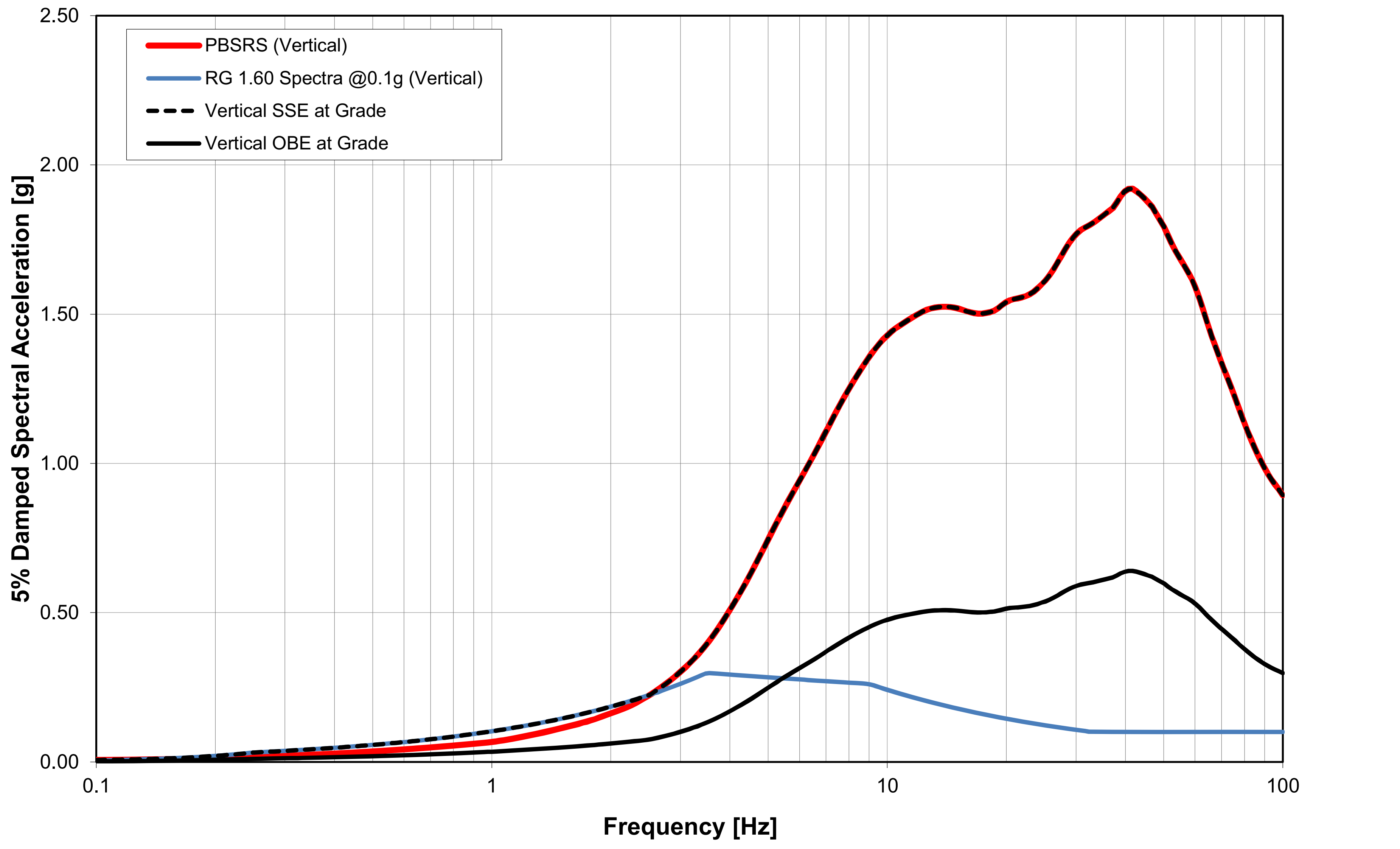
Figure 3.7.1-264 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, UP Component at Elevation 282 ft



NAPS SUP 3.7-2 **Figure 3.7.1-265 Development of Horizontal Site-Dependent SSE and OBE at Grade**



NAPS SUP 3.7-2 **Figure 3.7.1-266 Development of Vertical Site-Dependent SSE and OBE at Grade**



NAPS SUP 3.7-2 **Figure 3.7.1-267 Site-Dependent SSE and OBE at Grade**

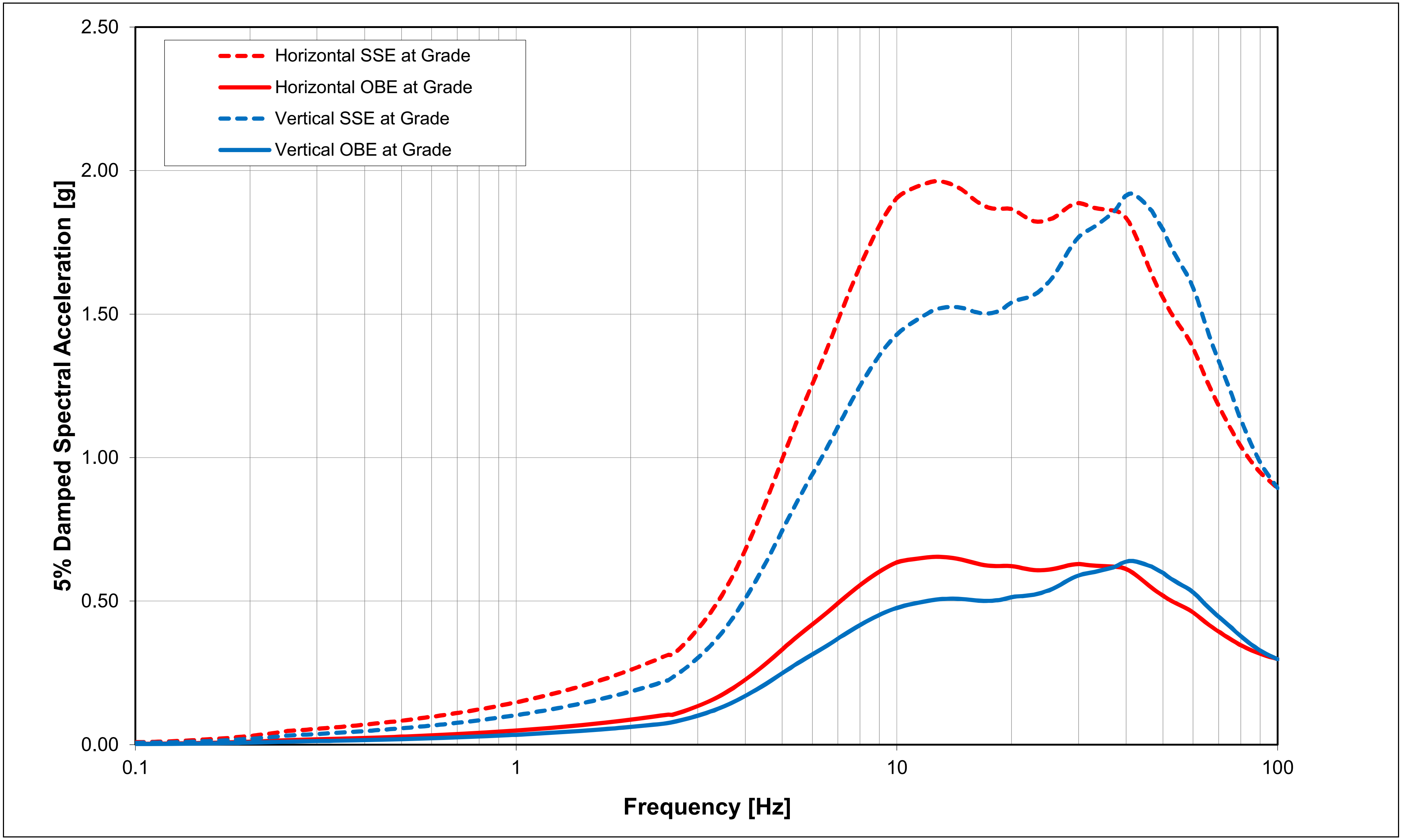
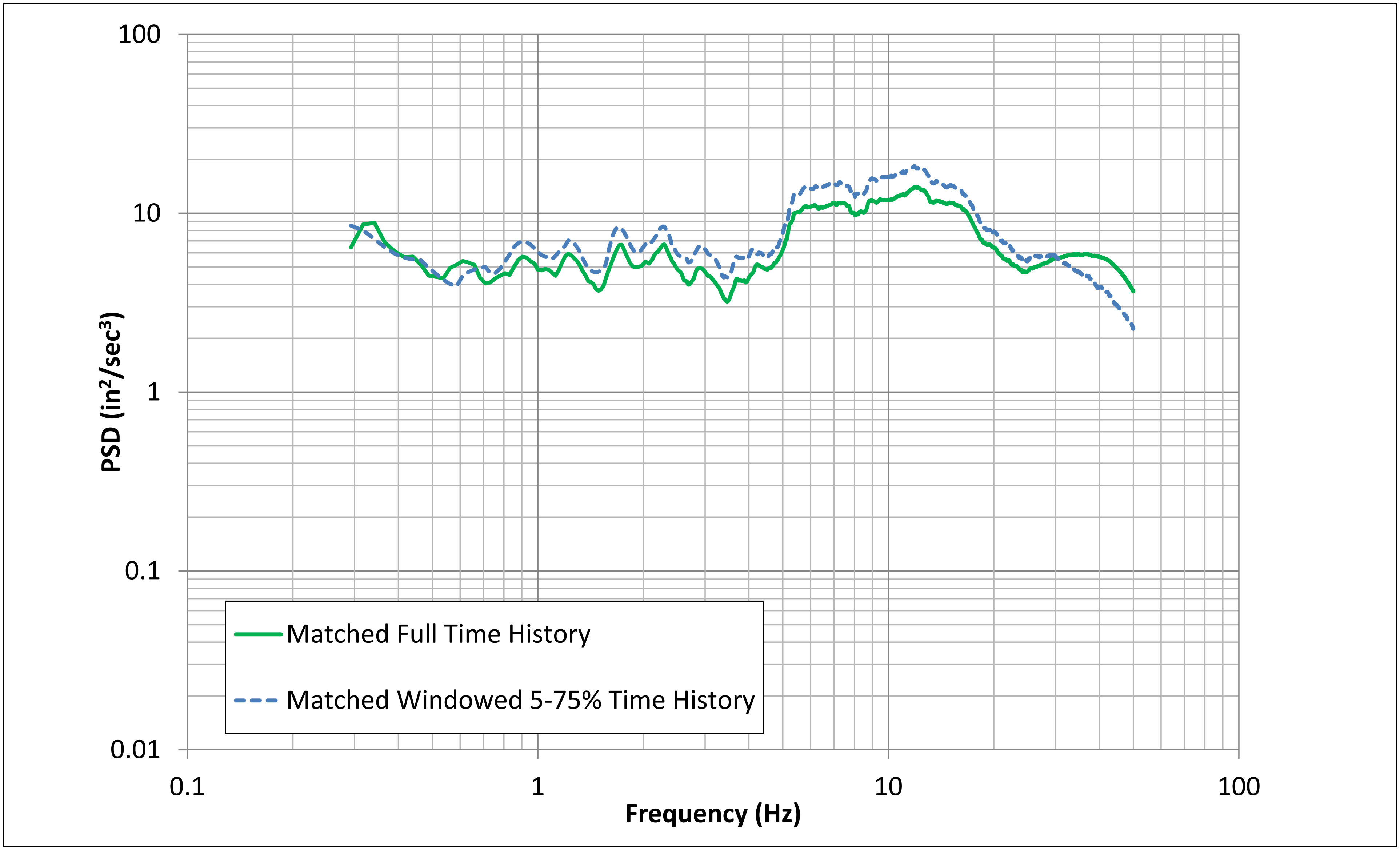


Figure 3.7.1-268 PSD for the H1 Component of the RB/FB Partial Profile Spectrum Compatible Acceleration Time History



NAPS SUP 3.7-2 **Figure 3.7.1-269 PSD for the H2 Component of the RB/FB Partial Profile Spectrum Compatible Acceleration Time History**

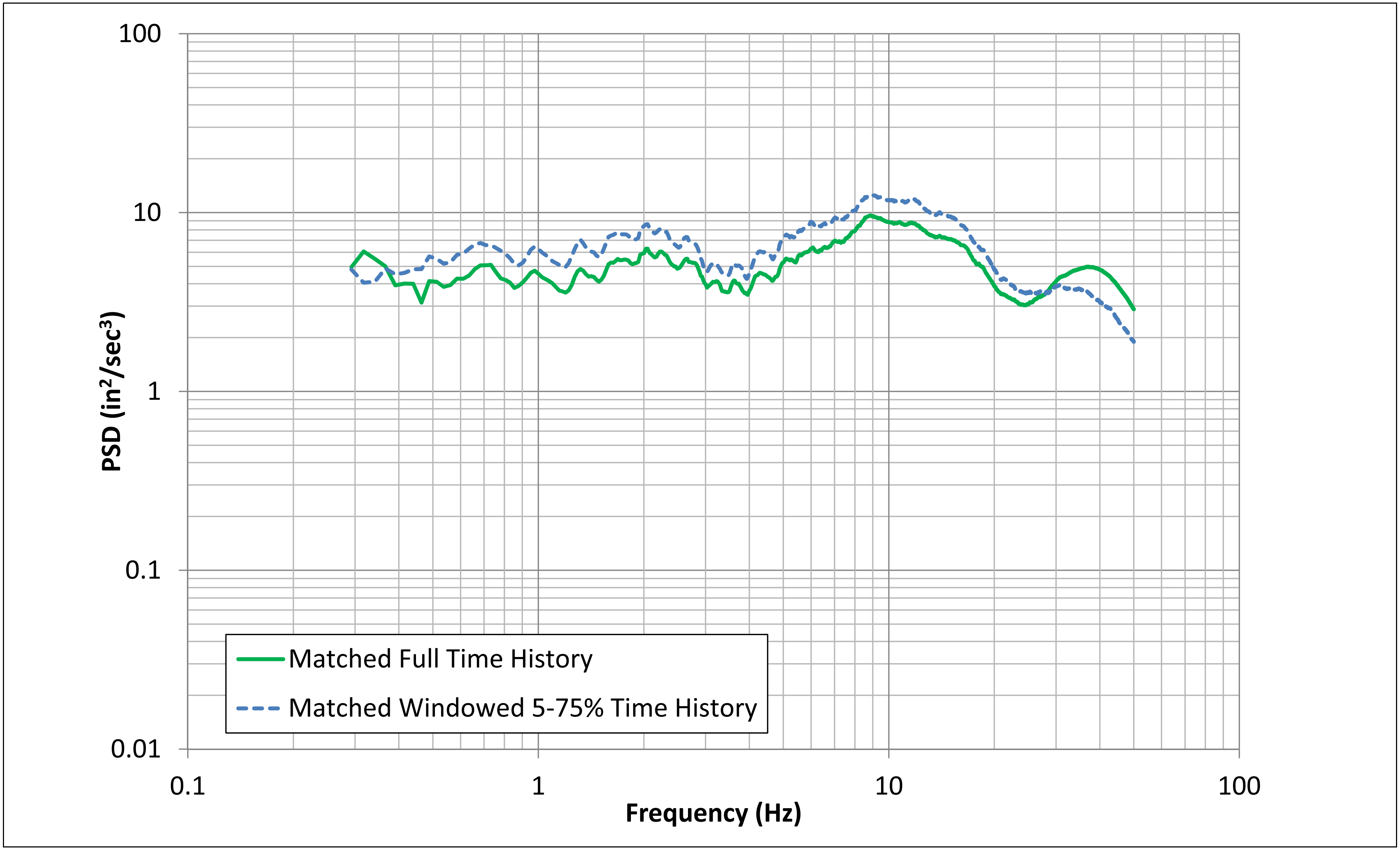
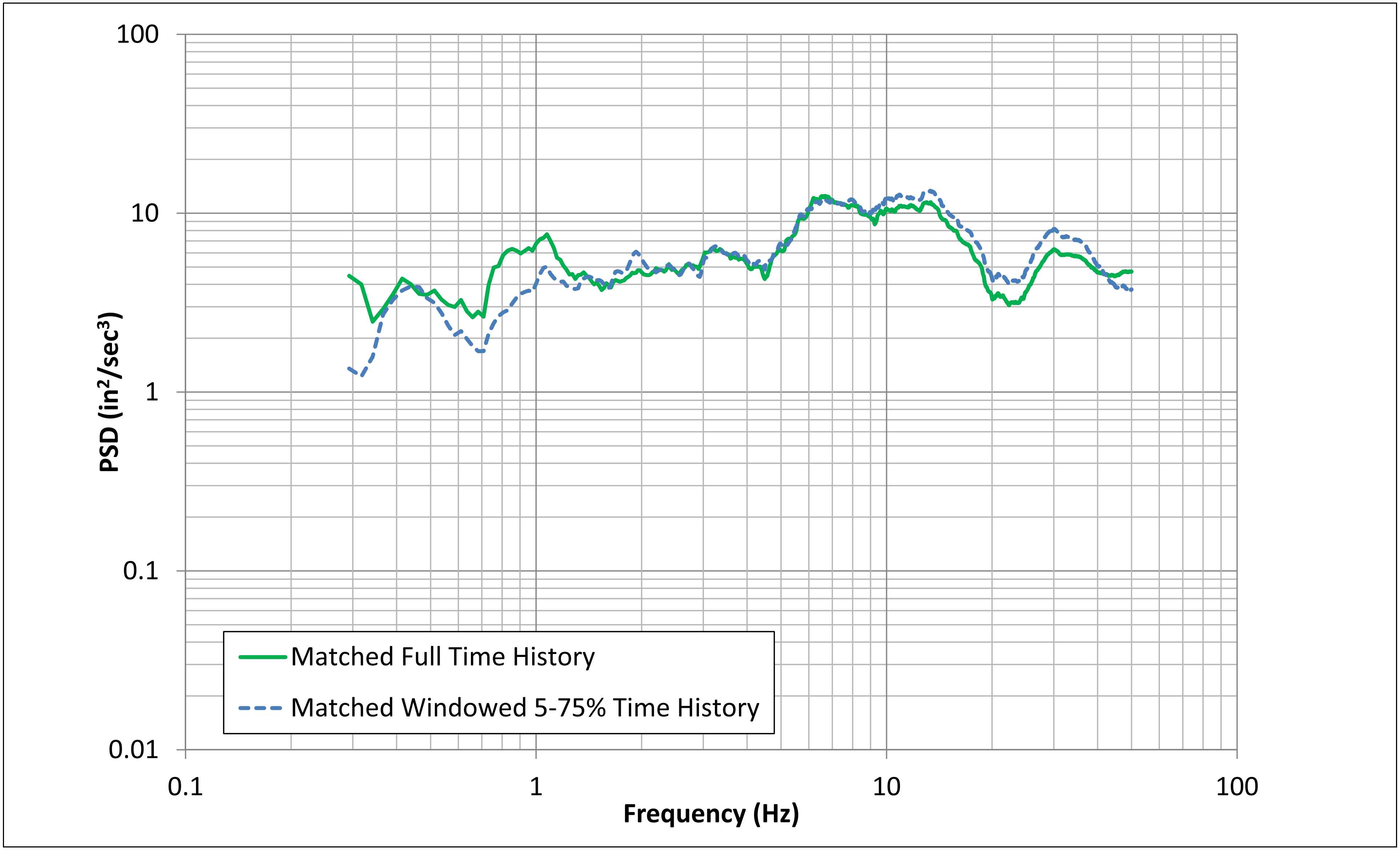


Figure 3.7.1-270 PSD for the UP Component of the RB/FB Partial Profile Spectrum Compatible Acceleration Time History



NAPS SUP 3.7-2

Figure 3.7.1-271 PSD for the H1 Component of the RB/FB Full Profile Spectrum Compatible Acceleration Time History

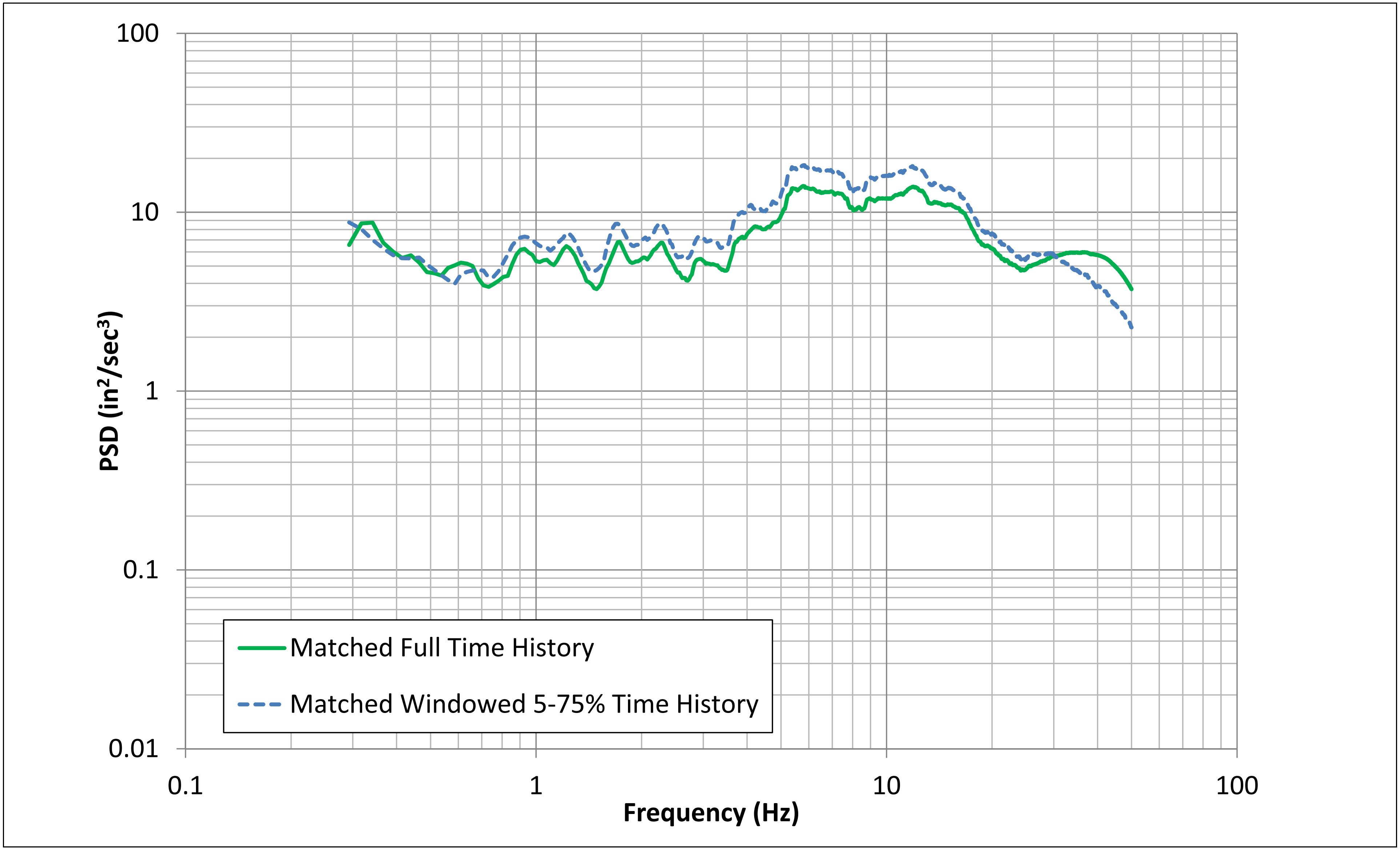
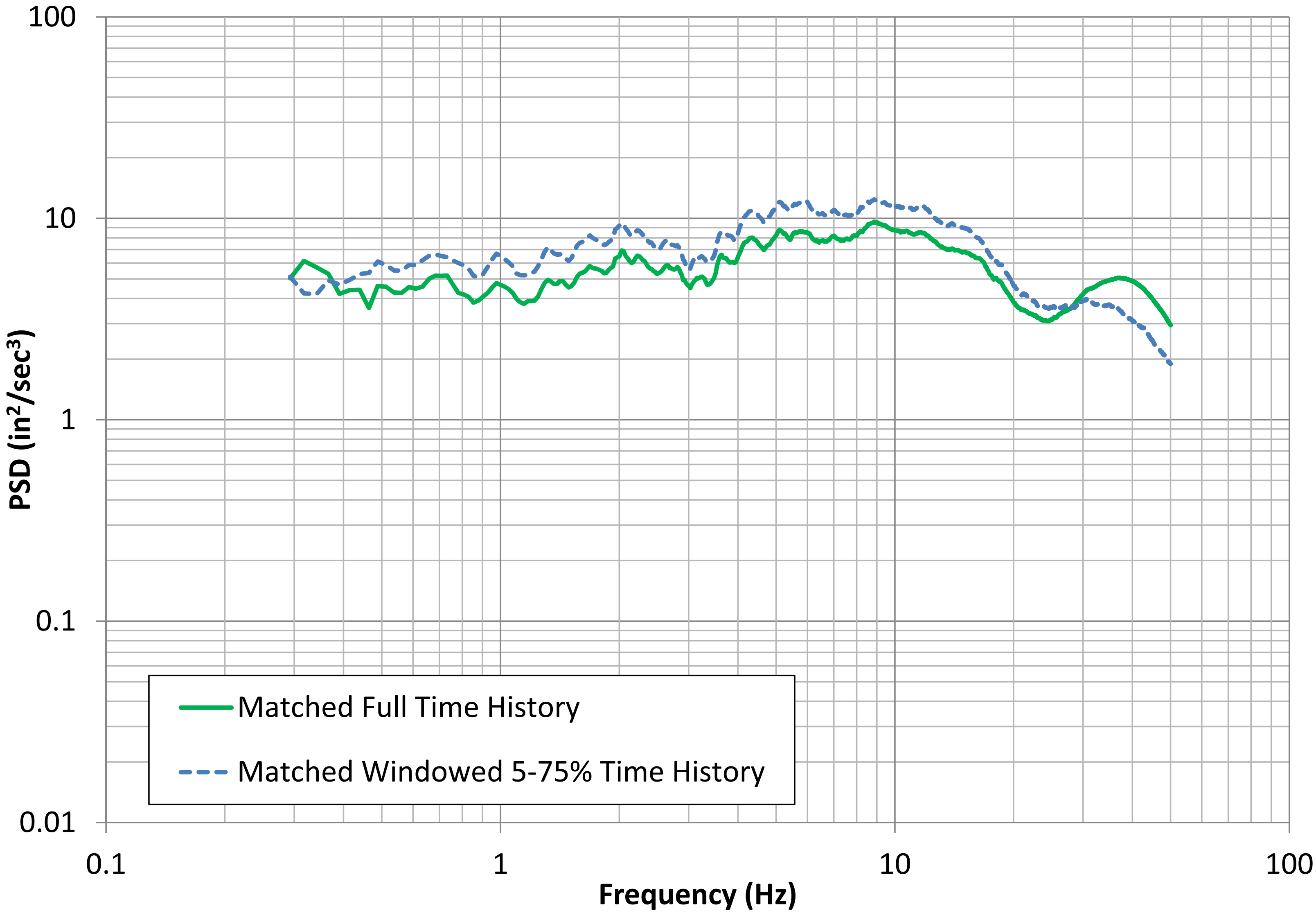
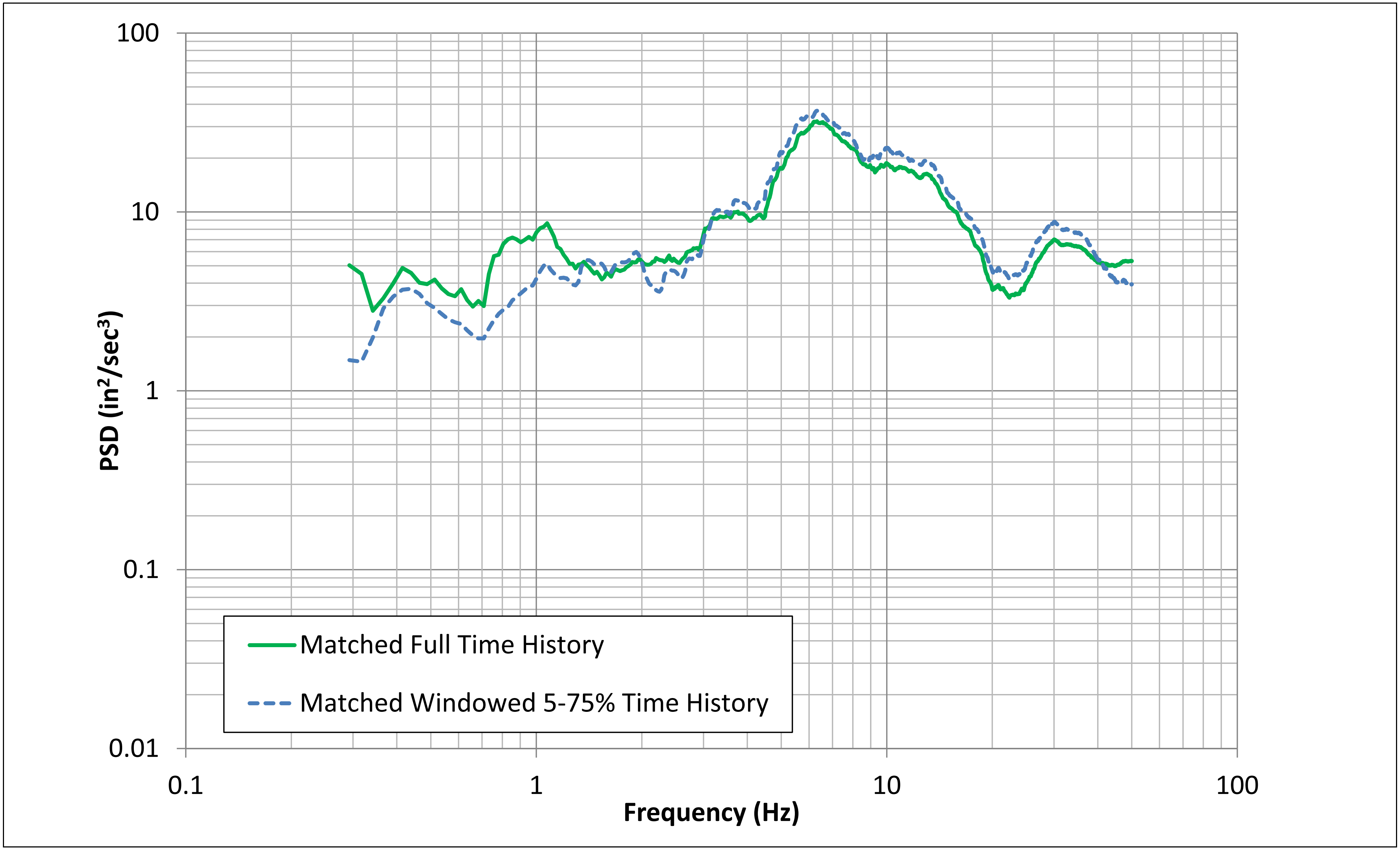


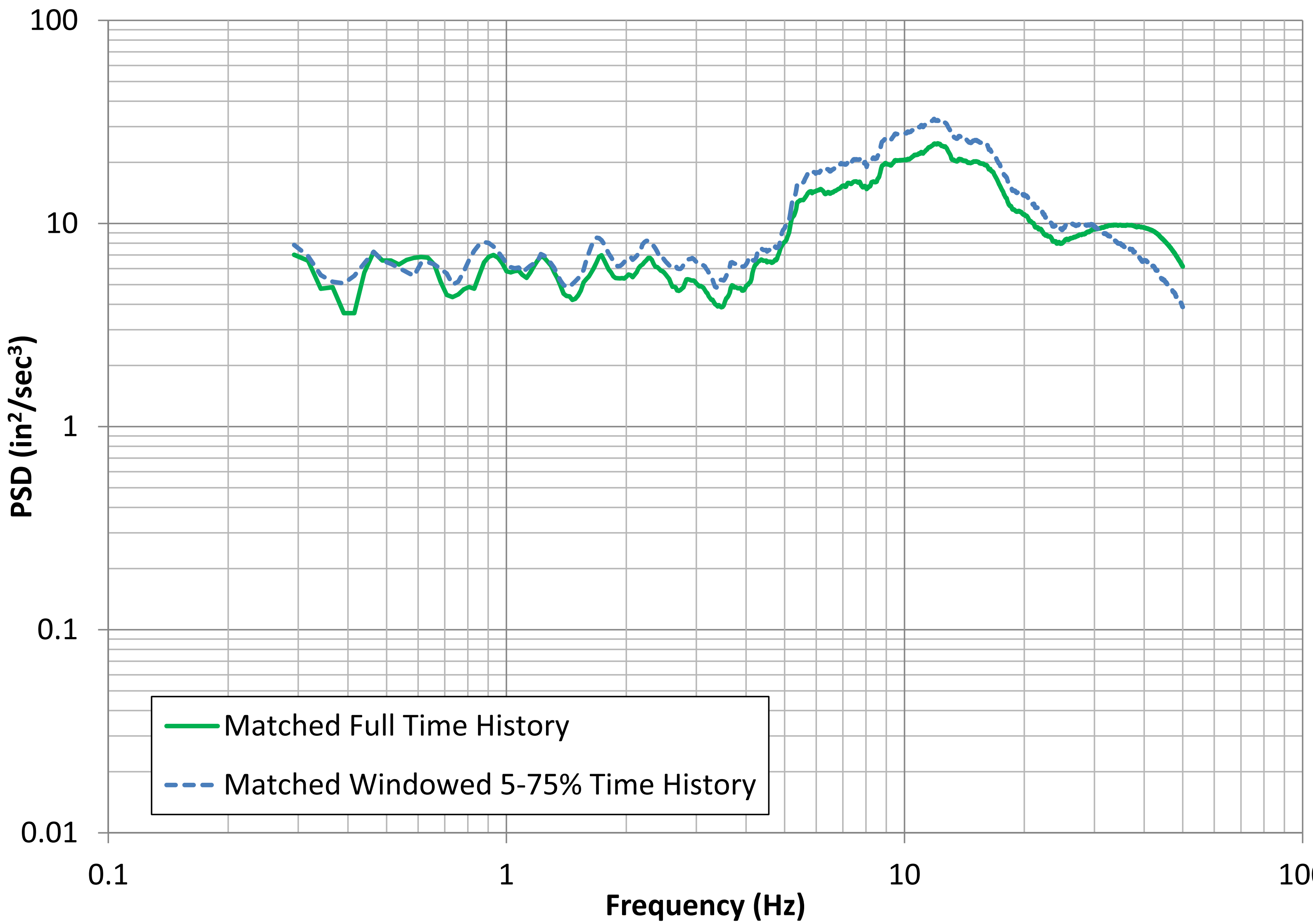
Figure 3.7.1-272 PSD for the H2 Component of the RB/FB Full Profile Spectrum Compatible Acceleration Time History



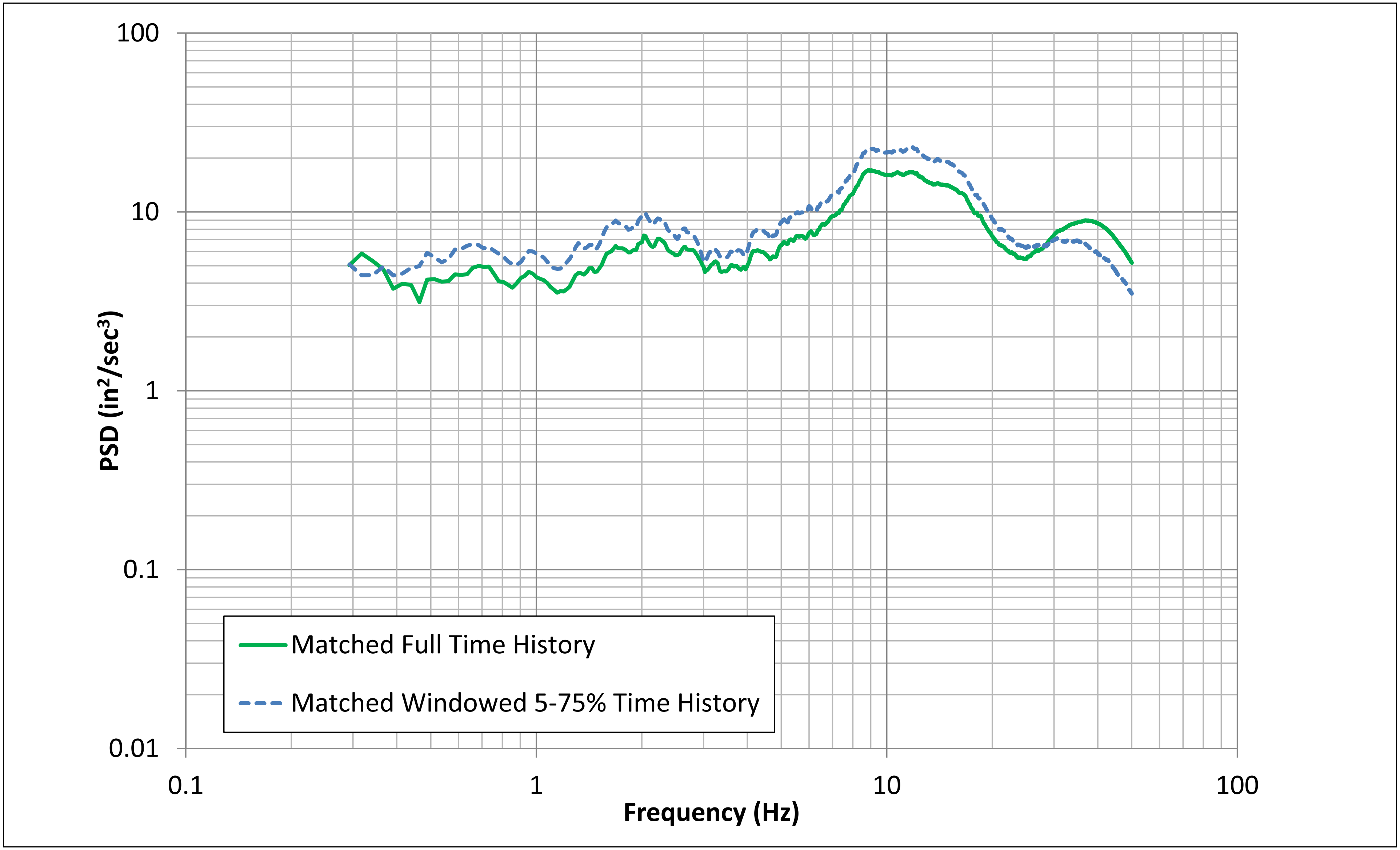
NAPS SUP 3.7-2 **Figure 3.7.1-273 PSD for the UP Component of the RB/FB Full Profile Spectrum Compatible Acceleration Time History**



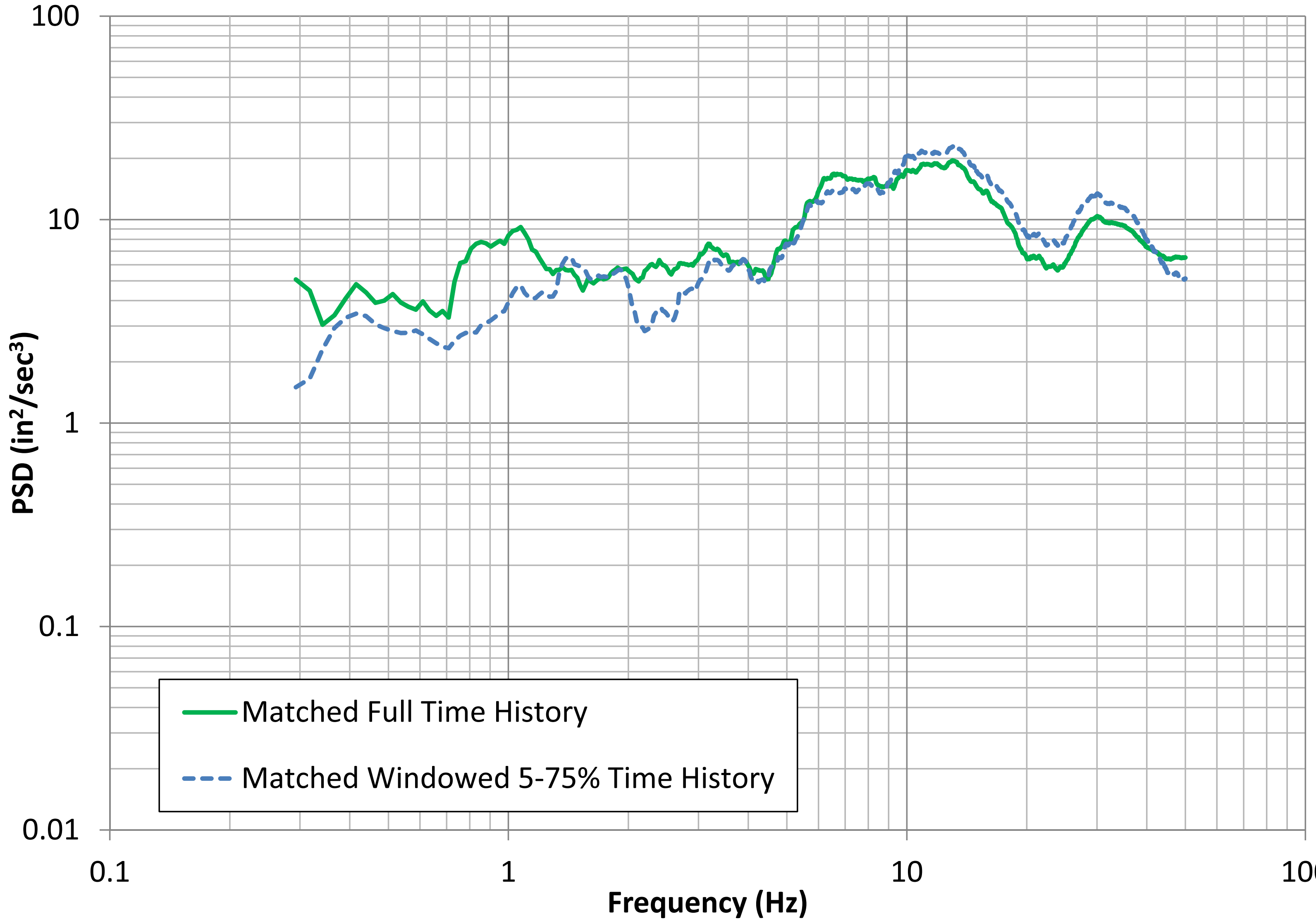
NAPS SUP 3.7-2 **Figure 3.7.1-274 PSD for the H1 Component of the CB Partial Profile Spectrum Compatible Acceleration Time History**



NAPS SUP 3.7-2 **Figure 3.7.1-275 PSD for the H2 Component of the CB Partial Profile Spectrum Compatible Acceleration Time History**

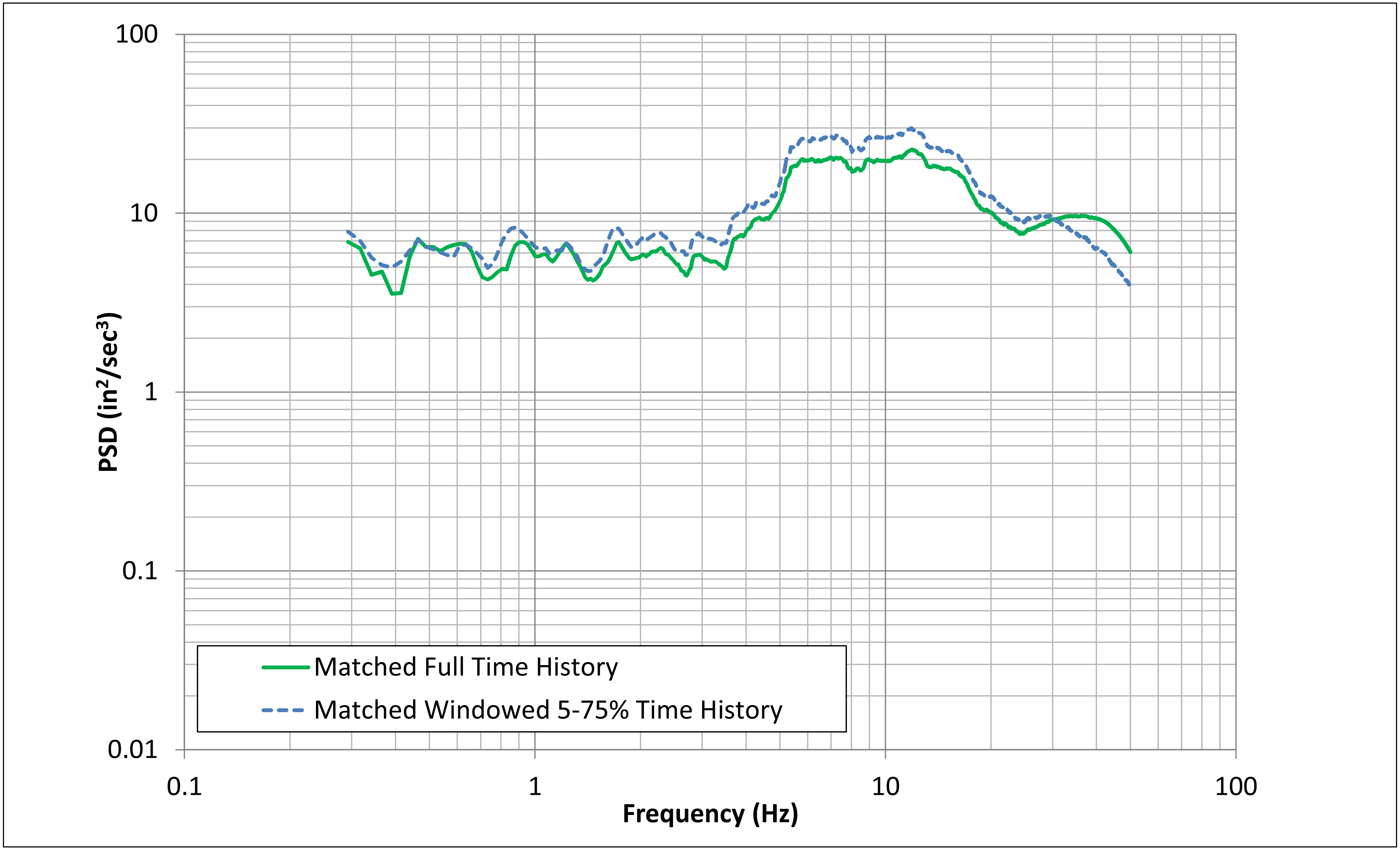


NAPS SUP 3.7-2 **Figure 3.7.1-276 PSD for the UP Component of the CB Partial Profile Spectrum Compatible Acceleration Time History**

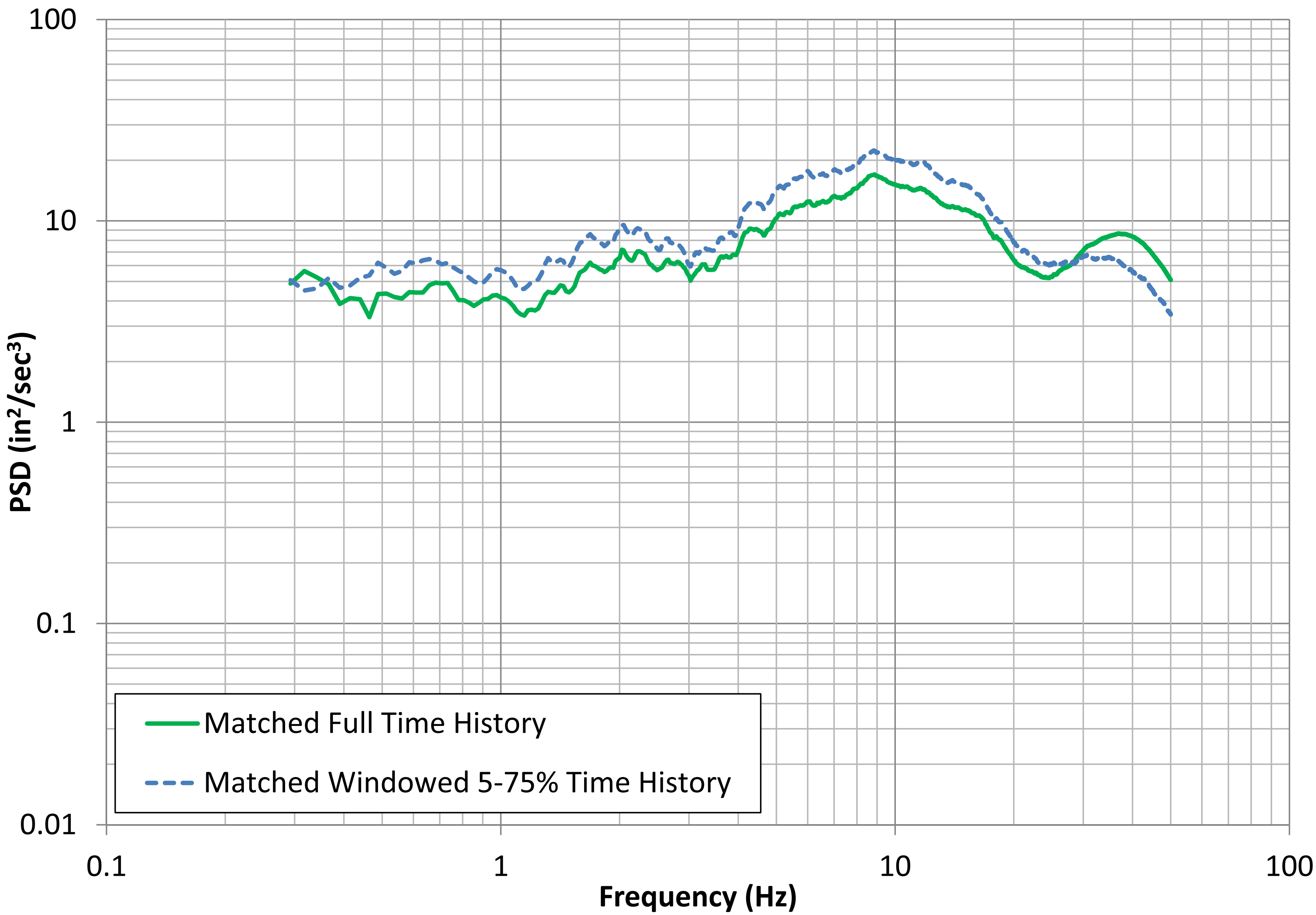


NAPS SUP 3.7-2

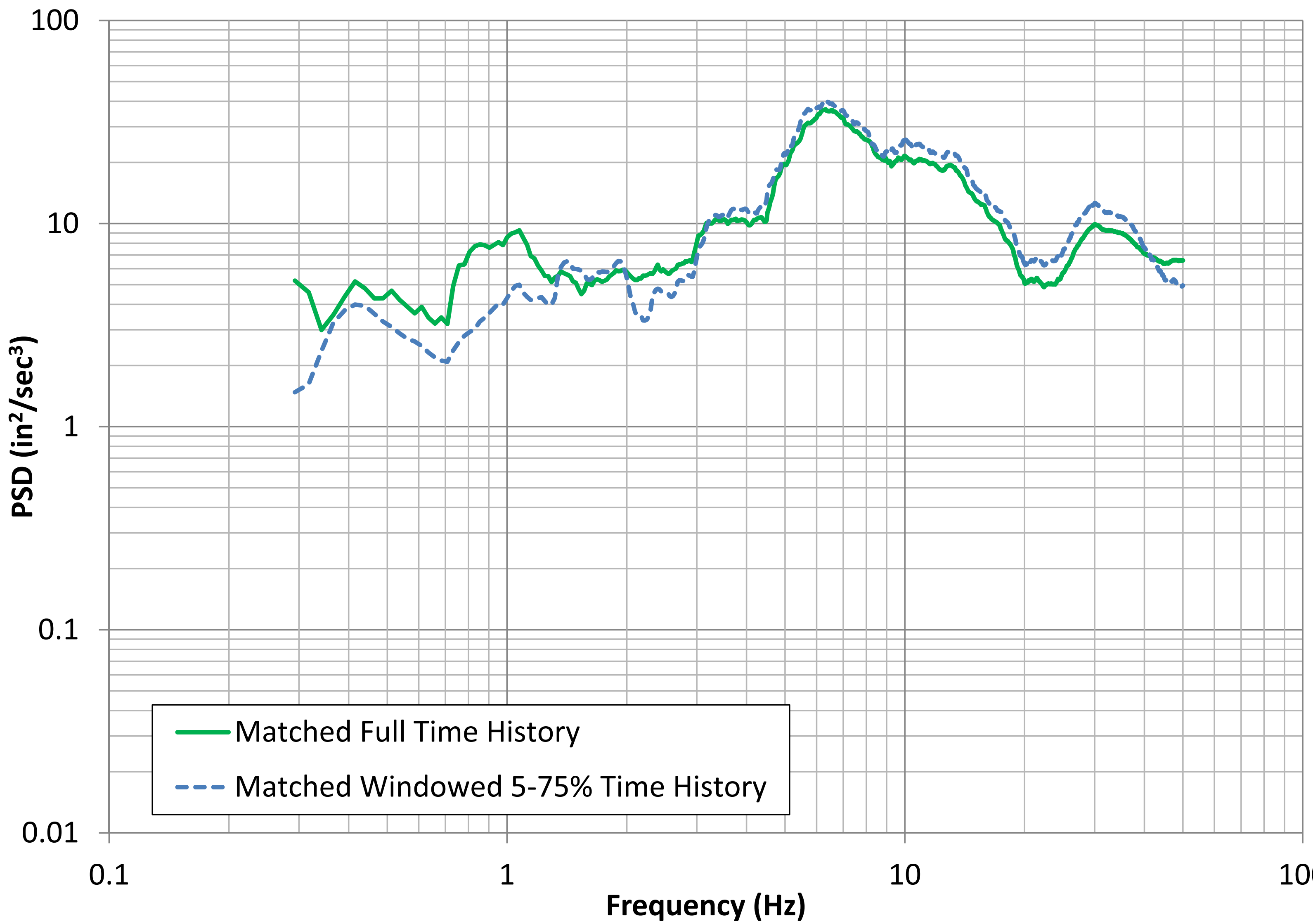
Figure 3.7.1-277 PSD for the H1 Component of the CB Full Profile Spectrum Compatible Acceleration Time History



NAPS SUP 3.7-2 **Figure 3.7.1-278 PSD for the H2 Component of the CB Full Profile Spectrum Compatible Acceleration Time History**

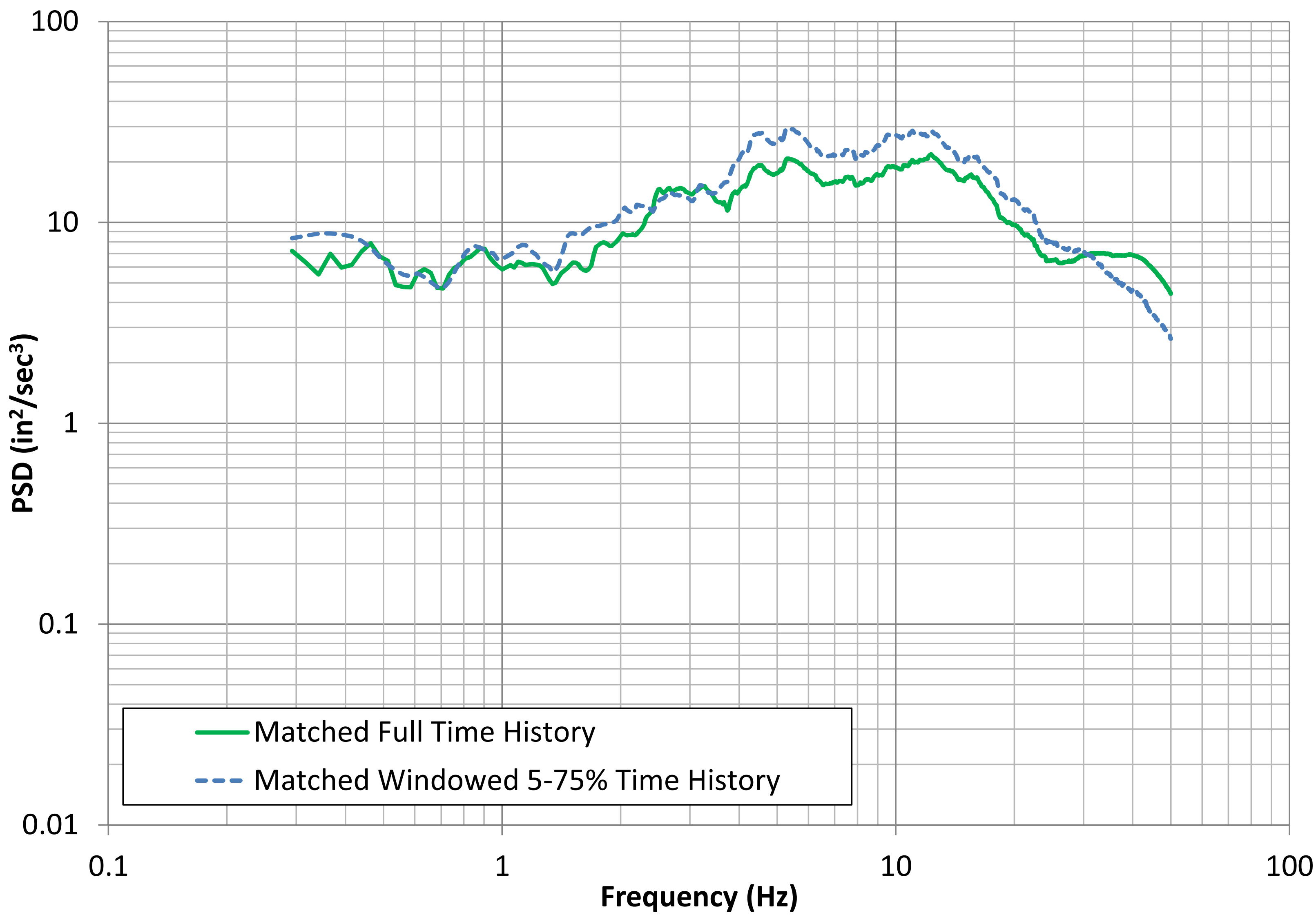


NAPS SUP 3.7-2 **Figure 3.7.1-279 PSD for the UP Component of the CB Full Profile Spectrum Compatible Acceleration Time History**

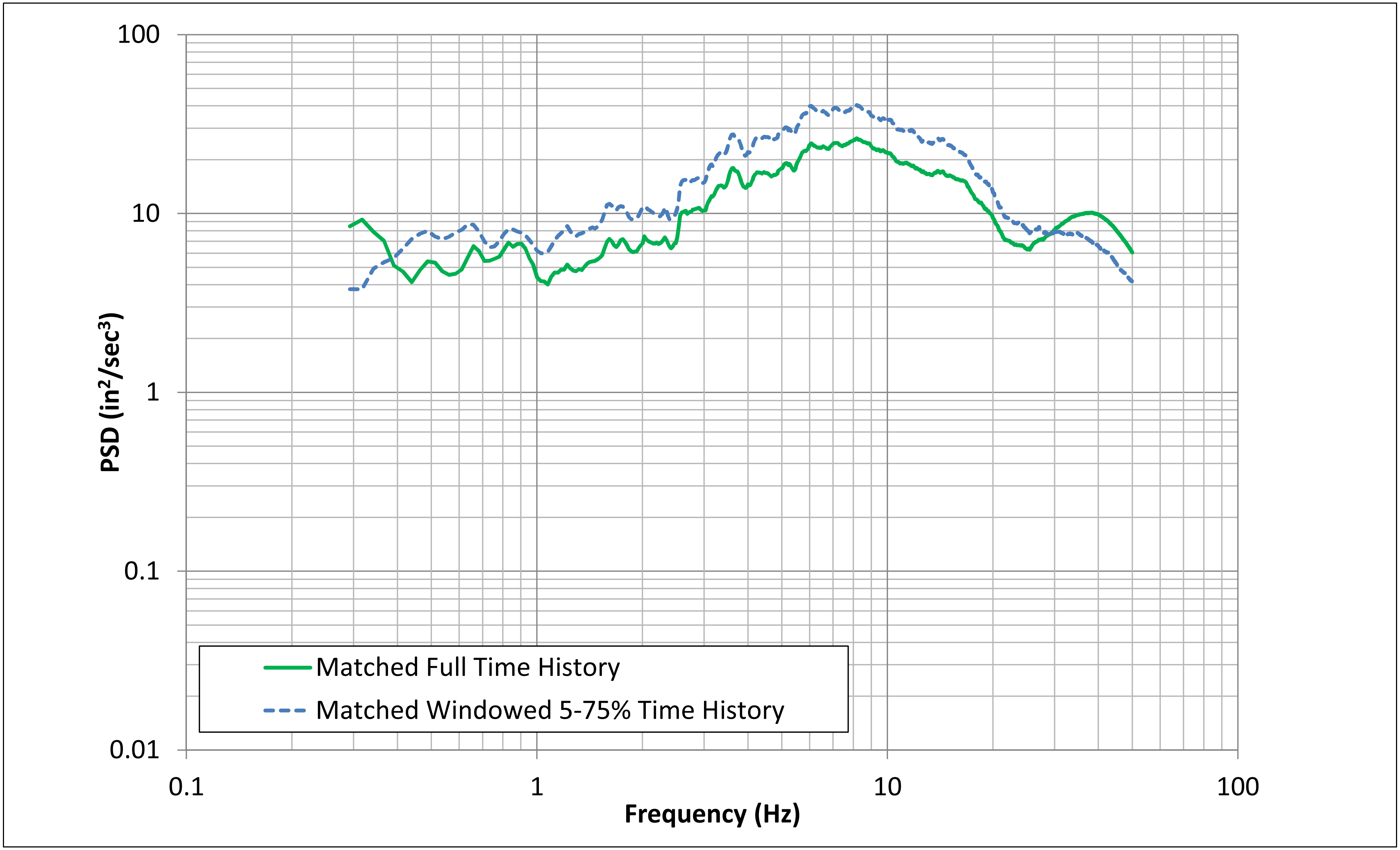


NAPS SUP 3.7-2

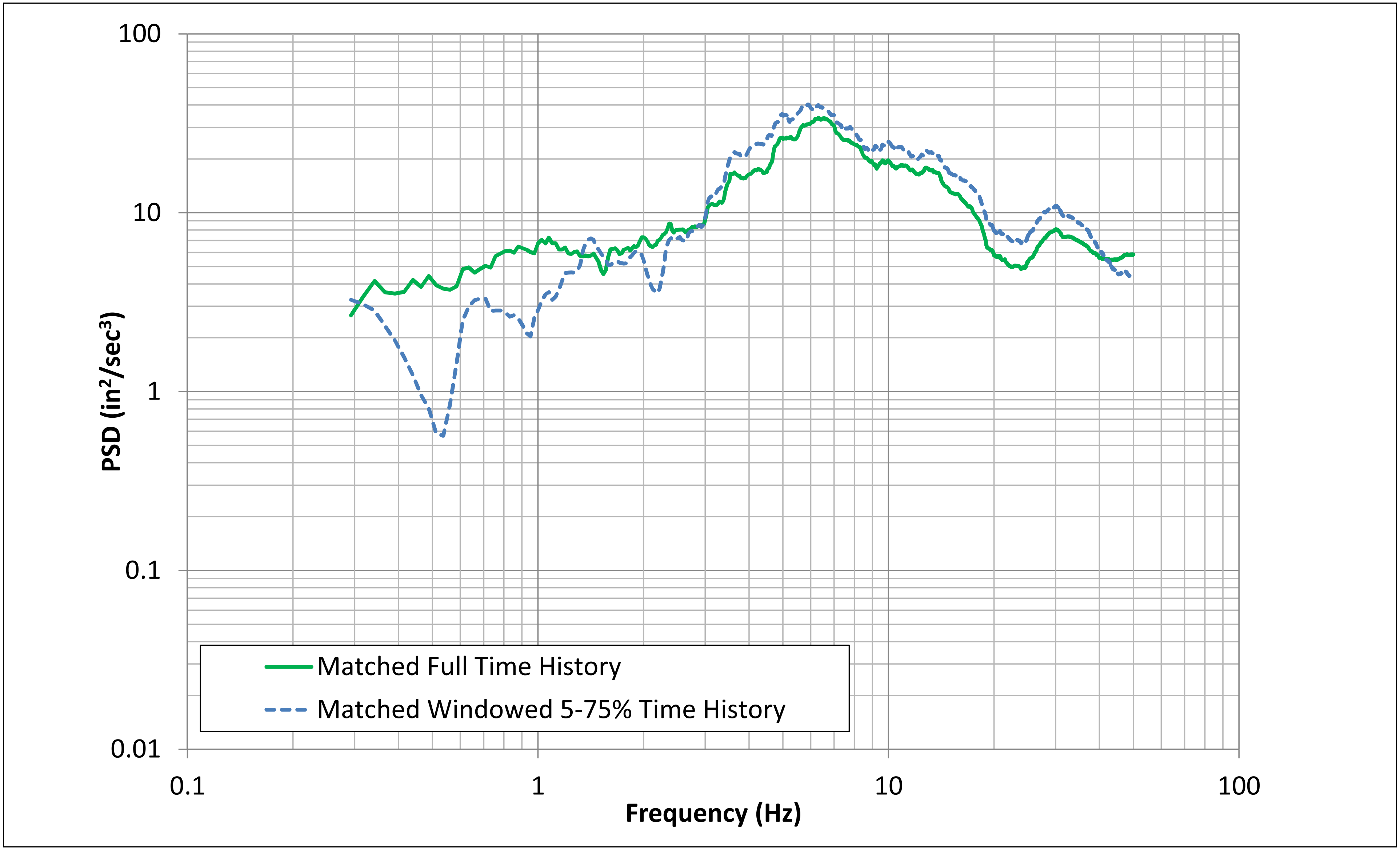
Figure 3.7.1-280 PSD for the H1 Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 282 ft



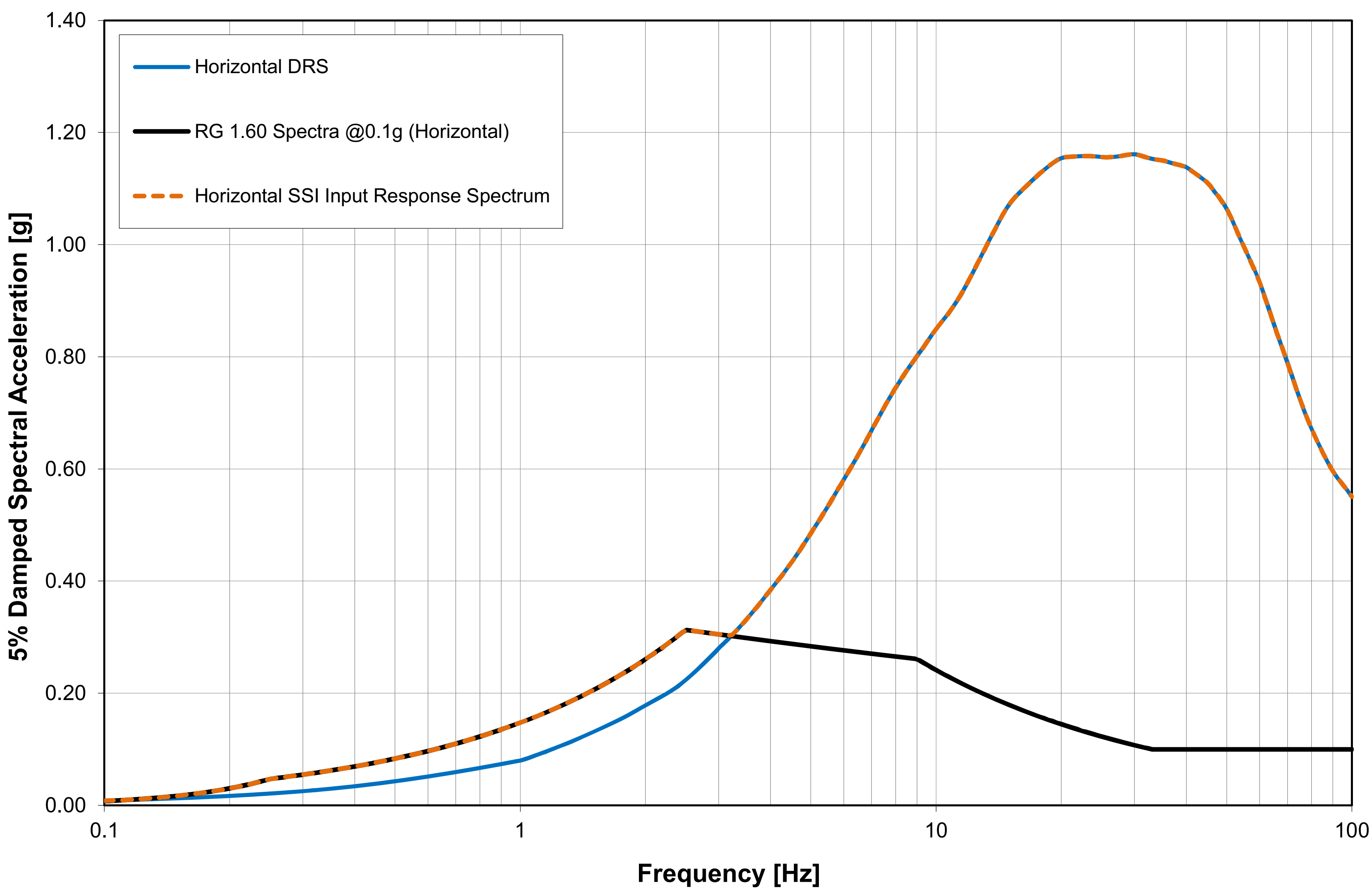
NAPS SUP 3.7-2 **Figure 3.7.1-281 PSD for the H2 Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 282 ft**



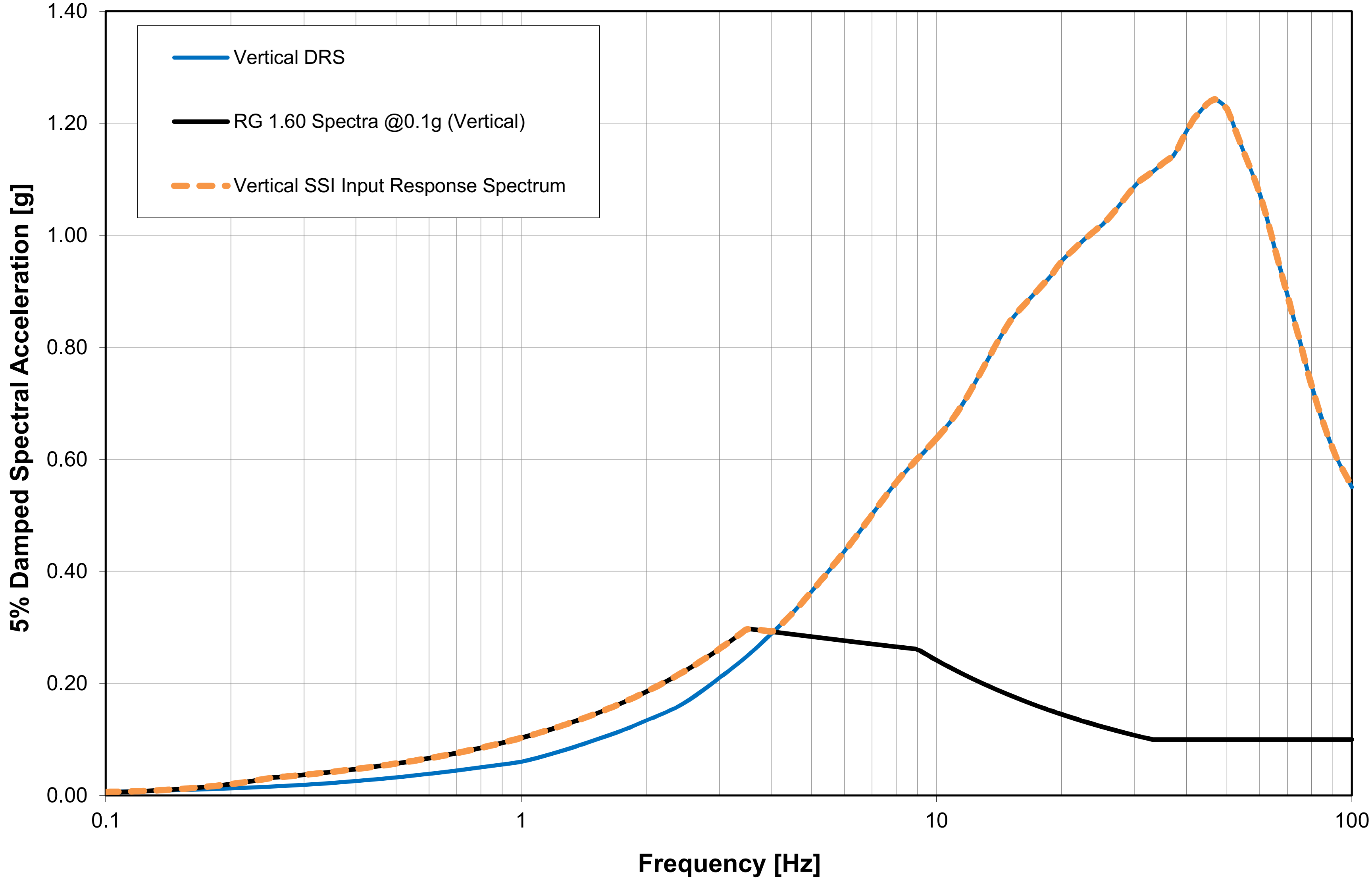
NAPS SUP 3.7-2 **Figure 3.7.1-282 PSD for the UP Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 282 ft**



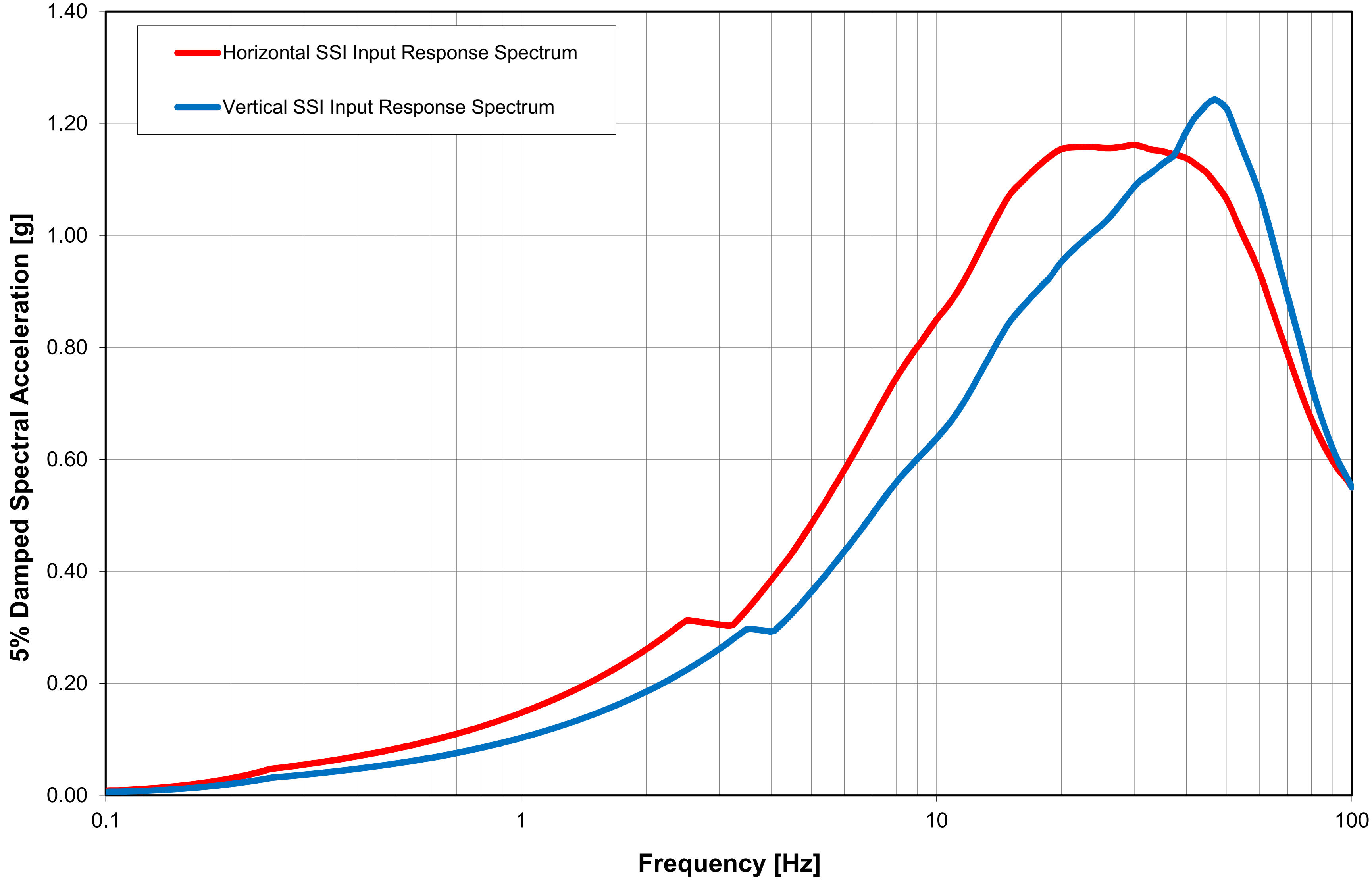
NAPS SUP 3.7-1 **Figure 3.7.1-283 Development of 5% Damped Final Horizontal SSI Input Response Spectrum at Elevation 220 ft for FWSC**



NAPS SUP 3.7-1 **Figure 3.7.1-284 Development of 5% Damped Final Vertical SSI Input Response Spectrum at Elevation 220 ft for FWSC**

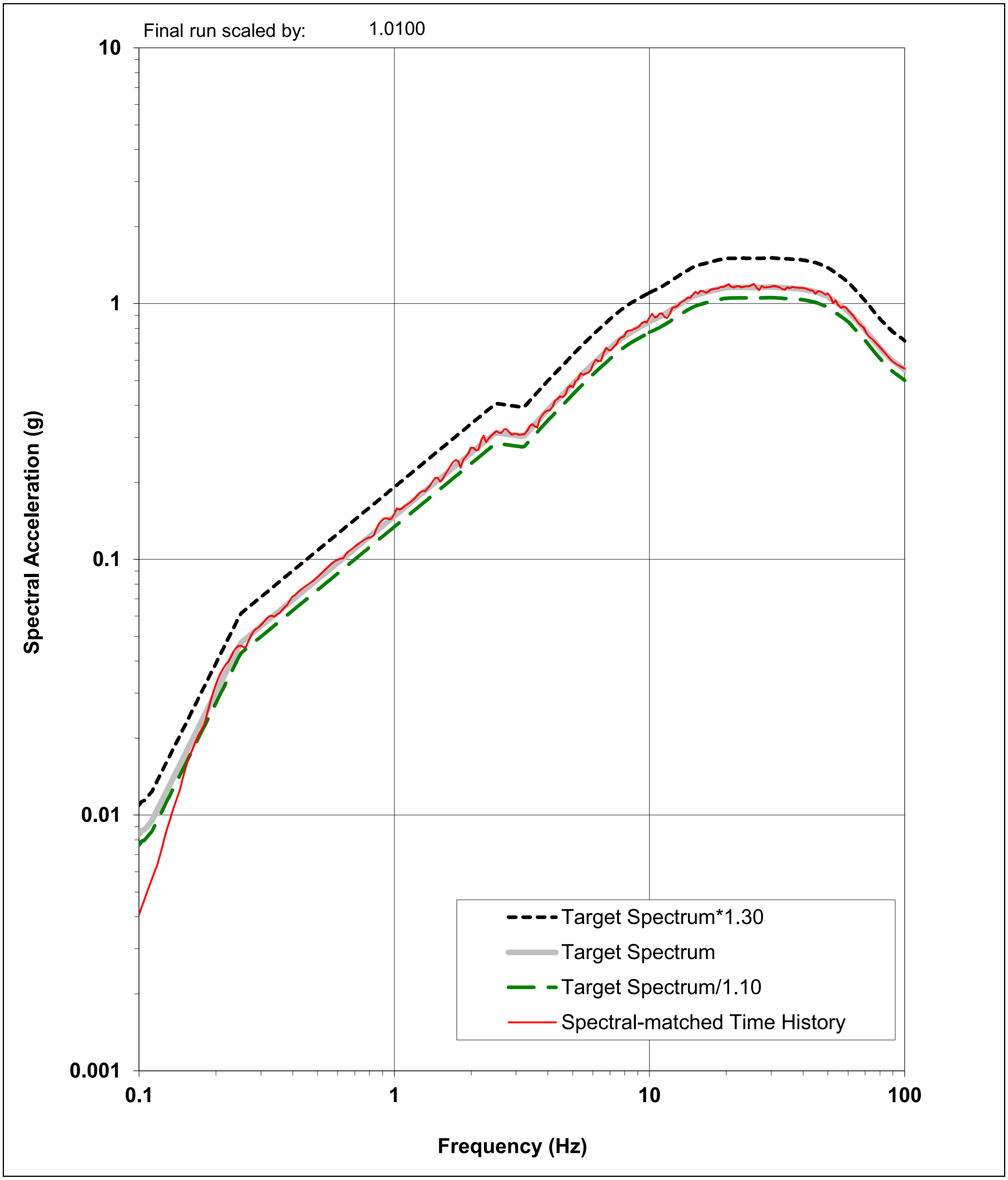


NAPS SUP 3.7-1 **Figure 3.7.1-285 5% Damped Final SSI Input Response Spectra at Elevation 220 ft for FWSC**

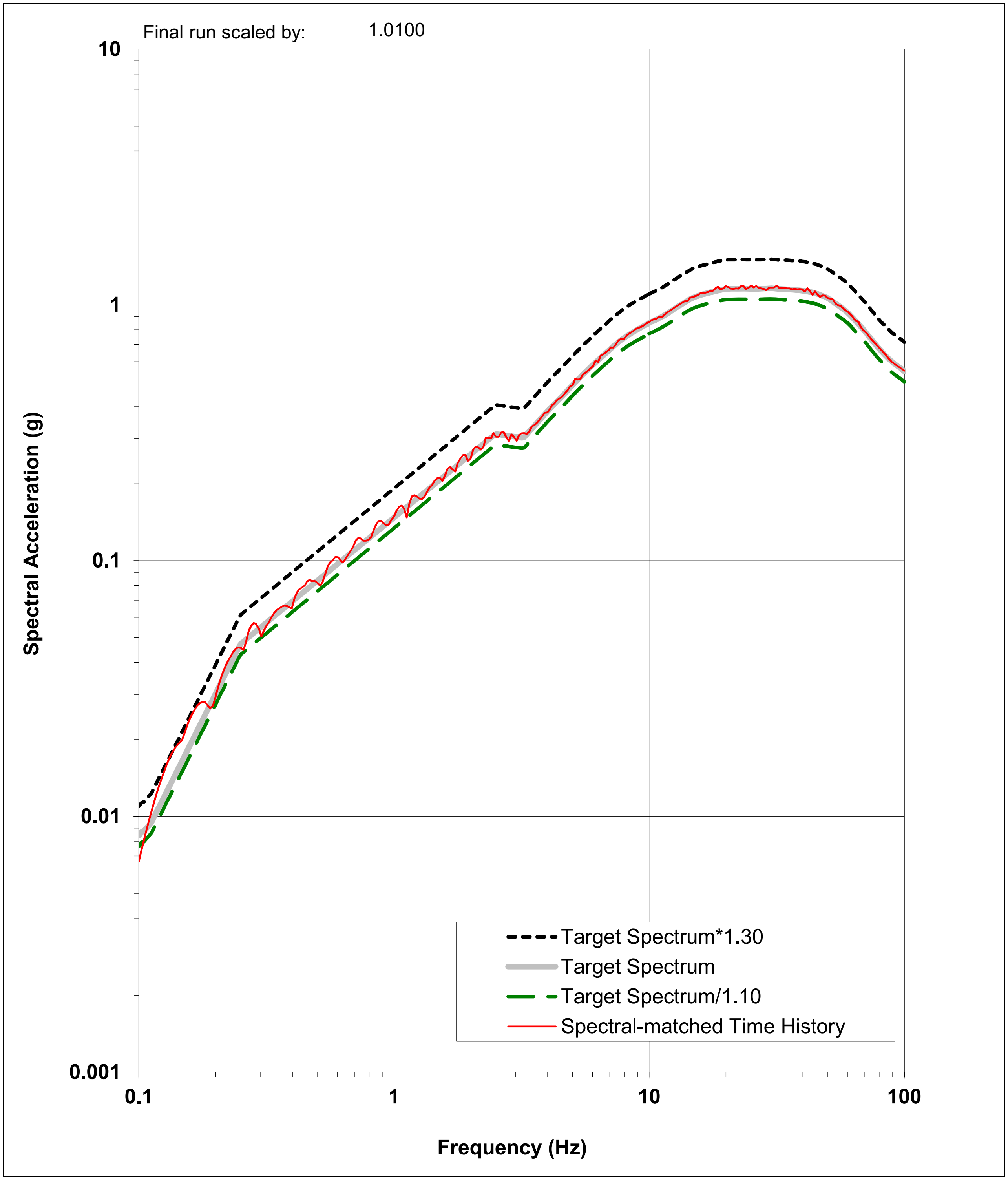


NAPS SUP 3.7-2

Figure 3.7.1-286 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, H1 Component at Elevation 220 ft



NAPS SUP 3.7-2 **Figure 3.7.1-287 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, H2 Component at Elevation 220 ft**



NAPS SUP 3.7-2 **Figure 3.7.1-288 Comparison between the Final Scaled Spectrum Compatible Response Spectrum, the Target Spectrum, and Upper and Lower Target Spectrum Bounds for the FWSC, UP Component at Elevation 220 ft**

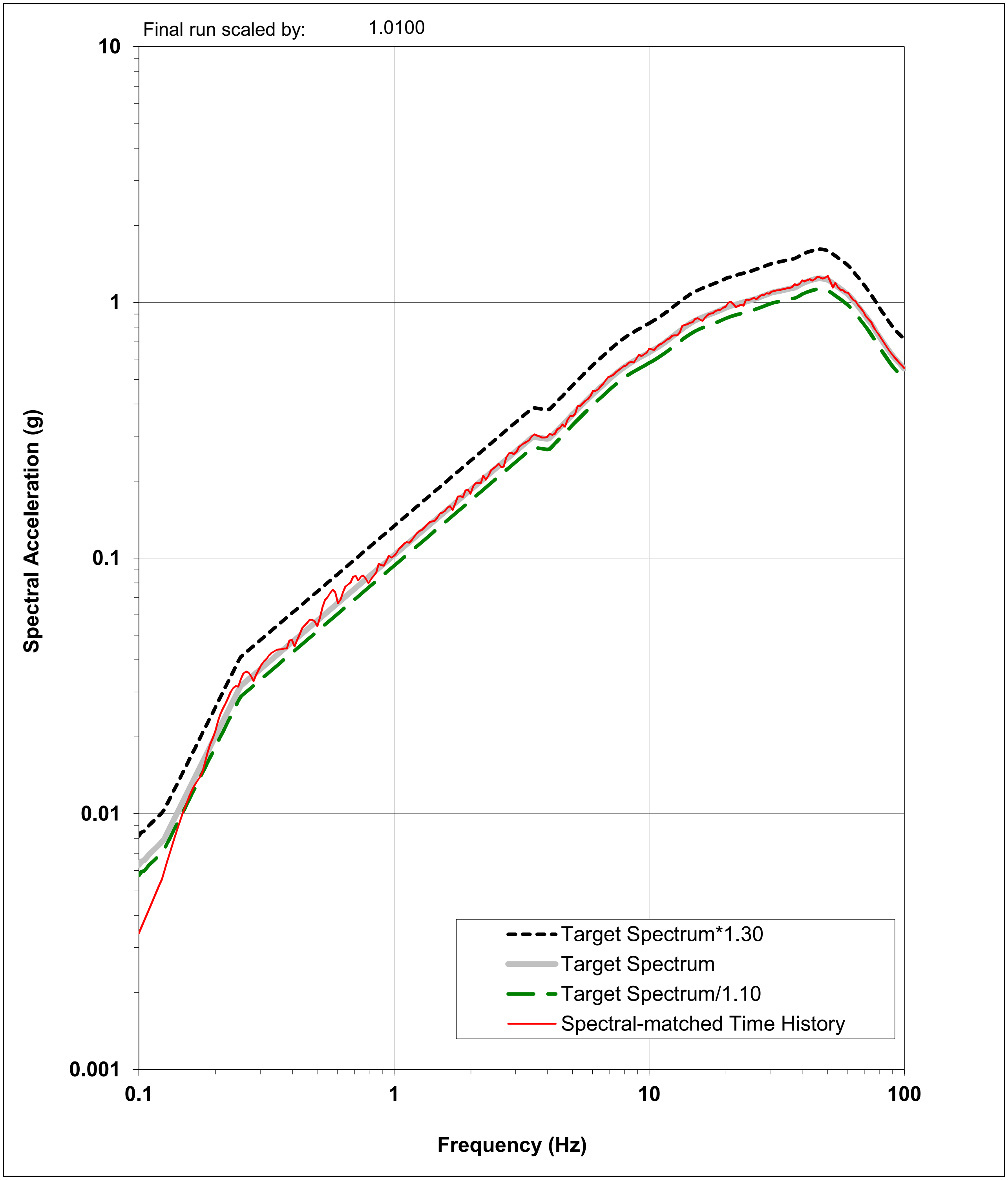
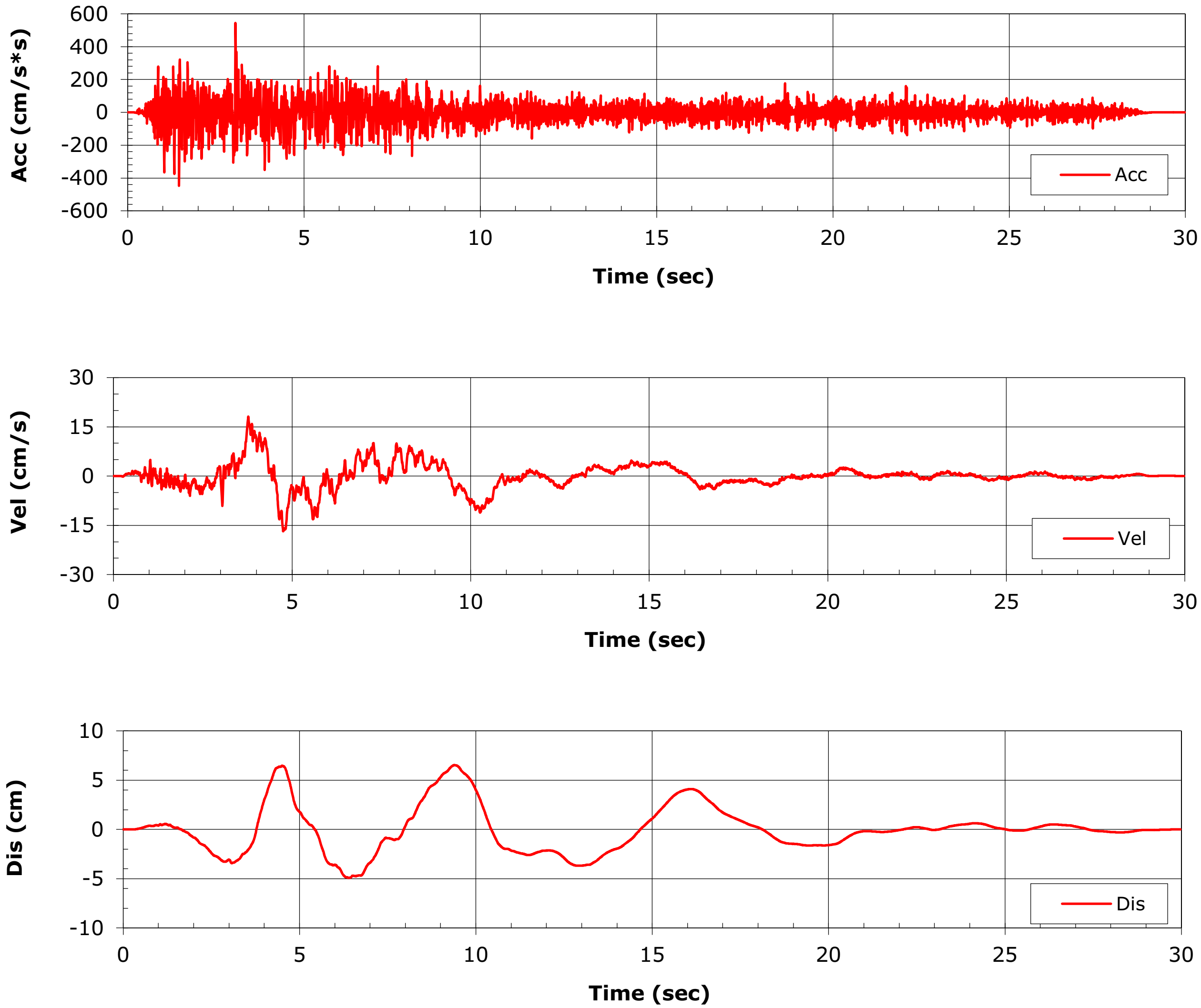
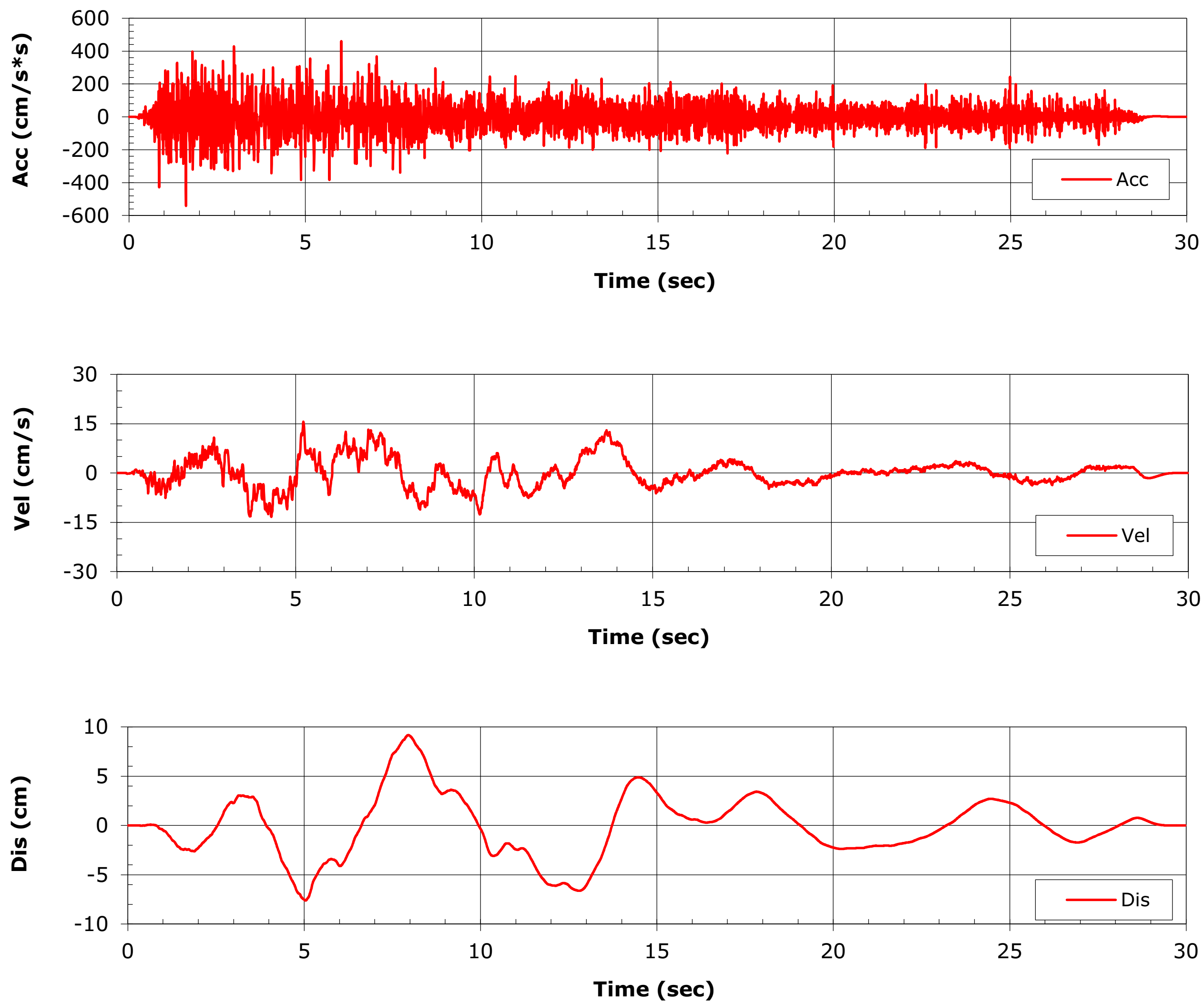


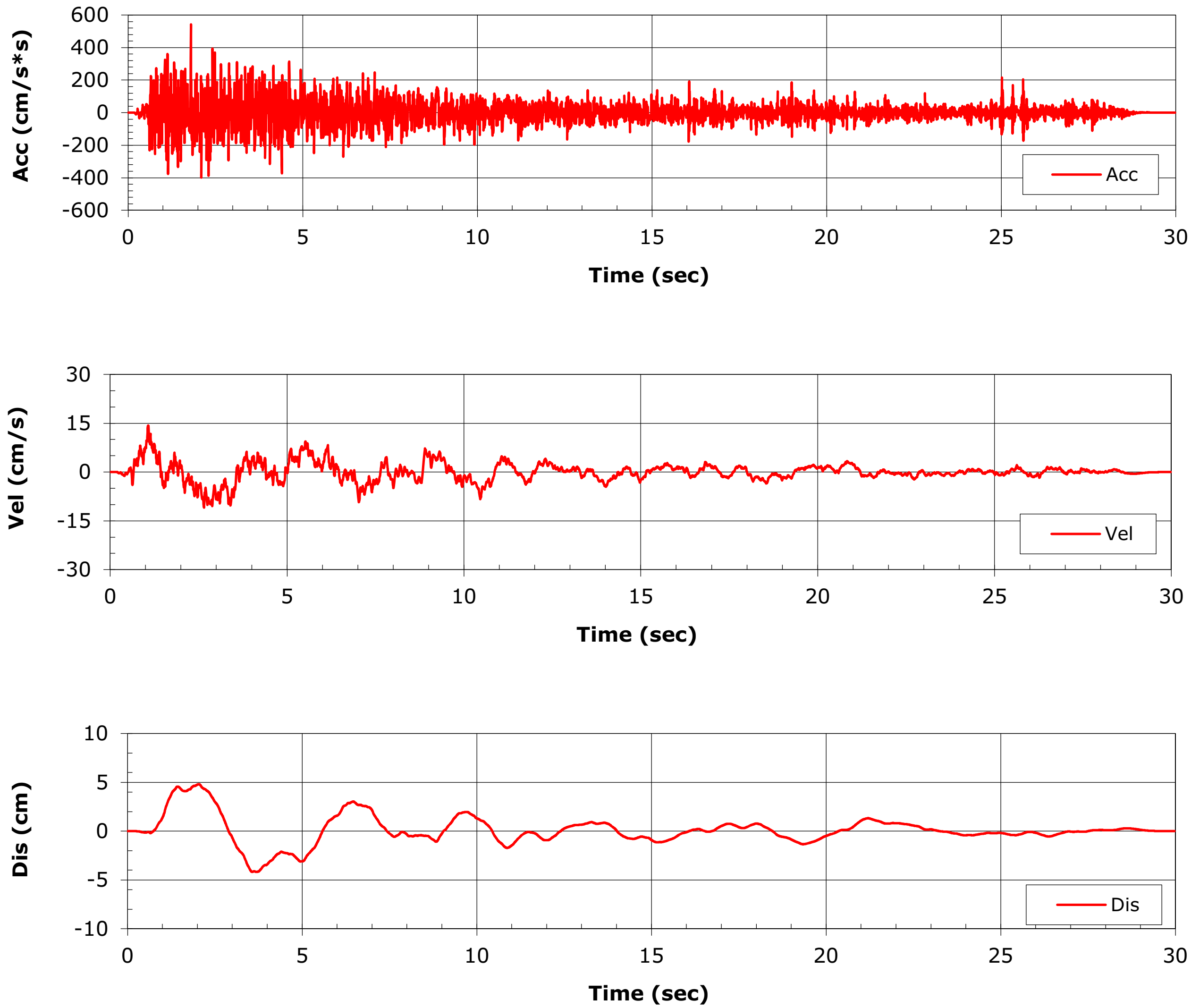
Figure 3.7.1-289 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, H1 Component at Elevation 220 ft



NAPS SUP 3.7-2 **Figure 3.7.1-290 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, H2 Component at Elevation 220 ft**

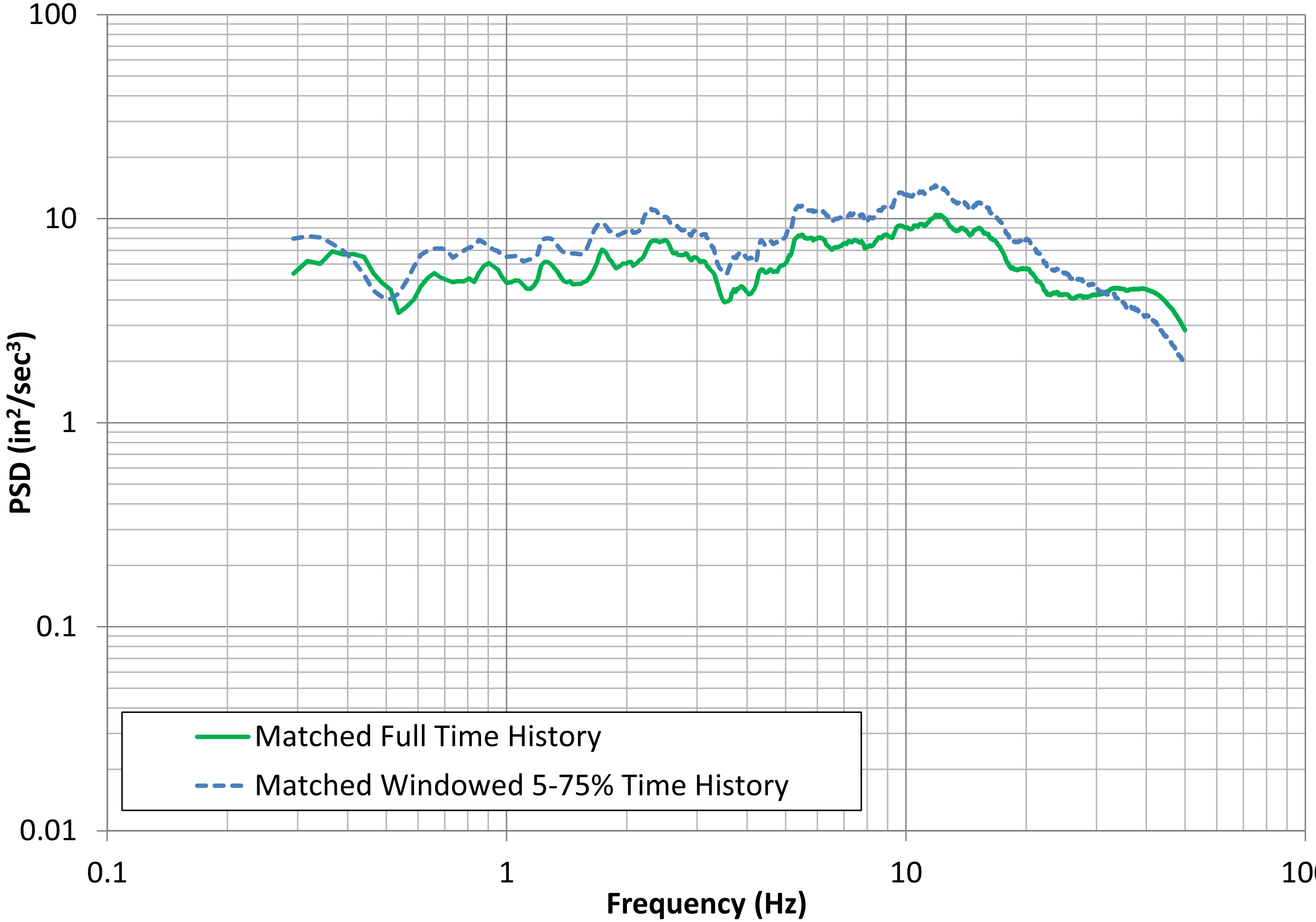


NAPS SUP 3.7-2 **Figure 3.7.1-291 Acceleration, Velocity, and Displacement Spectrally Matched Partial Column Outcrop Time Histories for the FWSC, UP Component at Elevation 220 ft**

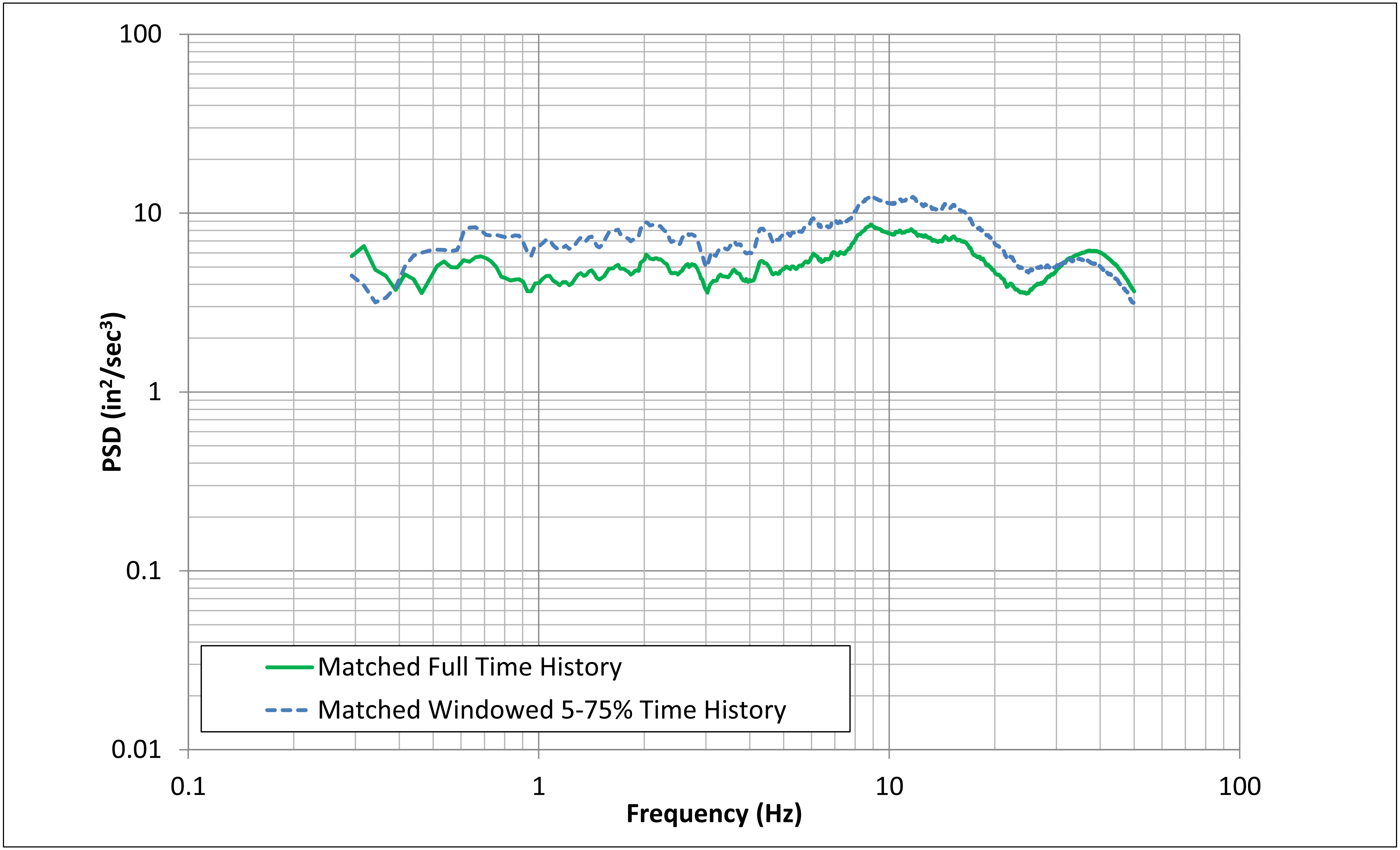


NAPS SUP 3.7-2

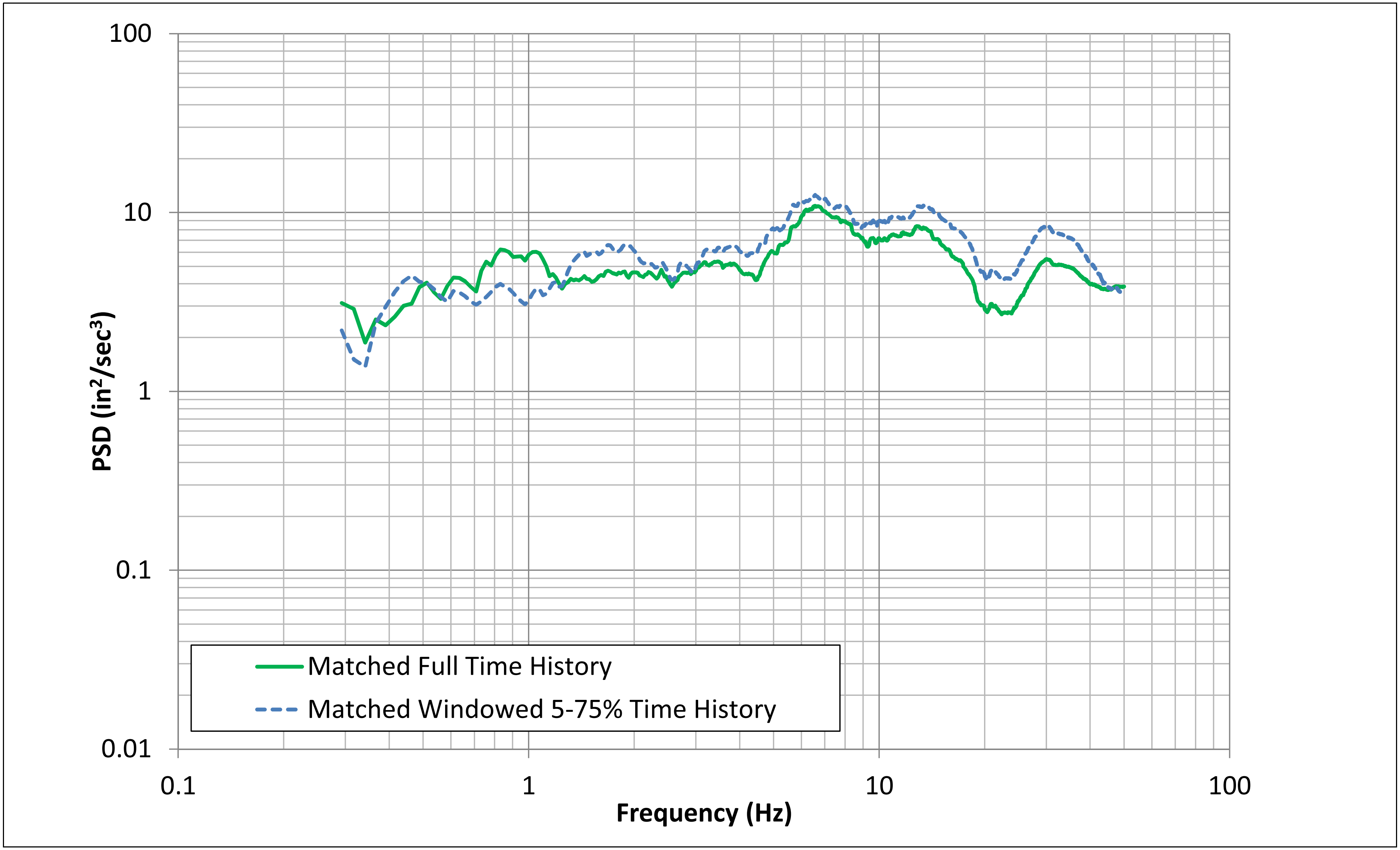
Figure 3.7.1-292 PSD for the H1 Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 220 ft



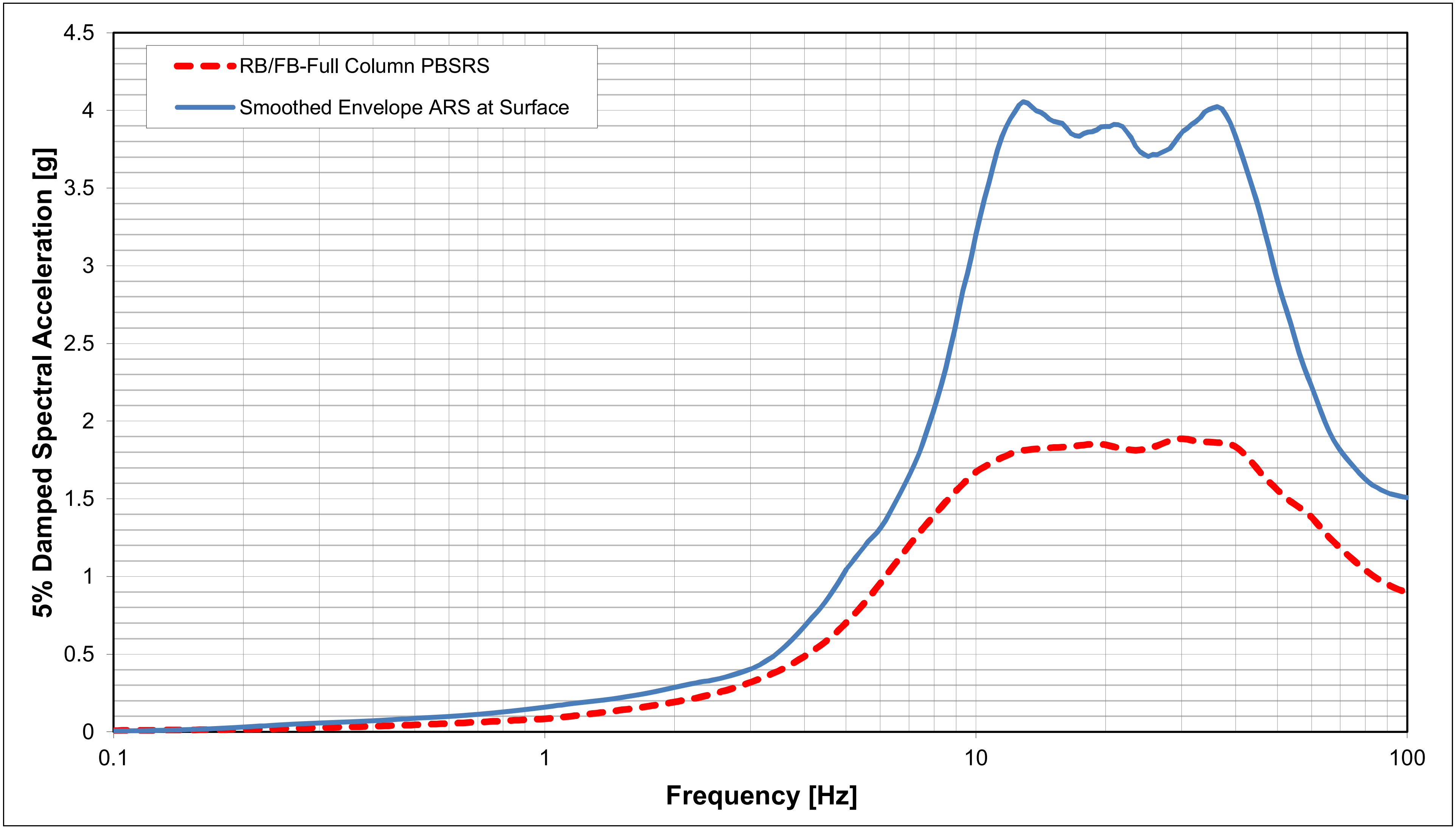
NAPS SUP 3.7-2 **Figure 3.7.1-293 PSD for the H2 Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 220 ft**



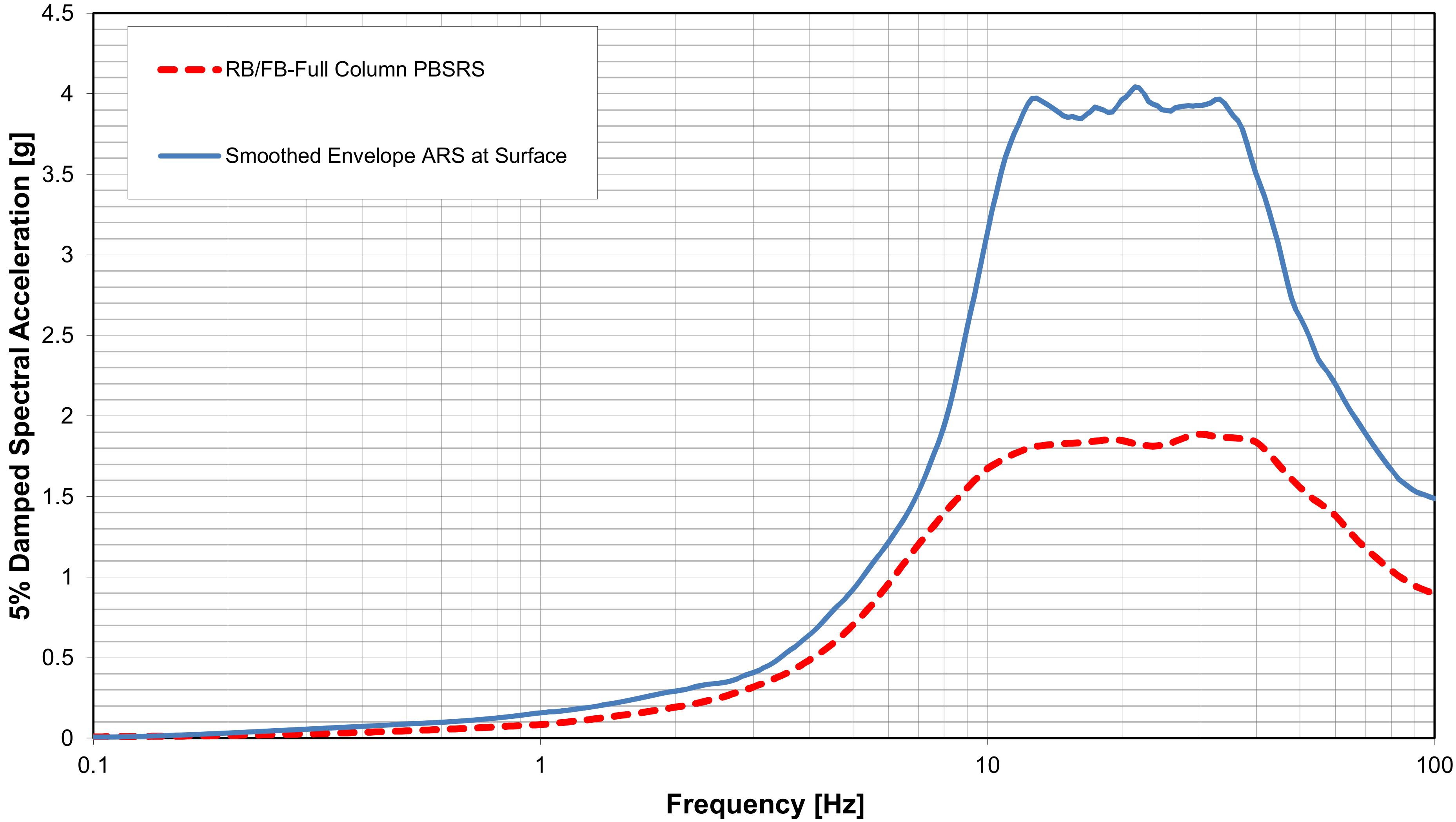
NAPS SUP 3.7-2 **Figure 3.7.1-294 PSD for the UP Component of the FWSC Spectrum Compatible Acceleration Time History at Elevation 220 ft**



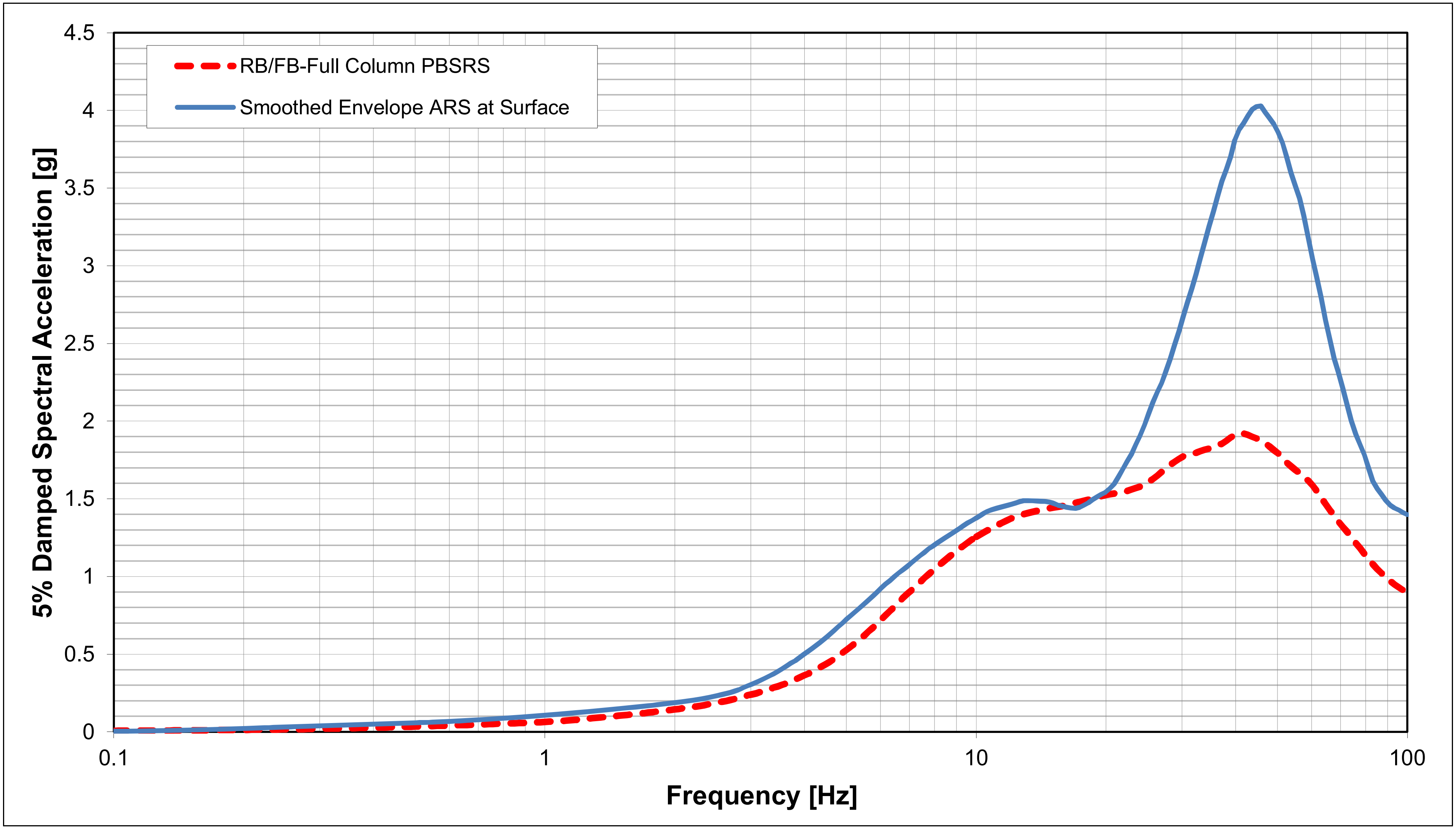
NAPS SUP 3.7-2 Figure 3.7.1-295 Confirmatory NEI Check for Fully Embedded RB/FB, H1



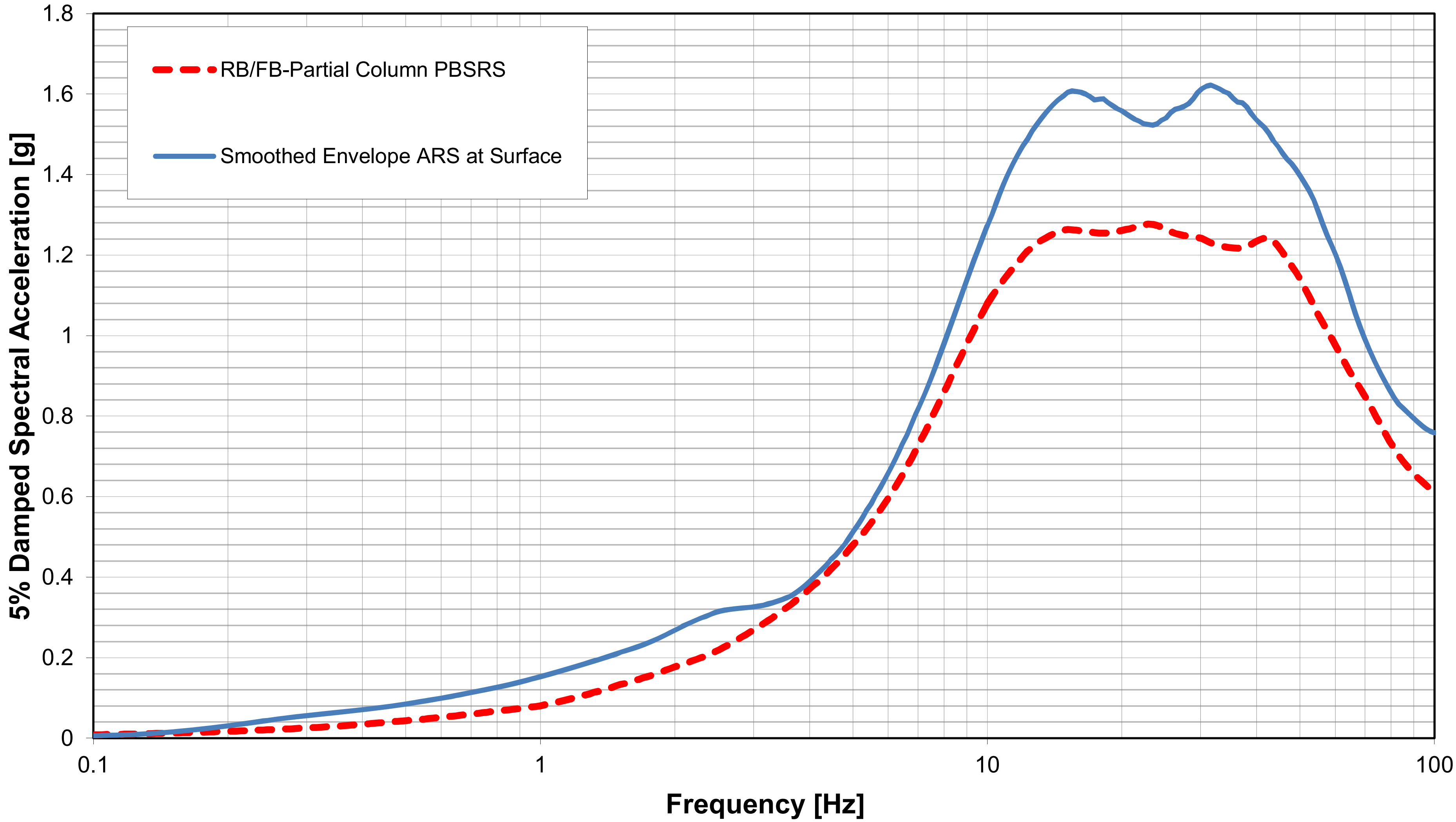
NAPS SUP 3.7-2 Figure 3.7.1-296 Confirmatory NEI Check for Fully Embedded RB/FB, H2



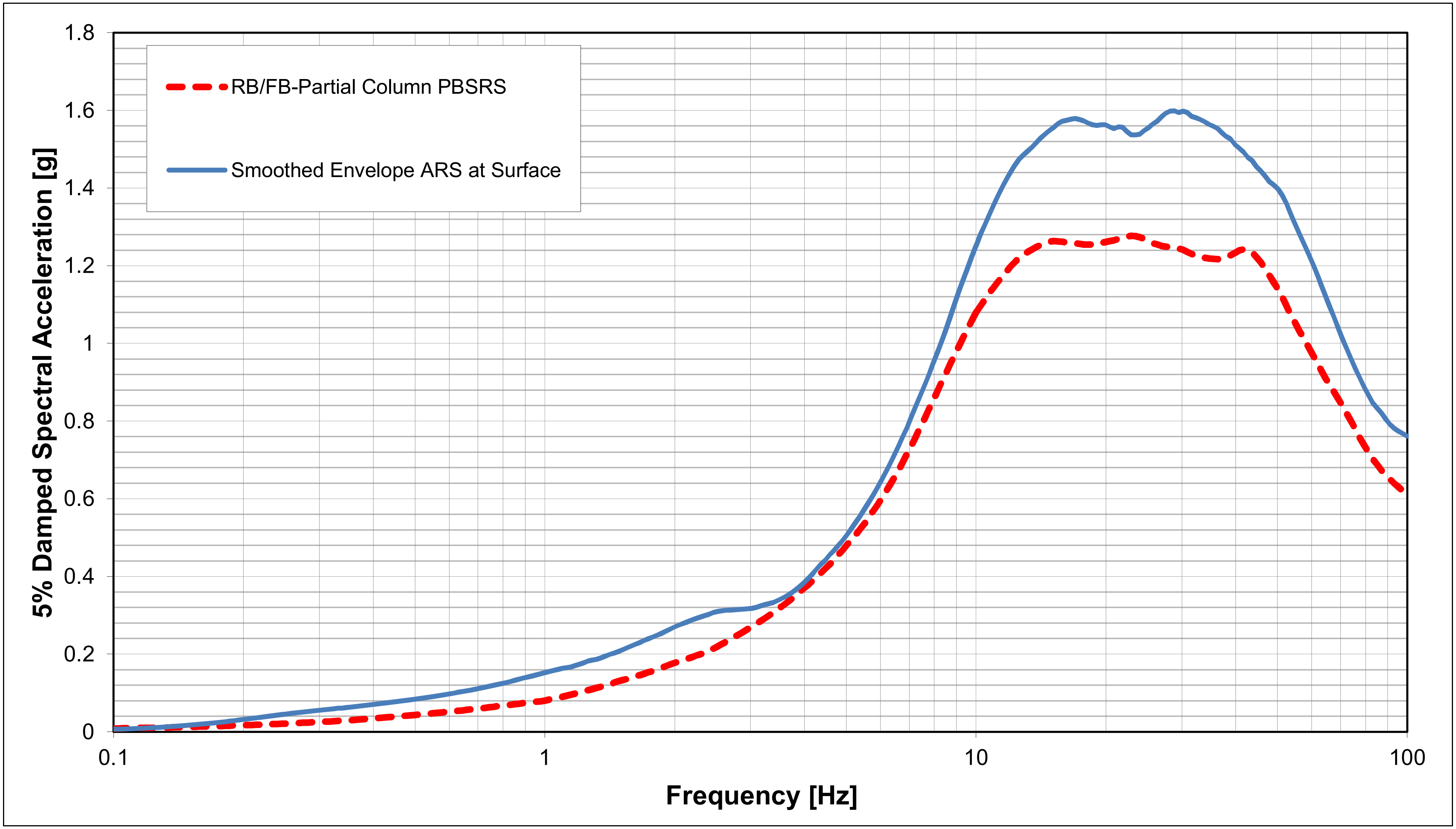
NAPS SUP 3.7-2 Figure 3.7.1-297 **Confirmatory NEI Check for Fully Embedded RB/FB, UP**



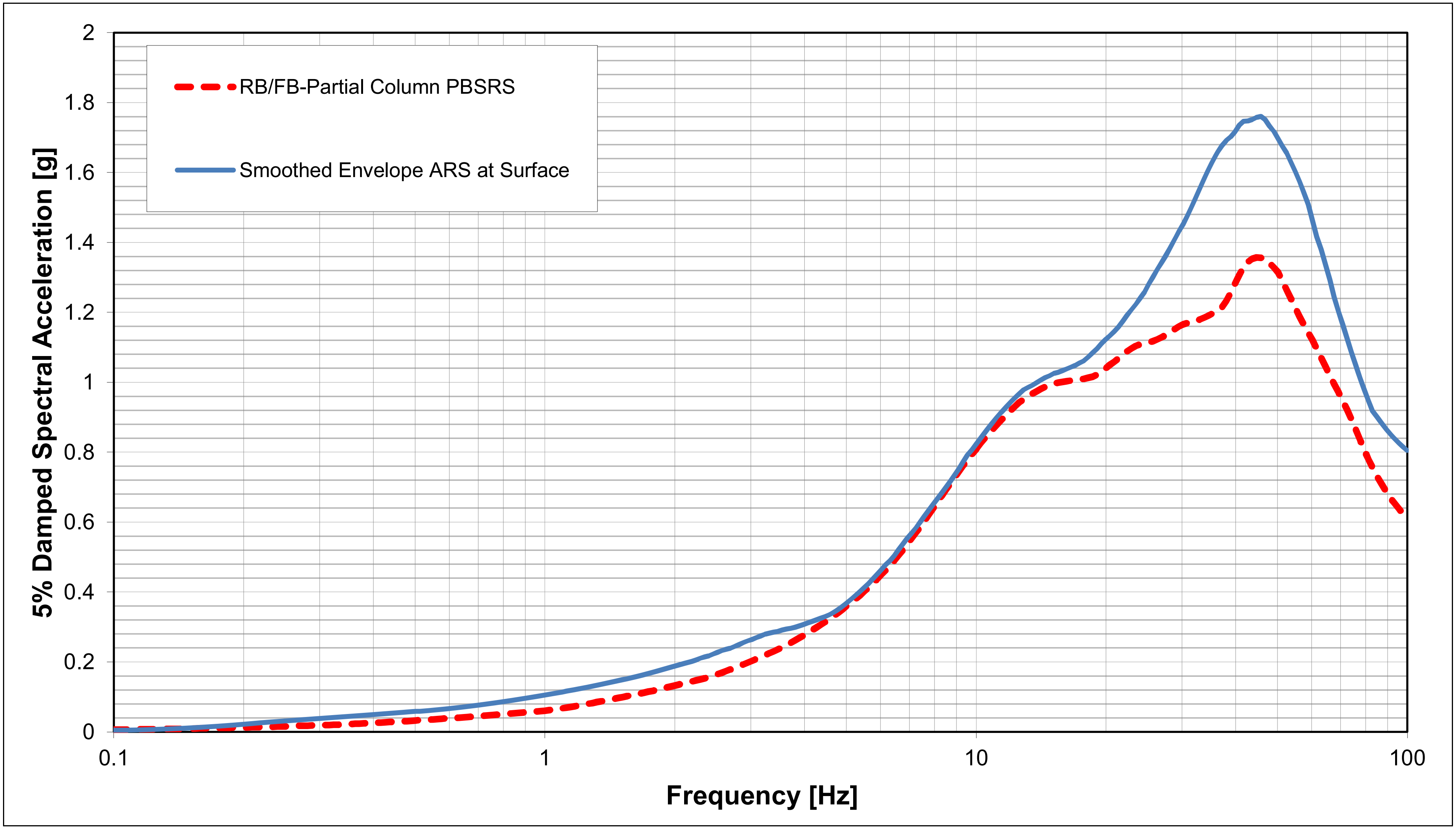
NAPS SUP 3.7-2 Figure 3.7.1-298 **Confirmatory NEI Check for Partially Embedded RB/FB, H1**



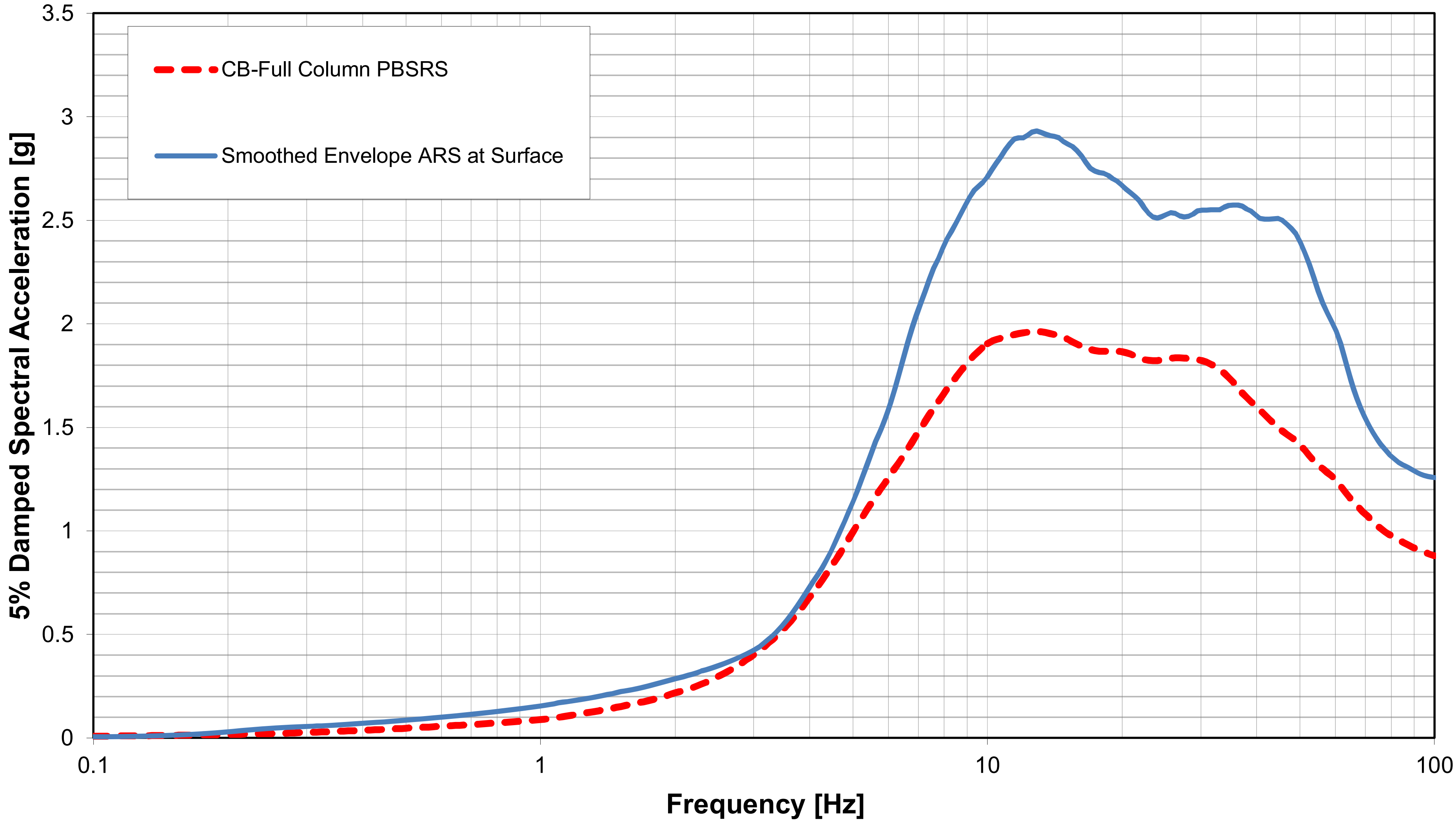
NAPS SUP 3.7-2 Figure 3.7.1-299 **Confirmatory NEI Check for Partially Embedded RB/FB, H2**



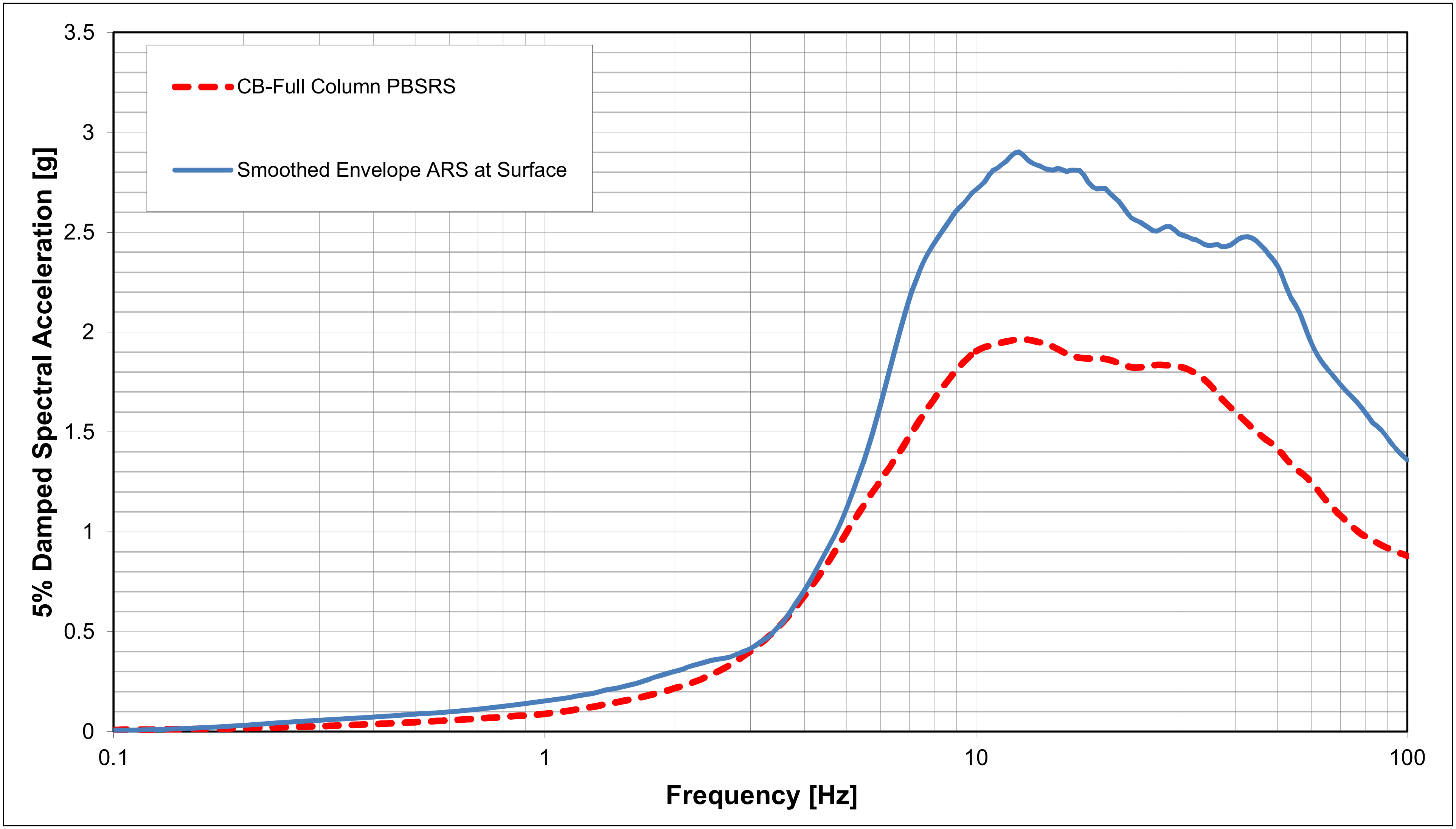
NAPS SUP 3.7-2 Figure 3.7.1-300 **Confirmatory NEI Check for Partially Embedded RB/FB, UP**



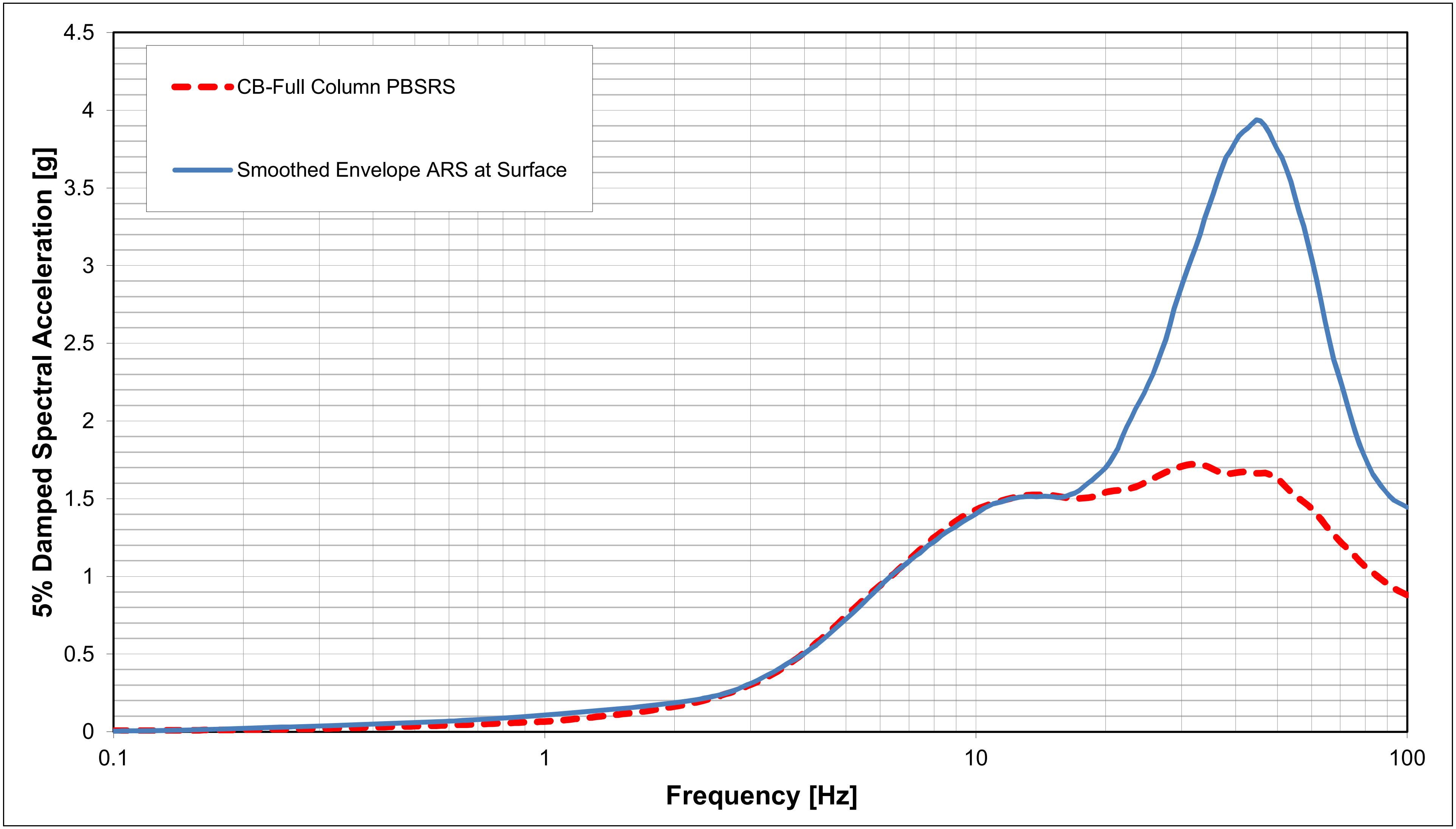
NAPS SUP 3.7-2 Figure 3.7.1-301 **Confirmatory NEI Check for Fully Embedded CB, H1**



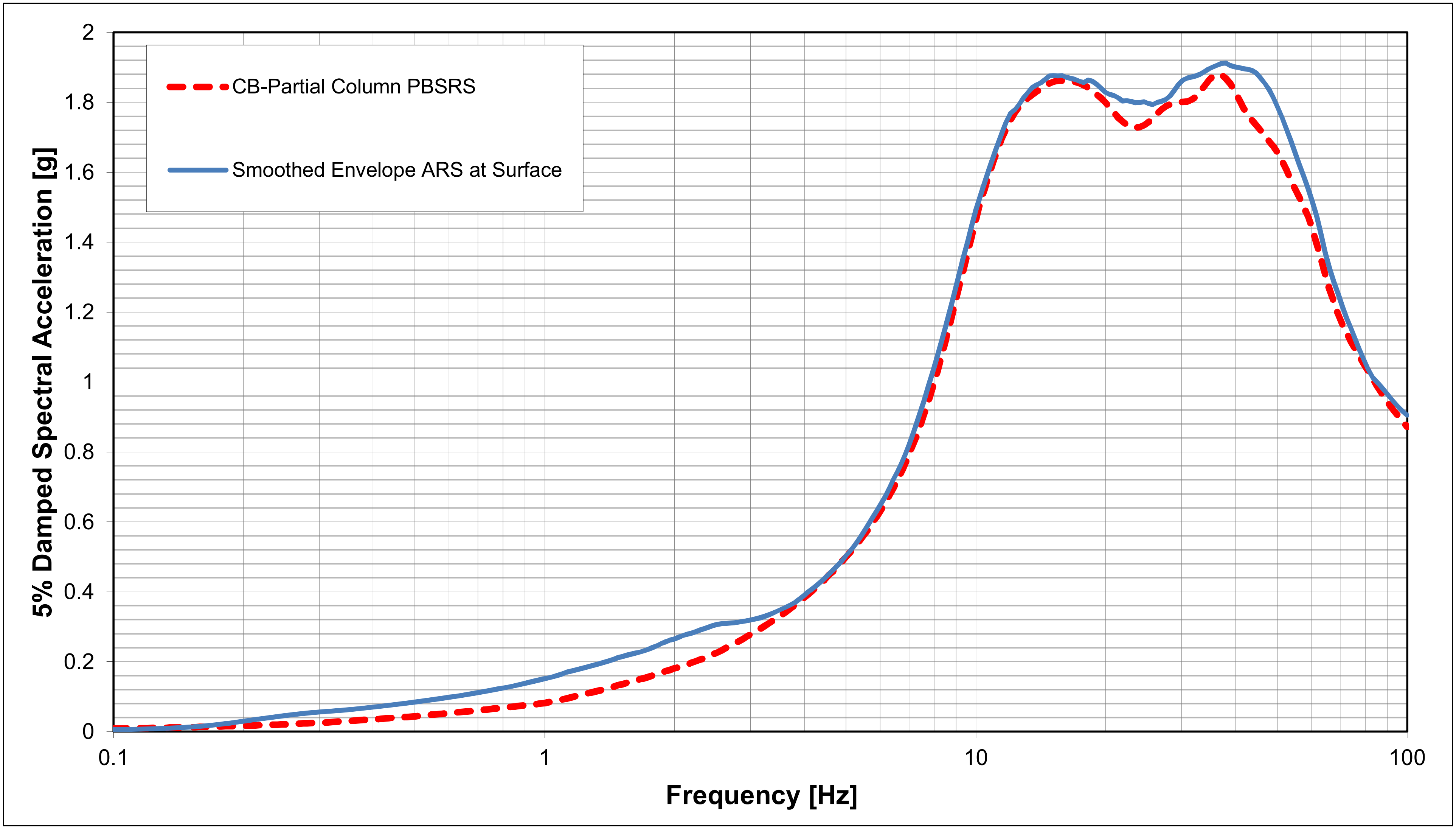
NAPS SUP 3.7-2 Figure 3.7.1-302 **Confirmatory NEI Check for Fully Embedded CB, H2**



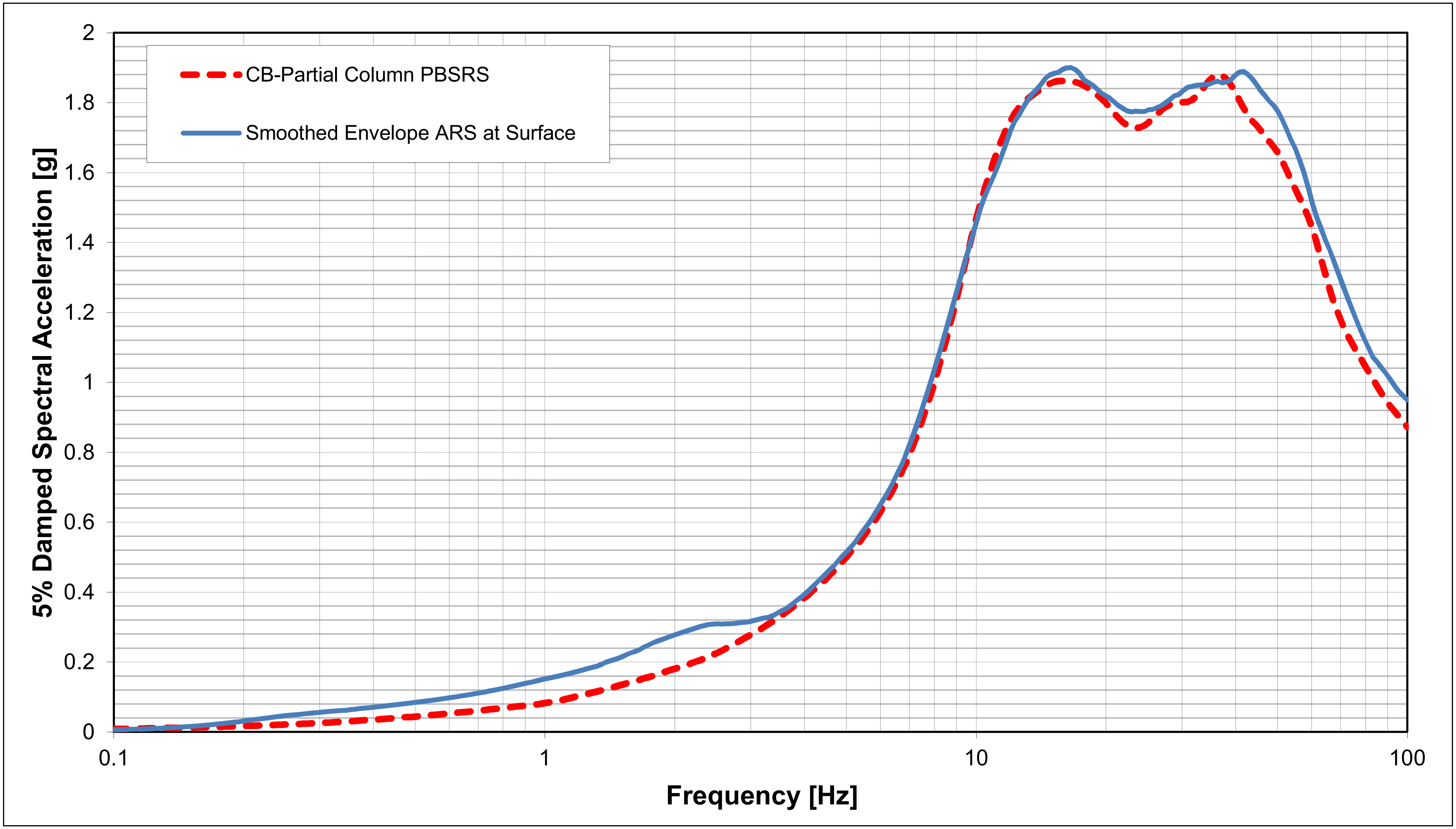
NAPS SUP 3.7-2 Figure 3.7.1-303 **Confirmatory NEI Check for Fully Embedded CB, UP**



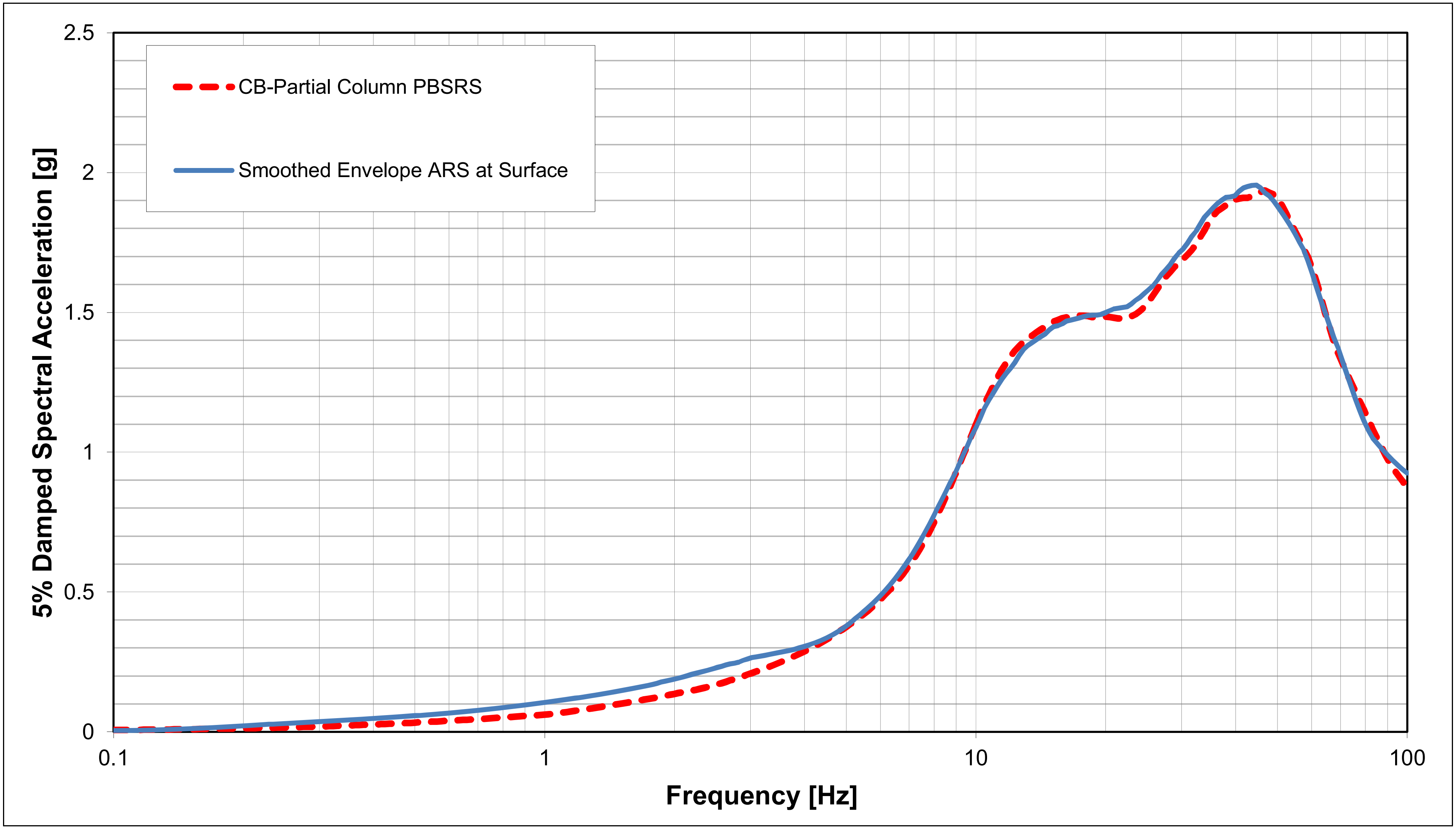
NAPS SUP 3.7-2 Figure 3.7.1-304 **Confirmatory NEI Check for Partially Embedded CB, H1**



NAPS SUP 3.7-2 Figure 3.7.1-305 **Confirmatory NEI Check for Partially Embedded CB, H2**



NAPS SUP 3.7-2 Figure 3.7.1-306 **Confirmatory NEI Check for Partially Embedded CB, UP**



3.7.2 Seismic System Analysis

Replace the last sentence of the first paragraph with the following.

NAPS DEP 3.7-1

[*DCD Table 3.7-3 provides a summary of the standard seismic analysis methods for primary building structures using generic site conditions.*]* ~~Section 3.7.2.4.1 describes~~ Sections 3.7.2.4 and 3A.10 through 3A.19 describe the supplemental Unit 3 site-specific SSI analyses for the Unit 3 site conditions using SASSI2010. SASSI2010 is described in Section 3C.7.4. ACS SASSI, which is used for Unit 3 sensitivity analyses, is described in Section 3C.7.6.

* Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.

3.7.2.2 Natural Frequencies and Responses

Replace the first sentence in this paragraph with the following.

NAPS DEP 3.7-1

[*Natural frequencies and SSI responses of Seismic Category I buildings obtained from the seismic response analyses forming the basis for seismic design of ESBWR Standard Plant are presented in DCD ~~Appendix 3A~~ Sections 3A.1 through 3A.9. The site-specific SSI responses ~~specific~~ for Unit 3 site-specific conditions are provided in ~~Section 3.7.2.4~~ Sections 3.7.2.4 and 3A.10 through 3A.19.*]*

* Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.

3.7.2.4 Soil-Structure Interaction

Add the following at the end of the first paragraph.

NAPS DEP 3.7-1

This section of the DCD, including associated DCD Appendix 3A in its entirety, is incorporated by reference with the following supplemental information for the Unit 3 site-specific soil-structure interaction (SSI) analyses for the RB/FB, CB, and FWSC. DCD Appendix 3A provides the SSI analysis approach and results for the standard design based on the CSDRS and generic soil conditions described in Section 3.7.1.

The site-specific SSI analysis considers SSI effects by following an approach that is consistent with those used for standard design. The structural models used for the site-specific SSI analyses have the same configuration ~~and the~~, stiffness, and mass inertia properties as the

standard design basis structural models presented in DCD Appendix 3A. ~~Only the stiffness properties and the meshing of the building basements are adjusted for Unit 3 site-specific conditions as discussed below.~~ Only the meshing of the below-grade portions of the buildings is adjusted for Unit 3 site-specific conditions.

As described in Section 2.0, the site-specific horizontal and vertical FIRS have been compared to the corresponding CSDRS used for design of ESBWR standard plant. These comparisons show that there are ranges of frequencies where both the horizontal and vertical site-specific FIRS exceed the CSDRS. In accordance with the requirements of DCD Tier 1, Section 5.1, to address these exceedances, site-specific SSI analyses are performed for Unit 3 Seismic Category I RB/FB, CB, and FWSC structures using the Unit 3 site-specific ground motion and strain compatible soil properties.

The results of these analyses serve to demonstrate the applicability of the seismic design of the ESBWR Standard Plant for the Unit 3 site-specific conditions. The responses obtained from the site-specific SSI analyses serve as basis for development of Unit 3 site-specific ISRS for all locations in the Seismic Category I buildings.

~~This section presents~~ Sections 3A.10 through 3A.19 present the approach and methodology used for the site-specific SSI analyses and the reconciliation of the ESBWR standard plant design for the Unit 3 site-specific conditions. Site-specific ISRS are also presented in this section for representative locations.

~~Add the following at the end of Section 3.7.2.4.~~

~~NAPS DEP 3.7-1~~

~~3.7.2.4.1 **Site-Specific Soil-Structure Interaction Analysis**~~

~~This section presents the site-specific SSI analyses of the Seismic Category I RB/FB, CB, and FWSC. The site-specific SSI analyses are performed to address the Unit 3 site-specific conditions.~~

~~The methodology used for site-specific SSI analyses is consistent with the methodology used for the SSI analyses for the ESBWR Standard Plant design. The site-specific SSI analysis is performed using the SASSI2010 computer program that is an updated version of the SASSI2000 computer program used for the standard plant SSI analysis. The structural dynamic models used for the site-specific SSI analyses are developed based on the standard design basis models described in~~

~~DCD Appendix 3A and are coupled with the Unit 3 site-specific strain compatible dynamic subsurface properties developed in Section 3.7.1.~~

~~The site-specific SSI analyses consider the RB/FB embedded into the Zone III rock and include the surrounding concrete fill. The effect of the structural fill above the top of the Zone III rock at DCD Elevation -0.68 m (273 ft NAVD88) on the seismic response of RB/FB is neglected as of minor importance (the DCD Elevation value is relative to the standard plant finished ground level grade EL 4.5 m per DCD Table 3.4-1, while the value following in parentheses is relative to the NA3 site design plant grade EL 290 ft NAVD88). Concrete fill used to fill the gap between the RB/FB and adjacent buildings and excavated in-situ rock up to the top of Zone III rock is included in the SASSI structural model. The input control motion is applied to the SSI model at the bottom of the RB/FB foundation.~~

~~The site-specific SSI analyses of the CB consider the building embedded into the Zone III rock and include the surrounding concrete fill in the SASSI structural model. The CB is founded on concrete fill layer resting on the surface of the Zone III/IV rock. The effect of the structural fill above the top of the Zone III rock at DCD Elevation -3.12 m (265 ft NAVD88) on the seismic response of CB is neglected as of minor importance. Concrete fill used to fill the gap between the CB and adjacent buildings and excavated in-situ rock up to the top of Zone III rock and the concrete fill layer under the CB that is resting on the surface of the Zone III/IV rock are all included in the SASSI structural model. The input control motion is applied to the SSI model at the bottom of the CB foundation resting on the concrete fill layer.~~

~~The site-specific SSI analyses consider the FWSC as a surface founded structure at DCD Elevation 2.15 m (282 ft NAVD88) where the input control motion is applied. The SASSI structural model also includes the concrete fill layer placed on the surface of the Zone III/IV rock up to the bottom of the FWSC basemat. The concrete fill has the same width of the basemat and is embedded in in-situ soil.~~

~~The site-specific SSI analyses results are presented and compared with the seismic responses obtained from the standard design SSI analysis presented in DCD Appendix 3A. These comparisons serve as basis for validation of the applicability of the ESBWR Standard Plant for the Unit 3 site-specific conditions as shown in Section 3.7.2.4.1.6. The responses obtained from the site-specific SSI analyses serve as basis for development of Unit 3 site-specific ISRS. In addition, the below grade~~

exterior walls of the RB/FB and the CB are evaluated for seismic lateral pressure demands in Section 3.8.4. The foundation stability and the dynamic bearing pressure demands are evaluated in Section 3.8.5 for the RB/FB, CB and FWSC based on the site-specific SSI analyses results.

3.7.2.4.1.1 **Strain-Compatible Dynamic Subsurface Material Properties**

The geology of the Unit 3 site is discussed in detail in Section 2.5.1. The subsurface materials encountered at the Unit 3 and the engineering properties of these subsurface materials site are discussed in detail in Section 2.5.4.

Three subsurface material profiles, a best estimate (BE) profile, a lower bound (LB) profile, and an upper bound (UB) profile, are used in the SSI analyses to account for variability in the subsurface materials properties at the Unit 3 site. The development of the site-specific strain compatible dynamic subsurface material properties associated with the BE, LB, and UB profiles is discussed in Section 3.7.1.1.4 and is in accordance with requirements of ISG-017. The strain compatible dynamic properties used for the BE, LB, and UB subsurface profiles used in the site-specific SSI analyses are provided in Table 3.7.1-201 for RB/FB, Table 3.7.1-203 for CB, and Table 3.7.1-205 for FWSC. The concrete fill is included in the SSI analyses structural model as 3-D solid elements as shown in Figures 3.7.2-202 and 3.7.2-203 for RB/FB, in Figures 3.7.2-205 and 3.7.2-206 for CB, and in Figures 3.7.2-208 and 3.7.2-209 for FWSC.

3.7.2.4.1.2 **SSI Input Response Spectra Compatible Ground Motion Time Histories**

Section 3.7.1.1.5 describes development of the site-specific ground motion time histories used as input control motion in the site-specific SSI analyses. Each site-specific SSI analysis is performed using a single set of three ground motion time histories for the three orthogonal components (two horizontal and one vertical) that are applied to the SSI model as free field ground motion at the corresponding foundation bottom elevations. In accordance with ISG-017, the NEI method serves as basis for the development of in-column motion acceleration time histories used as input control motion for the site-specific SSI analyses of RB/FB and CB. The in-column motion time histories at the basemat bottom elevation are developed for each subsurface profile. The site-specific SSI analysis of the FWSC surface mounted model uses a

single set of time histories for the three components of the out-crop motion at plant grade. The duration of time history is 29.98 seconds and the time step is 0.005 seconds.

3.7.2.4.1.3 **Soil-Structure Interaction Analysis Method**

The SSI analysis is performed using the SASSI2010 computer program, an updated version of SASSI2000 computer program used for the standard design SSI analysis described in DCD Appendix 3A. The explicit direct (excavated volume) method is used for the computation of the foundation impedance of the embedded RB/FB and CB models. The same method is also used for the surface founded FWSC to calculate the impedances of the concrete fill block underneath the foundation that is embedded in in-situ soil. The effects of the ground motion incoherency that reduce the responses at higher frequencies are conservatively neglected by considering coherent input ground motion. As shown in Table 3.7.2-201, the cut-off frequency of 50 Hz is used for all site-specific SSI analyses except for the SSI analysis of FWSC with LB profile that uses cut-off frequency of 29 Hz. This LB cutoff frequency is sufficient to capture a low and intermediate frequency range of significance that is typically associated with softer subsurface profiles. The number of FFT points is 8192.

3.7.2.4.1.4 **SSI Analysis Structural Models**

The site-specific SSI structural models for the RB/FB, CB and FWSC are constructed from the building stick models coupled with the foundation finite element model following the methodology described in DCD Section 3A.7.3. The RB/FB, CB and FWSC stick models are shown in DCD Figures 3A.7-4, 3A.7-6, and 3A.7-7, respectively. The plate elements for basemat and basement exterior walls, and overall site-specific SSI structural models are shown in Figures 3.7.2-201 through 3.7.2-203 for the RB/FB, Figures 3.7.2-204 through 3.7.2-206 for the CB, and Figures 3.7.2-207 through 3.7.2-209 for the FWSC. These figures also present the excavated soil volume elements that are part of the SASSI structural models. The excavated soil volumes have mesh that is consistent with the FE mesh of the basemats and basement exterior walls finite elements. The excavated soil elements are assigned with in-situ strain compatible soil properties as the ones used in the site profile models.

SASSI2010 criteria that the size of the elements shall be at least one fifth of the wave length to be able to accurately pass the seismic wave is used to determine the mesh size of the site and excavated volume models. The passing and cut-off frequencies are shown in Table 3.7.2-201. The meshes of the embedded finite element models used for site-specific SSI analyses of the RB/FB and CB are refined enough to ensure passage of seismic waves with 50 Hz frequency in all directions for all subsurface profiles. The mesh of the finite element models of excavated volume and concrete fill under the FWSC foundation are identical and are sized to pass waves with 50 Hz frequency in all directions for the UB subsurface profile. Based on the SASSI2010 criteria, the FWSC models used for the SSI analyses of LB and BE soil profiles are capable of transmitting frequencies up to 19 Hz and 33 Hz, respectively. The SSI analyses for these two soil profiles are performed for frequencies up to 50 Hz and 29 Hz that are 50 percent higher than the SASSI2010 criteria. The review of the transfer function results from these two analyses indicate that the use of frequencies of analysis beyond those specified in the SASSI2010 manual does not compromise the accuracy of the results.

The in-situ subgrade in the SSI models is represented by horizontally infinite layers resting on surface of elastic half space. The site models used for the SSI analyses of the RB/FB, CB and FWSC model consist of 13, 17 and 22 layers, respectively. These layers are developed to match the original site profile using the equivalent wave travel time procedure for adjusted shear and compression wave velocities as shown below.

$$Vs_{ave} = \frac{H}{\sum_i d_i / Vs_i} \qquad Vp_{ave} = \frac{H}{\sum_i d_i / Vp_i}$$

where: H is the thickness of the adjusted layer, d_i, Vs_i and Vp_i are the thickness, shear wave and compression wave velocities of the layers in the original site profiles. The unit weight and damping ratios of the adjusted layers are determined as weighted averages with respect to the layer thickness.

The top of the half space in the RB/FB, CB and FWSC models is established at DCD elevation -42.7 m (135 ft NAVD88), -45.8 m (125 ft NAVD88) and -45.5 m (126 ft NAVD88), respectively. Consistent with SASSI manual recommendations, the half-space simulation consists of additional ten layers with viscous dashpots added at the base of the site

finite element model to account for the dissipation of energy at the model lower boundary. The half-space model has a thickness of 1.5 V_S/ f, where Vs is the shear wave velocity of the halfspace and f is the frequency of the analysis. The total depth of the site model used for SSI analyses of RB/FB, CB and FWSC is more than 95.9 m, 104.1 m and 116.3 m, respectively, which is close or exceeds two times the footprint dimension of the analyzed structure.

Concrete stiffness properties are assigned to the lumped-mass stick model elements based on the SASSI stress results for in-plane shear and out-of-plane bending moment. The shear stiffness of the stick elements is reduced by 50 percent, if the member in-plane shear stress exceeds concrete rupture stress ($3 \times f_c'^{0.5}$ psi, where f_c' is specified compressive strength of concrete, psi). The cracking criteria defined in ACI 349-01, Section 9.5.2.3 are used as the basis for reduction of the out-of-plane bending stiffness by 50 percent based on comparison with the bending moment results. The stress check to determine the state of concrete cracking is performed for the seismic response obtained from the SSI analysis of the BE profile. This is to maintain consistency with best estimate properties considered in the structural models.

Figure 3.7.2-210 shows in red the elements of the RB/FB lumped mass stick model that are assigned with reduced cracked concrete stiffness properties and SSE damping. The RB/FB SDOF oscillators that are shown in red are also assigned with reduced cracked concrete stiffness properties and SSE damping. Reduced cracked concrete stiffness properties and SSE damping are assigned to all stick elements of the CB lumped mass stick. The FWSC model is assigned full (uncracked concrete) stiffness properties and OBE damping.

The concrete fill surrounding the RB/FB and CB, and below the FWSC, is included in the structural models. In the site-specific SSI analyses, the concrete fill surrounding the RB/FB and CB, and below the FWSC, is modeled consistent with the mesh size of plate elements for basemat and exterior walls. 3-D spring elements are established at the interface between the concrete fill solid elements and the building shell elements. These spring elements are assigned global stiffness properties high enough to ensure they do not affect the dynamic properties of the analyzed SSI system. The interface spring elements provide spring force results that serve as input for calculation of the site-specific wall lateral pressure and foundation bearing pressure demands in Sections 3.8.4.5.6

~~and 3.8.5.5.2, respectively. The spring forces results also serve as input for calculation of seismic driving forces for the site-specific stability evaluations in Section 3.8.5.5.1.~~

~~The SASSI2010 model X-direction and Y-direction represent plant north-south (NS) and east-west (EW) directions, respectively. The positive X axis is oriented to the south. The positive Y axis is oriented to the east. The positive Z- axis is oriented upward.~~

~~3.7.2.4.1.5 Soil-Structure Interaction Analysis Cases~~

~~The site-specific SSI analyses cases are summarized in Table 3.7.2-202 for the RB/FB, Table 3.7.2-203 for the CB, and Table 3.7.2-204 for the FWSC. To account for variability in the subsurface material properties, BE, LB, and UB profiles are considered.~~

~~Each analysis case consists of three directions of excitation (two horizontal and one vertical) applied independently to the SSI model. The calculated resulting co-directional ISRS in the X-, Y-, and Z-directions are combined using the SRSS method. The ISRS are developed for responses at the edges of the building by taking into account coupling effects between vertical and rocking and between lateral and torsion motions. The co-directional response structural loads from each direction of excitation for each case are combined using the algebraic sum method in the time domain or the SRSS method to obtain the response due to the three components of the earthquake. The ISRS results obtained from the analysis of the BE, LB and UB profiles are enveloped to develop site-specific design ISRS having the maximum amplitude for each frequency. The structural responses obtained from the SSI analyses of BE, LB, and UB profiles are enveloped to obtain the site-specific design basis values for the maximum enveloping member forces, accelerations and displacements.~~

~~3.7.2.4.1.6 Soil-Structure Interaction Analysis Results~~

~~The following sections present the results of the site-specific SSI analyses for the BE, LB, and UB subsurface profiles. The site-specific SSI analyses results for response at key locations are compared herein with the standard seismic design envelopes presented in DCD Appendix 3A. Comparisons are provided for maximum seismic structural loads and ISRS.~~

~~The results of the SSI analyses for the spring contact forces at the foundation bottom are used to evaluate the potential loss of contact with~~

the subgrade due to foundation uplift. The plots of the contact pressures at the bottom of the foundation at the critical instances of time show that the contact ratio remains above 80 percent thus demonstrating that the potential uplift of the foundations have a negligible effect on the results of the SSI analyses.

3.7.2.4.1.6.1 **SSI Enveloping Maximum Structural Loads**

For the RB/FB model, the enveloping seismic loads from the site-specific SSI analyses based on the BE, LB, and UB subsurface profiles (herein called Unit 3 site-specific SSI enveloping seismic loads) are presented in Tables 3.7.2-205 through 3.7.2-209.

The site-specific SSI enveloping seismic loads for the RB/FB are presented in Table 3.7.2-205. The site-specific SSI enveloping seismic loads are compared with the enveloping seismic loads provided in DCD Table 3A.9-1a for the RB/FB stick model. Table 3.7.2-205 also presents the percentage ratio of the site-specific SSI enveloping seismic loads to the seismic loads used for the standard design of RB/FB structures. Table 3.7.2-205 shows that the site-specific seismic loads for the RB/FB are partially larger than the corresponding standard design loads. Table 3.7.2-216(a) shows the result of stress checks using a scale factor for RB/FB Wall. Scale factors are calculated as the maximum value of the ratios of site-specific to standard design structural load components. This is a conservative approach since not all load components contributing to stresses experience the same degree of increase. The calculated scale factors are then applied to the worst stress ratio of the standard design governing seismic load combination to Code allowable stress. This approach provides upper bound estimate for Unit 3 stresses since the scale factor determined from the seismic load alone is applied to the combined stress of seismic plus other loads. The estimates of site-specific stresses ratio would be smaller if the scale factor is applied to the seismic stress component only. All values of Unit 3 stress ratio are confirmed to be within the code allowable stress limits which demonstrate the applicability of the standard design of RB/FB structures for Unit 3 site conditions.

The site-specific SSI seismic loads for the Reinforced Concrete Containment Vessel (RCCV) stick model are presented in Table 3.7.2-206. The site-specific SSI seismic loads are compared with the corresponding standard design seismic loads provided in DCD Table 3A.9-1b. Table 3.7.2-206 also presents the percentage ratio of the

site-specific SSI seismic loads to the seismic loads used for standard design of the RCCV. Table 3.7.2-206 shows that the site-specific seismic loads for the RCCV are partially larger than the standard design seismic loads. Table 3.7.2-216(b) shows the result of stress check using a scale factor for RCCV Wall. All values of stress are confirmed to be within allowable stress, thus demonstrating the applicability of the standard design of the RCCV structures for Unit 3 site conditions.

The site-specific seismic loads for the Vent Wall/Pedestal stick model are presented in Table 3.7.2-207. The site-specific seismic loads are compared with the corresponding seismic loads provided in DCD Table 3A.9-1c and used for the standard design of Vent Wall/Pedestal structural members. Table 3.7.2-207 also presents the percentage ratio of the site-specific seismic loads to the seismic loads used for standard design of the Vent Wall/Pedestal structural members. Table 3.7.2-207 shows that the site-specific seismic loads for the Vent Wall/Pedestal are partially larger than the corresponding standard design seismic loads. Table 3.7.2-216(c) and (d) shows the result of the stress check using a scale factor for the Vent Wall/Pedestal Wall.The stresses at the Vent Wall and Pedestal bottom slightly exceed the allowable stress when the scale factor is applied to the combined stress of seismic plus others. However, when the scale factor is applied to the seismic stress alone, the combined stress of the Vent Wall/Pedestal structure is confirmed to be within stress allowables, thus demonstrating the applicability of the standard design of the Vent Wall/Pedestal wall for Unit 3 site conditions.

The site-specific seismic loads for the Reactor Shield Wall (RSW) are presented in Table 3.7.2-208. The site-specific seismic loads are compared with the corresponding RSW standard design seismic loads provided in DCD Table 3A.9-1d. Table 3.7.2-208 also presents for the RSW the percentage ratio of the site-specific seismic loads to the standard design enveloping seismic loads. Table 3.7.2-208 shows that the RSW site-specific seismic loads are partially larger than the corresponding seismic loads in DCD Table 3A.9-1d. Table 3.7.2-216(e) shows the result of a stress check using a scale factor. The RSW stress exceeds the allowable stress when the scale factor is applied to the combined stress of seismic plus others. However, when the scale factor is applied to seismic stress alone the combined stress of the RSW structure is confirmed to be within stress allowables, thus demonstrating

~~the applicability of the standard design of the RSW structure for Unit 3 site conditions.~~

~~As described in DCD Sections 3.7.2 and 3.7.2.3, the reactor pressure vessel (RPV) is not a primary structural component. A lumped mass stick model of the RPV is included in the SSI model to capture its dynamic interaction with the supporting structure. Although this model is not used for the design of the RPV, the responses obtained from the RPV lumped mass stick model are considered for the purpose of this site-specific evaluation. The site-specific SSI enveloping loads for the RPV stick model are presented in Table 3.7.2-209. The site-specific SSI enveloping seismic loads are compared with the standard design seismic loads provided in DCD Table 3A.9-1e for the RPV lumped mass stick model. Table 3.7.2-209 presents the percentage ratio of the site-specific SSI enveloping seismic loads to the standard design SSI analysis enveloping seismic loads for the RPV stick model. Table 3.7.2-209 shows that the site-specific SSI enveloping seismic loads for the RPV stick model exceed the DCD standard design SSI analysis enveloping seismic loads. To address these exceedances of the RPV seismic loads, a decoupled model of the RPV subsystem is analyzed using input SSE loads based on the results of the site-specific SSI analysis.~~

~~The site-specific seismic loads for CB stick model are presented in Table 3.7.2-217. The site-specific seismic loads are compared with the corresponding CB standard design seismic loads provided in DCD Table 3A.9-1f. Table 3.7.2-217 also presents the percentage ratio of the site-specific seismic loads to the corresponding seismic loads used for standard design of the CB structure. Table 3.7.2-217 shows that the CB site-specific seismic loads are partially larger than the corresponding standard design seismic loads. Table 3.7.2-219 shows the result of the stress check using a scale factor. As shown in Table 3.7.2-219, it is confirmed that all stresses are lower than stress allowables except for the wall between DCD elevations -7.4 m (250.9 ft NAVD88) and -2.0 m (268.7 ft NAVD88). However, further evaluation with the scale factor applied to seismic load alone confirms that the combined stress in this wall also remains within stress allowables, thus demonstrating the applicability of the standard design of the CB structure for Unit 3 site conditions.~~

~~For the FWSC model, the site-specific seismic loads obtained from site-specific SSI analyses of the Fire Water Storage Tank (FWS) and Fire~~

~~Pump Enclosure (FPE) stick models are presented in Tables 3.7.2-220 and 3.7.2-221 respectively. These site-specific seismic loads are compared with the corresponding FWSC standard design seismic loads provided in DCD Tables 3A.9-1g and 3A.9-1h. Table 3.7.2-220 and Table 3.7.2-221 also presents the percentage ratio of the site-specific seismic loads to the seismic loads used for the standard design of the FWSC structure. Tables 3.7.2-220 and 3.7.2-221 show that the site-specific FWSC seismic loads are lower than the corresponding standard design seismic loads except for torsion. The structural design considers seismic torsion loads that include both torsion loads due to eccentricities of the SSI analysis model and accidental torsion that is calculated as a product of 5 percent of the largest plan dimension of the building and the floor horizontal load. The comparisons in Table 3.7.2-222 demonstrate that the total torsion loads used in the standard design envelope the site-specific total torsion loads. The comparisons in Tables 3.7.2-220, 3.7.2-221, and 3.7.2-222 demonstrate the applicability of the standard design of the FWSC structures for Unit 3 site conditions.~~

~~The Unit 3 site-specific SSI enveloping maximum vertical accelerations for the RB/FB stick model are presented in Table 3.7.2-210. The Unit 3 site-specific SSI enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in DCD Table 3A.9-3a, for the RB/FB stick model. Table 3.7.2-210 also presents the percentage ratio of the site-specific SSI enveloping maximum vertical accelerations to the enveloping maximum vertical accelerations of the DCD for the RB/FB stick model. Table 3.7.2-210 shows that the Unit 3 site-specific SSI enveloping maximum vertical accelerations for the RB/FB stick model are partially larger than the DCD enveloping maximum vertical accelerations. Table 3.7.2-216(a) shows the result of the stress check using a scale factor for RB/FB Wall. All values of stress are confirmed to be within stress allowables.~~

~~The Unit 3 site-specific SSI enveloping maximum vertical accelerations for the RCCV stick model are presented in Table 3.7.2-211. The Unit 3 site-specific SSI enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in DCD Table 3A.9-3b, for the RCCV stick model. Table 3.7.2-211 also presents the percentage ratio of the Unit 3 site-specific SSI enveloping maximum vertical accelerations to the DCD enveloping maximum vertical~~

~~accelerations for the RCCV stick model. Table 3.7.2-211 shows that the site-specific SSI enveloping maximum vertical accelerations for the RCCV stick model are partially larger than the DCD enveloping maximum vertical accelerations. Table 3.7.2-216(b) shows the result of the stress check using a scale factor for RCCV Wall. All values of stress are confirmed to be within stress allowables.~~

~~The site-specific SSI enveloping maximum vertical accelerations for the Vent Wall/Pedestal stick model are presented in Table 3.7.2-212. The site-specific SSI enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in DCD Table 3A.9-3c, for the Vent Wall/Pedestal stick model. Table 3.7.2-212 also presents the percentage ratio of the site-specific SSI enveloping maximum vertical accelerations to the DCD enveloping maximum vertical accelerations for the Vent Wall/Pedestal stick model. Table 3.7.2-212 shows that the site-specific SSI enveloping maximum vertical accelerations for the Vent Wall/Pedestal stick model are partially larger than the DCD enveloping maximum vertical accelerations. Table 3.7.2-216(c) and (d) shows the result of the stress check using a scale factor for Vent Wall/Pedestal. The stresses at the Vent Wall and Pedestal bottom slightly exceed the allowable stress when the scale factor is applied to the combined stress of seismic plus others. However, when the scale factor is applied to the seismic stress alone, the combined stress of the Vent Wall/Pedestal structure is confirmed to be within stress allowables, thus demonstrating the applicability of the standard design of the Vent Wall/Pedestal structure for Unit 3 site conditions.~~

~~The site-specific SSI enveloping maximum vertical accelerations for the RSW stick model are presented in Table 3.7.2-213. The site-specific enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in the DCD Table 3A.9-3d. Table 3.7.2-213 also presents the percentage ratio of the site-specific enveloping maximum vertical accelerations to the DCD enveloping maximum vertical accelerations for the RSW stick model. Table 3.7.2-213 shows that the site-specific enveloping maximum vertical accelerations for the RSW stick model are larger than the DCD enveloping maximum vertical accelerations. Table 3.7.2-216(e) shows the result of a stress check using a scale factor. The RSW stress exceeds the allowable stress when the scale factor is applied to the combined~~

stress of seismic plus others. However, when the scale factor is applied to seismic stress alone, the combined stress of the RSW structure is confirmed to be within stress allowables, thus demonstrating the applicability of the standard design of the RSW structure for Unit 3 site conditions.

The site-specific SSI enveloping maximum vertical accelerations for the RB/FB Flexible Slab Oscillators are presented in Table 3.7.2-214. The site-specific SSI enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in the DCD Table 3A.9-3e, for the RB/FB Flexible Slab Oscillators. Table 3.7.2-214 also presents the percentage ratio of the site-specific SSI enveloping maximum vertical accelerations to the DCD enveloping maximum vertical accelerations for the RB/FB Flexible Slab Oscillators. Table 3.7.2-214 shows that the site-specific SSI enveloping maximum vertical accelerations for the RB/FB Flexible Slab Oscillators are partially larger than the DCD enveloping seismic loads. Table 3.7.2-216(f) shows the result of the stress checks of RB/FB flexible slabs performed using scale factors. Only the site-specific stress demands on the Suppression Pool (S/P) slab is larger than allowable stress. However, when the scale factor is applied to seismic stress alone, the combined stress of the S/P slab is only 6 percent larger than allowable. Because the strains under combined primary and secondary forces meet the ASME Code Section CC-3422.1(d) requirements, the design adequacy of the S/P is confirmed.

The site-specific SSI enveloping maximum horizontal accelerations for the RB/FB Wall Out-of-Plane Oscillators are presented in Table 3.7.2-215. The site-specific SSI enveloping maximum horizontal accelerations are compared with the enveloping maximum horizontal accelerations provided in DCD Table 3A.9-3f, for the RB/FB Wall Out-of-Plane Oscillators. Table 3.7.2-215 also presents the percentage ratio of the Unit 3 site-specific SSI enveloping maximum horizontal accelerations to the DCD enveloping maximum horizontal accelerations for the RB/FB Wall Out-of-Plane Oscillators. Table 3.7.2-215 shows that the Unit 3 site-specific SSI enveloping maximum horizontal accelerations for the RB/FB Wall Out-of-plane Oscillators are partially larger than the standard design values with a maximum exceedance of approximately 69 percent. These exceedances of the oscillator accelerations affect only the magnitude of the out-of-plane loads related to the mass participation

of the particular flexible mode of vibration of the wall. The design of the wall is performed using a total out of plane load that include the contribution of all flexible modes of vibrations represented by the oscillators and the remaining rigid mass response of the wall. The site-specific stress evaluations of the RB/FB flexible walls presented in Table 3.7.1-216(g) are based on the consideration of total out-of-plane loads. These evaluations confirm that the combined site-specific stress demands on the RB/FB flexible walls are all lower than the stress allowables.

The results of the Unit 3 site-specific SSI enveloping maximum vertical accelerations for the CB stick model are presented in Table 3.7.2-218. The enveloping maximum vertical accelerations are compared with the enveloping maximum vertical accelerations provided in the DCD Table 3A.9-3g, for the CB stick model. Table 3.7.2-218 also presents the percentage ratio of the site-specific SSI enveloping maximum vertical accelerations to the DCD enveloping maximum vertical accelerations for the CB stick model. Table 3.7.2-218 shows that the site-specific SSI enveloping maximum vertical accelerations for the CB stick model are partially larger than the DCD enveloping seismic loads. The stress checks in Table 3.7.2-219(a) performed using scale factors conservatively applied on the total load show that the stresses in all walls but one are below the stress allowable. Only the stresses in the wall between DCD elevations -7.4 m (250.9 ft NAVD88) and -2.0 m (268.7 ft NAVD88) are larger than allowable stresses. However, further evaluation with the scale factor applied to seismic load alone confirms that the combined stress in this wall also remains within stress allowable. The stress checks in Table 3.7.2-219(b) using scale factors applied on the total slab load confirm that the stress in the CB slabs are all lower than the stress allowable thus demonstrating the applicability of the standard design of the CB slabs for Unit 3 site.

The results of Unit 3 site-specific SSI analysis of FWSC stick model for enveloping maximum vertical accelerations at FWS and FPE lumped mass locations are presented in Tables 3.7.2-223 and 3.7.2-224 respectively. These site-specific enveloping maximum vertical accelerations are compared with those provided in the DCD Tables 3A.9-3h and 3A.9-3i. The percentage ratio of the FWSC site-specific enveloping maximum vertical accelerations to the DCD enveloping maximum vertical accelerations in Tables 3.7.2-223

and 3.7.2-224 show that the site-specific accelerations are lower than the DCD values thus demonstrating the applicability of standard design of FWS and FPE slabs for the Unit 3 conditions.

3.7.2.4.1.6.2 **Comparison of the Site-Specific SSI Floor Response Spectra**

The 5 percent damping ISRS obtained from the site-specific SSI analyses of RB/FB, CB and FWSC models for the BE, LB, and UB subsurface profiles are compared with the standard design enveloping ISRS presented in DCD Section 3A.9.2.

Figures 3.7.2-211 through 3.7.2-228 compare the 5 percent damping site-specific ISRS obtained from the site-specific SSI analysis of the RB/FB model with the corresponding standard design ISRS in DCD Section 3A.9.2. The comparisons show that the site-specific ISRS exceed the standard design ISRS for a range of frequencies. Peak broadened site-specific design ISRS are developed for all locations within the RB/FB to envelop the results obtained from the SSI analyses of RB/FB for the three site-specific subgrade conditions. The RB/FB site-specific SSE design floor response spectra at selected locations for critical damping ratios of 2, 3, 4, 5, 7, 10 and 20 percent are shown in Figures 3.7.2-229 through 3.7.2-246.

Figures 3.7.2-247 through 3.7.2-252 present the comparisons of, the 5 percent damping site-specific SSI ISRS obtained from the site-specific SSI analyses of the CB with the corresponding standard design ISRS presented in DCD Section 3A.9.2. The comparisons show that the CB site-specific SSI ISRS exceed the standard design ISRS for a range of frequencies. Peak broadened site-specific design ISRS are developed for all locations within the CB to envelop the results obtained from the SSI analyses of CB for the three site-specific subgrade conditions. The CB site-specific SSE design floor response spectra at selected locations for critical damping ratios of 2, 3, 4, 5, 7, 10 and 20 percent are shown in Figures 3.7.2-253 through 3.7.2-258.

Figure 3.7.2-259 through 3.7.2-270 present the comparison of the 5 percent damping site-specific ISRS obtained from the site-specific SSI analysis of the FWSC model with the corresponding standard design ISRS presented in DCD Section 3A.9.2. The comparisons show that FWSC site-specific ISRS exceed the standard design ISRS for a range of frequencies. Peak broadened site-specific design ISRS are developed for

~~all locations within the FWSC to envelop the results obtained from the SSI analyses of FWSC for the three site-specific subgrade conditions. The FWSC site-specific SSE design floor response spectra at selected locations for critical damping ratios of 2, 3, 4, 5, 7, 10 and 20 percent are shown in Figures 3.7.2-271 through 3.7.2-282.~~

~~3.7.2.4.1.7 **Site-Specific SSI Analyses Conclusions**~~

~~Based on the site-specific SSI analyses, the following conclusions apply to the Unit 3 site:~~

- ~~The structural design of the ESBWR Standard Plant based on the CSDRS ground motion and generic subgrade conditions, is applicable to the RB/FB, CB and FWSC Seismic Category I structures at the Unit 3 site.~~
- ~~Site-specific ISRS are developed for all damping values and locations within RB/FB, CB and FWSC and adopted for the design since the site-specific ISRS exceed the standard plant design basis ISRS at some locations for a range of frequencies. Site-specific seismic qualification and analyses are performed for the equipment and components at these locations to demonstrate that their standard design is applicable for Unit 3 site-specific conditions.~~

~~3.7.2.4.1.8 **Site-Specific Seismic Design and Analysis of Structures, Systems, and Components**~~

~~The Unit 3 seismic design and analysis of structures, systems, and components (SSCs), are based on SSE defined by the following two sets of the free-field outcrop spectra at the foundation level (bottom of the base slab):~~

1) ~~the CSDRS shown in the DCD Figures 2.0-1 and 2.0-2; and~~
2) ~~the site-specific FIRS for each individual structure (RB/FB, CB, and FWSC) defined in Figures 2.5.2-307, 2.5.2-308, and 2.5.2-312~~

~~Thus, where a SSC is required to meet Seismic Category I requirements or to withstand SSE, the two sets of spectra define the SSE for design and analysis of these SSCs for Unit 3.~~

~~However, as specified in DCD Section 2.0, liquefaction potential and slope stability evaluations are exceptions for each site and use the site-specific SSE.~~

~~The seismic design of systems and components is evaluated to both the ISRS input from the standard design CSDRS and the ISRS input from the Unit 3 FIRS. As described in Section 3.7.2.4.1.6.2, the ISRS obtained from the site-specific SSI analyses exceed the standard design ISRS at certain locations within the building for a range of frequencies. Peak broadened site-specific design floor response spectra are developed for these locations and used for seismic design and qualification of substructure, components and equipment. Since PCCS Condenser site-specific floor response spectra exceed the standard design spectra, additional analyses are performed using the methodology described in DCD Section 3G.1.5.4.1.5 to confirm that the acceptance criteria in DCD Tier 1 ITAAC 5 in Table 2.15.4-2 are met. The site-specific ISRS also exceed the standard design floor response spectra at the locations of the new fuel storage rack and spent fuel storage rack in the deep pit. Additional analyses are also performed for these nuclear fuel racks using the methodology described in DCD Reference 9.1-1 to confirm that the acceptance criteria in DCD Tier 1 ITAAC 1 and 2 in Table 2.5.6-1 are met.~~

~~Table 3.7.2-209 shows that the site-specific SSI analysis yield enveloping seismic loads for the RPV that exceed the corresponding DCD standard design enveloping seismic loads. The seismic capability of the RPV subsystem is verified through the DCD Tier 1, Table 2.1.1-3, ITAAC 6 based on the results of seismic analysis of a decoupled model of the RPV subsystem that use input SSE loads developed from the results of site-specific SSI analysis of the RB/FB model.~~

~~Based on the results of the site-specific SSI analysis for enveloping, seismic loads, it is demonstrated in Section 3.7.2.4.1.6.1 that the standard design of Seismic Category I structures envelops the Unit 3 site-specific structural demands. The site-specific SSE loads for the Seismic Category II, and Radwaste Building structures are developed in the same manner as for the Category I structures based on the results of site specific SSI analyses. The SSI analyses for the Seismic Category II and Radwaste Building structures are identified as site-specific ITAAC in COLA Part 10.~~

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

Add the following at the end of this section.

NAPS SUP 3.7-5

The locations of structures are provided in Figure 2.1-201 and DCD Figure 1.1-1.

NAPS DEP 3.7-1

The locations of structures around the Unit 3 power block area are depicted in the plant layout provided in Figure 2.1-201 and DCD Figure 1.1-1. All Non-Seismic Category I structures that are within the scope of the standard design are addressed in this section. Each other Non-Seismic Category structure that is outside the scope of the DCD is at least a distance of its height above grade from Seismic Category I structures. Thus, the collapse of any site-specific non-Seismic Category I SSC will not cause the non-Seismic Category I SSC to compromise the structural integrity and/or the functions of any Seismic Category I SSC.

Two sets of site-specific seismic response analyses are performed using the Unit 3 site-specific design ground motion and subgrade dynamic properties to demonstrate the adequacy of the standard plant:

- Site-specific SSI analyses of the stand alone TB, RW, SB, and Ancillary Diesel Building (ADB) structures following methodology consistent with the site-specific seismic SSI analyses of the Seismic Category I structures presented in Section 3.7.2.4.
- Site-specific seismic structure-soil-structure interaction (SSSI) analyses to evaluate any adverse effects of seismic interaction between the TB, RW, SB, and ADB structures and adjacent Seismic Category I structures, ~~as necessary~~ to ensure that the design precludes adverse interactions.

Results of these site-specific seismic SSI and seismic SSSI analyses will be discussed as part of the ITAAC completion package for the TB, RW, SB, and ADB structures to demonstrate that acceptance criteria in ITAAC Tables 2.4.15-1, 2.4.16-1, 2.4.17-1, and 2.4.18-1, respectively, are met.

3.7.2.8.1 Turbine Building

Replace the second paragraph with the following. Add the following at the end of this section.

NAPS DEP 3.7-1

[The site-specific SSI analysis and seismic evaluation of the Turbine Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4.1. A seismic design motion is used that is compatible to site-specific FIRS that are developed considering the site-specific subgrade conditions under the Turbine Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3. The site-specific effects of structure-soil-structure interaction with adjacent Seismic Category I structures are evaluated, if necessary, following an approach consistent with the one used for the standard design that is described in DCD Section 3A.8.11.

The Turbine Building location is shown in Figure 2.1-201. The seismic gaps between the Turbine Building and the Reactor Building are no less than the calculated maximum relative displacements between the two buildings during an SSE event, considering out of phase motion.] *

NAPS DEP 3.7-1

The site-specific SSI analysis and seismic evaluation of the Turbine Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4. A seismic design motion is used that is compatible with site-specific FIRS that are developed considering the site-specific subgrade conditions under the Turbine Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible with the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3. To ensure the site-specific design precludes adverse interactions, an approach consistent with the

* Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.

approach used for the Seismic Category I structures is used to evaluate the effects of seismic SSSI with adjacent Seismic Category I structures.

The Turbine Building location is shown in Figure 2.1-201 and DCD Figure 1.1-1. The seismic gaps between the Turbine Building and the Reactor Building are no less than the calculated maximum relative displacements between the two buildings during an SSE event, considering out-of phase motion.

3.7.2.8.2 Radwaste Building

~~Replace the second paragraph with the following.~~ Add the following at the end of this section.

NAPS DEP 3.7-1

~~[*The site-specific SSI analysis and seismic evaluation of the Radwaste Building (RW) are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4.1. A seismic design motion is used that is compatible to site-specific FIRS that are developed considering the site-specific subgrade conditions under the RW foundation. The development of these FIRS follows the same methodology as the one used for the development of the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis of RW uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3. The site-specific effects of structure-soil-structure interaction with adjacent Seismic Category I structures are evaluated, if necessary, following an approach consistent with the one used for the standard design that is described in DCD Section 3A.8.11.*~~

~~*The RW location is shown in Figure 2.1-201. It is at least 10 meters from the RB. The building height is shown in DCD Figure 1.2-25.*]*~~

NAPS DEP 3.7-1

The site-specific SSI analysis and seismic evaluation of the Radwaste Building (RW) are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4. A seismic design motion is used that is compatible with site-specific FIRS that are developed considering the site-specific subgrade conditions under the RW foundation. The development of these FIRS follows the same methodology as the one used for the development of the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis of RW uses subgrade dynamic properties that are compatible to

the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3. To ensure the site-specific design precludes adverse interactions, an approach consistent with the approach used for the Seismic Category I structures is used to evaluate the effects of seismic SSSI with adjacent Seismic Category I structures.

The RW location is shown in Figure 2.1-201 and DCD Figure 1.1-1. It is at least 10 meters from the RB. The building height is shown in DCD Figure 1.2-25.

Add the following at the end of this section.

NAPS SUP 3.7-8

To meet the requirements for safe separation distance from the liquid hydrogen storage tanks, the RW exterior walls have a static wall pressure capacity of at least 3 psi. based on analysis using the exterior wall thickness, wall span, and wall reinforcement ratio as defined by ACI 349, as referenced in RG 1.143 (see DCD Sections 3.8.4.4.2 and 3.8.4.5.4).

3.7.2.8.3 Service Building

~~Replace the second paragraph with the following.~~ Add the following at the end of this section.

~~**NAPS DEP 3.7-1**~~

~~[*The site-specific SSI analysis and seismic evaluation of the Service Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4.1. A seismic design motion is used that is compatible to site-specific FIRS that are developed considering the site-specific subgrade conditions under the Service Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3.*~~

~~*The site-specific effects of structure-soil-structure interaction with adjacent Seismic Category I structures are evaluated, if necessary, following an approach consistent with the one used for the standard design that is described in DCD Section 3A.8.11.*~~

~~*The Service Building location is shown in Figure 2.1-201. The seismic gaps between the Service Building and the Reactor Building/Fuel*~~

~~*Building are no less than the calculated maximum relative displacements between the two buildings during an SSE event, considering out-of-phase motion.*]*~~

The site-specific SSI analysis and seismic evaluation of the Service Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4. A seismic design motion is used that is compatible with site-specific FIRS that are developed considering the site-specific subgrade conditions under the Service Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Seismic Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3.

To ensure the site-specific design precludes adverse interactions, an approach consistent with the approach used for the Seismic Category I structures is used to evaluate the effects of seismic SSSI with adjacent Seismic Category I structures.

The Service Building location is shown in Figure 2.1-201 and DCD Figure 1.1-1. The seismic gaps between the Service Building and the Reactor Building/Fuel Building are no less than the calculated maximum relative displacements between the two buildings during an SSE event, considering out-of-phase motion.

3.7.2.8.4 Ancillary Diesel Building

~~Replace the second paragraph with the following:~~ Add the following at the end of this section.

NAPS DEP 3.7-1

~~[*The site-specific SSI analysis and seismic evaluation of the Ancillary Diesel Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4.1. A seismic design motion is used that is compatible to site-specific FIRS that are developed considering the site-specific subgrade conditions under the Ancillary Diesel Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology*~~

~~*described in Section 3.7.1.1.3. The site-specific effects of structure-soil-structure interaction with adjacent Seismic Category I structures are evaluated, if necessary, following an approach consistent with the one used for the standard design that is described in DCD Section 3A.8.11.*~~

~~*The Ancillary Diesel Building location is shown in Figure 2.1-201. It is at least 15.2 meters from the Fuel Building.*]*~~

NAPS DEP 3.7-1

The site-specific SSI analysis and seismic evaluation of the Ancillary Diesel Building are performed following the same methodology as the one used for the Seismic Category I buildings in Section 3.7.2.4. A seismic design motion is used that is compatible with site-specific FIRS that are developed considering the site-specific subgrade conditions under the Ancillary Diesel Building. The development of these FIRS follows the same methodology as the one used for the FIRS for Category I buildings in Section 2.5.2. The site-specific SSI analysis uses subgrade dynamic properties that are compatible to the strains generated by the site-specific ground motion and are developed using the methodology described in Section 3.7.1.1.3. To ensure the site-specific design precludes adverse interactions, an approach consistent with the approach used for the Seismic Category I structures is used to evaluate the effects of seismic SSSI with adjacent Seismic Category I structures.

The Ancillary Diesel Building location is shown in Figure 2.1-201 and DCD Figure 1.1-1. It is at least 15.2 meters from the Fuel Building.

* ~~Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.~~

Table 3.7.2-201 [Deleted]

Table 3.7.2-202 [Deleted]

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3.7.3.13 Seismic Category I Buried Piping, Conduits and Tunnels

Replace the sixth paragraph sixth bullet as follows.

NAPS DEP 3.7-1

- [*Seismic input motions are based on the single envelope design response spectra as defined in DCD Table 3.7-2, using the applicable scale factor, and site-specific SSE FIRS.*]*

Replace the seventh paragraph as follows.

[*Seismic Category I utilities and Safety Class RW-IIa radwaste piping installed in trenches or tunnels are analyzed in accordance with the standard requirements of DCD Section 3.7.3. Seismic input motions for the portions located below ground are based on the single envelope design response spectra as defined in DCD Table 3.7-2, using applicable scale factors, and site-specific SSE FIRS.*]*

Site-specific SSE FIRS define the design input ground motion at the bottom elevations of Seismic Category I trenches, tunnels and duct bank structures that contain piping and conduits following the same methodology as used for the development of full column FIRS for the design of Seismic Category I buildings as described in Section 2.5.2. These FIRS consider, as applicable, the variations of subgrade conditions and the strain-compatible dynamic properties of in-situ subgrade and/or backfill materials under and above these structures and components. The site-specific SSE FIRS are amplified as necessary to include the effects of the adjacent heavy foundations on the free field motion and address the effects of structure-soil-structure interaction on the seismic response of these buried piping, conduits and tunnels.

* Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.

3.7.4 Seismic Instrumentation

Add the following at the end of the first paragraph.

NAPS SUP 3.7-6

The seismic monitoring program described in this subsection, including the necessary test and operating procedures, will be implemented prior to receipt of fuel on site.

3.7.4.2 Location and Description of Instrumentation

Add the following after the second paragraph.

NAPS SUP 3.7-6

The subsurface geologic structure (layering) of the site is considered in determination of the location of the free-field seismic instrument at the site. Appropriate spectral ratios are applied to the acceleration response spectra and velocity response spectra of the recorded motion to account for potential differences between the subsurface geologic conditions at the location of the free-field instrument and the power block area.

3.7.4.4 Comparison of Measured and Predicted Responses

Replace the last sentence (including the bulletized list) of the first paragraph as follows:

NAPS DEP 3.7-1

The plant is shut down if the walkdown inspections discover damage to equipment that would affect the safe operation of the plant, or the recorded motion in the free-field in any of the three directions (two horizontal and one vertical) exceeds both the certified design and site-specific response spectrum limits and the cumulative absolute velocity limit as follows:

- Certified design response spectrum limit is exceeded if:
    - at frequencies between 2 and 10 Hz, the recorded response spectral accelerations of 5 percent damping exceed one-third of the corresponding CSDRS values or 0.2g, whichever is greater; or
    - at frequencies between 1 and 2 Hz, the recorded response spectral velocities of 5 percent damping exceed one-third of the corresponding CSDRS values or 6 in/sec (152.4 mm/sec), whichever is greater.
- Site-specific response spectrum limit is exceeded if:
    - at frequencies between 2 and 10 Hz, the recorded response spectral accelerations of 5 percent damping exceed the corresponding site dependent OBE at grade presented in Table 3.7.1-216 or 0.2g, whichever is greater; or
    - at frequencies between 1 and 2 Hz, the recorded response spectral velocities of 5 percent damping exceed the corresponding OBE values presented in Table 3.7.1-217 or 6 in/sec (152.4 mm/sec), whichever is greater

- Cumulative absolute velocity limit is exceeded if the cumulative absolute velocity value calculated according to the procedures in EPRI TR-100082, December 1991 (DCD Reference 3.7-12), is greater than 0.16g/sec.

3.7.5 Site-Specific Information

Replace DCD Section 3.7.5 with the following.

NAPS DEP 3.7-1

(1) [*See Table 2.0-201 and Section 3.7.1 for seismology requirements of site-specific SSE ground response spectra.*

(2) *See Table 2.0-201 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity and liquefaction potential. For sites not meeting the soil property requirements, a site-specific analysis is required to demonstrate the adequacy of the standard plant design. Site-specific SSI analyses for the Seismic Category I RB/FB, CB, and FWSC structures are described in Section 3.7.2 to demonstrate the adequacy of the standard plant design of these buildings.*]*

* Text sections that are bracketed and italicized with an asterisk following the brackets are designated as Tier 2*. Prior NRC approval is required to change.

3.7.6 References

3.7-201 Kramer, Steven L. (1996), Geotechnical Earthquake Engineering, Prentice-Hall, ISBN 0-13-374943-6.

3.7-202 U.S. Nuclear Regulatory Commission (2010), Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses, DC/COL-ISG-17, March 2010.

3.7-203 NEI White Paper, “Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation,” NEI, June 2009.

3.7-204 McGuire, R. K., W. J. Silva, and C. J. Costantino (2001), “Technical Basis for Revision of Regulatory Guidance on Design Ground Motions, Hazard- and Risk-consistent Ground Motion Spectra Guidelines,” prepared for Nuclear Regulatory Commission, NUREG/CR-6728.

3.7-205 American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.”

3.7-206 DC/COL-ISG-017, “Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses,” March 24, 2010.

3.7-207 Philippacopoulos, A. J., “Recommendations for Resolution of Public Comments on USI A-40, “Seismic Design Criteria”,” Brookhaven National Laboratory, NUREG/CR-5347, May 1989.

3.8 Seismic Category I Structures

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

~~Add the following at the end of this section.~~

~~The evaluations in Section 3.7.2.4.1.6.1 demonstrate the adequacy of the standard plant design of RB/FB, CB and FWSC structural members for Unit 3 site-specific seismic load demands. The evaluations are based on comparison of results from site-specific SSI analysis with the standard design seismic loads and structural member capacities. Section 3.8.4.5.6 demonstrates that the Unit 3 site-specific lateral pressure loads on below grade exterior walls are also enveloped by the standard design.~~

3.8.1 Concrete Containment

3.8.1.6 Material, Quality Control and Special Construction Techniques

3.8.1.6.1 Concrete

Add the following paragraph at the end of Section (2) Aggregates.

NAPS SUP 3.8-1

ASTM Standards C1260 and C1293 are used in testing aggregates for potential alkali-silica reactivity (ASR).

3.8.4 Other Seismic Category I Structures

~~3.8.4.5 Structural Acceptance Criteria~~

Add the following at the end of this section.

NAPS DEP 3.7-1

~~3.8.4.5.6 Below Grade Exterior Wall Design~~

~~Unit 3 exterior below grade wall designs for the RB/FB and CB are evaluated through comparison of site-specific lateral pressure demands with the lateral loads used for standard design. The results of the Unit 3 site-specific SSI analyses for the RB/FB and CB presented in Section 3.7.2.4.1 provide the Unit 3 site-specific seismic lateral pressure loads applied to the below grade exterior walls of RB/FB and CB. The applicability of the standard design of below grade exterior walls is demonstrated by showing that the lateral loads used for standard design envelope the site-specific lateral load demands.~~

Figures 3.8.4-201 through 3.8.4-204 show the Unit 3 site-specific lateral soil pressure demands on each of the four exterior walls of the RB/FB. The distributions of the site-specific total lateral pressures that include the site-specific hydrostatic pressure, the static lateral pressure and the dynamic lateral pressure obtained from Unit 3 site-specific SSI analyses are presented. The figures also present the distribution of the site-specific passive resistance pressures on the walls that are required for the sliding stability of the Unit 3 RB/FB as discussed in Section 3.8.5.5. These site-specific lateral pressure demands are compared with the corresponding standard design lateral pressure loads, the standard design total soil pressure load and the passive pressure load considered for the standard design wall capacity check. The RB/FB wall capacity check for the standard design is performed conservatively considering the maximum passive resistance pressures required to meet the sliding stability of the building for the different generic soil conditions in conjunction with the static lateral pressure loads.

As shown in Figures 3.8.4-201 through 3.8.4-204, the Unit 3 lateral pressure demands on the walls of the RB/FB are bounded by the DCD design soil pressures and DCD wall capacity passive pressures except at the top of the Zone III rock. These sharp exceedances of the site-specific lateral pressures are due to SASSI calculations of the site-specific seismic lateral pressures at the top of the concrete fill model. Their effects on the out-of-plane bending moments and shear forces in the walls are small and are bounded by the standard design.

Figures 3.8.4-205 through 3.8.4-208 show the site-specific lateral soil pressure demands on each of the four exterior walls of the CB. The total lateral pressure demands include the site-specific hydrostatic pressure, the static lateral pressure and the dynamic lateral pressure obtained from site-specific SSI analyses of CB. The site-specific passive resistance pressures on the exterior walls are required for the sliding stability of the Unit 3 CB as discussed in Section 3.8.5.5. These site-specific lateral pressure demands are compared with the corresponding standard design lateral pressure loads, the standard design total soil pressure load and the passive pressure load considered for the standard design wall capacity check. The CB wall capacity check for the standard design is performed conservatively considering the maximum passive resistance pressures required to meet the sliding stability of the building for the

~~different generic soil conditions in conjunction with the static lateral pressure loads.~~

~~As shown in Figures 3.8.4-205 through 3.8.4-208, the Unit 3 lateral pressure demands on the CB exterior walls are bounded by the lateral pressure loads considered for the standard design of the CB structure except at the top of the Zone III rock. These sharp exceedances of the site-specific lateral pressures are due to SASSI calculations of the site-specific seismic lateral pressures at the top of the concrete fill model. Their effects on the out-of-plane bending moments and shear forces in the CB walls are small and are bounded by the standard design.~~

Unit 3 site-specific structural evaluations for the RB/FB, CB, and FWSC are described in Sections 3G.7 through 3G.10.

3.8.5 Foundations

~~3.8.5.5 Structural Acceptance Criteria~~

Add the following at the end of this section.

NAPS DEP 3.7-1

~~3.8.5.5.1 Foundation Stability~~

~~Unit 3 site-specific foundation stability for the RB/FB, CB, and FWSC are evaluated against overturning and sliding based on the results from the Unit 3 site-specific SSI analyses for the RB/FB, CB, and FWSC presented in Section 3.7.2.4.1. The stability evaluation for overturning and sliding follow the methodology in DCD Section 3.8.5.5.~~

~~The sliding stability of the building is evaluated on time step basis by calculating safety factor for different instances of time. The minimum value obtained during the duration of the site-specific ground motion is adopted as the safety factor for sliding stability of the building. A 0.03 second moving average window is applied on these time histories to obtain the lateral resistance force demands for the RB/FB, CB, and FWSC foundation. The applied moving average window helps to filter out the spurious peak in the vertical reaction time history when the magnitude of the upward seismic force is near or exceeds the effective weight of the building.~~

~~The factor of safety against overturning due to earthquake loading is determined by the energy approach method for both NS and EW direction as described in DCD Section 3.7.2.14. The calculated site-specific factors of safety against overturning based on Unit 3~~

site-specific SSI for the RB/FB, CB, and FWSC are shown in Tables 3.8.5-201, 3.8.5-202, and 3.8.5-203, respectively. The Unit 3 site-specific factors of safety against overturning for the RB/FB, CB, and FWSC are larger than the required minimum factor of safety of 1.1.

Unit 3 site-specific sliding evaluation is performed using driving forces calculated from the results of the site-specific SSI analyses, which neglects the effect of the structural fill placed above the top of the Zone III rock. The sliding shear resistance forces (F_{ub}) are calculated using the Unit 3 site-specific value for static sliding coefficient of friction of 0.60 for the critical sliding planes located at interface of the RB/FB, CB, and FWSC foundations with the underlying concrete fill and/or Zone III-IV rock. The base shear resistance values are calculated considering the effect of vertical component of the input earthquake motion and the ground water buoyancy forces that are calculated based on the Unit 3 nominal maximum ground water level for RB/FB, CB, and FWSC. The site-specific sliding evaluations for the Unit 3 RB/FB and CB also consider the lateral resistance provided by the concrete fill and Zone III rock. The stability evaluations neglect the resistance provided by the following skin friction resistance forces:

1. F_{us} = Skin friction resistance force provided by basemat side parallel to the direction of motion (i.e., F_{us} = 0)

2. F_{us}' = Skin friction resistance force provided by shear key side parallel to the direction of motion (when shear keys are used (i.e., F_{us}' = 0).

The lateral resistance provided by the RB/FB shear key (F_r') is also neglected so no credit is taken for the shear key in the RB/FB stability evaluations. The calculated Unit 3 site-specific factors of safety against sliding for the RB/FB and CB are shown in Tables 3.8.5-201 and 3.8.5-202, respectively. The table lists the maximum values of the lateral resistance forces and pressures along the RB/FB and CB embedded exterior walls and basemat opposite to the direction of motion that are needed to achieve a minimum factor of safety of 1.1 against sliding. The corresponding maximum required lateral passive resistance pressures are 0.20 MPa for RB/FB and 0.50 MPa for CB. These site-specific passive resistance pressures are well within the allowable bearing pressure of the concrete fill, Zone III rock and the lateral pressure capacity of the buildings below grade walls as shown in

Figures 3.8.4-201 through 3.8.4-208. These lateral passive resistance forces are associated with very small deformation of the concrete fill that will not result in motion of the foundation relative to the supporting subgrade. Therefore, the use of static coefficient of friction to calculate the base shear resistance against sliding is adequate.

The sliding of the FWSC is evaluated using driving seismic forces obtained from the Unit 3 site-specific SSI analyses. The site-specific sliding evaluations for the Unit 3 FWSC also consider the lateral resistance provided by the shear keys embedded in concrete fill below the FWSC foundation. The calculated Unit 3 site-specific factors of safety against sliding for the FWSC are shown in Table 3.8.5-203. The table lists the maximum values of the lateral resistance forces and pressures along the FWSC shear key opposite to the direction of motion that are needed to achieve a minimum factor of safety of 1.1 against sliding. The corresponding maximum required lateral passive resistance pressure is 0.50 MPa. This site-specific passive resistance pressure is well within the allowable bearing pressure of concrete fill and the lateral pressure capacity of the shear key.

3.8.5.5.2 **Foundation Dynamic Bearing Pressures**

The maximum soil dynamic bearing pressure demands for the RB/FB, CB and FWSC foundation basemats at Unit 3 site are calculated using the Energy Balance Method/Modified Energy Balance Method described in DCD Reference 3G.1-2. The results of the Unit 3 site-specific SSI analyses for BE, UB, and LB subsurface profiles presented in Section 3.7.2.4.1, provide time histories of the vertical seismic force and overturning seismic moment base reactions for each one of the three Seismic Category I foundations. A 0.03 second moving average window is applied on these time histories to obtain the bearing pressure demands under the CB foundation. The applied moving average window helps to filter out the spurious peak in the vertical reaction time history when the magnitude of the upward seismic force is near or exceeds the effective weight of the building. At this instance of time, the corresponding angle of rotation of the basemat predicted by the soil-structure interaction model is extremely small. The same moving average window approach is used to calculate the contact ratio of the CB foundation directly from the results of SSI analysis as described in Section 3.7.2.4.1.6. The maximum values of the dynamic pressures calculated for the whole duration of the site-specific ground motion are selected to represent the maximum

~~site-specific bearing pressure demands under the RB/FB, CB, and FWSC basemats.~~

~~The Unit 3 site-specific maximum dynamic soil bearing pressure demands for the RB/FB, CB, and FWSC foundations are shown in Tables 3.8.5-204, 3.8.5-205 and 3.8.5-206, respectively.~~

Unit 3 site-specific structural evaluations for the RB/FB, CB, and FWSC foundations are described in Sections 3G.7 through 3G.10.

Table 3.8.5-201 ~~Factors of safety for RB/FB Foundation Stability~~ (Deleted)

	Overturning		Sliding	
Load Combination	SRP 3.8.5 Required FS	Calculated FS	SRP 3.8.5 Required FS	Calculated FS
D + H + E'	1.1	533	1.1	≥1.1

Where,
D = Dead Load
H = Lateral soil pressure
E' = Safe Shutdown Earthquake
Note: The maximum required lateral resistance force is 110 MN.
The maximum required lateral pressure is 200 kPa

Table 3.8.5-202 ~~Factors of safety for CB Foundation Stability~~ (Deleted)

	Overturning		Sliding	
Load Combination	SRP 3.8.5 Required FS	Calculated FS	SRP 3.8.5 Required FS	Calculated FS
D + H + E'	1.1	559	1.1	≥1.1

Where,
D = Dead Load
H = Lateral soil pressure
E' = Safe Shutdown Earthquake
Note: The maximum required lateral resistance force is 53 MN.
The maximum required lateral pressure is 500 kPa

Table 3.8.5-203 ~~Factors of safety for FWSC Foundation Stability~~ (Deleted)

~~Load Combination~~	~~Overturning SRP 3.8.5 Required FS~~	~~Overturning Calculated FS~~	~~Sliding SRP 3.8.5 Required FS~~	~~Sliding Calculated FS~~
~~D + H + E'~~	~~1.1~~	~~902~~	~~1.1~~	~~≥1.1~~

~~Where,~~
~~D = Dead Load~~
~~H = Lateral soil pressure~~
~~E' = Safe Shutdown Earthquake~~
~~Note: The maximum required lateral resistance force is 43 MN.~~
~~The maximum required lateral pressure is 500 kPa~~

Table 3.8.5-204 ~~Maximum Soil Dynamic Bearing Pressure Demand for RB/FB (Unit: kPa)~~ (Deleted)

~~Site Conditions~~	~~Lower Bound Subsurface Profile~~	~~Best Estimate Subsurface Profile~~	~~Upper Bound Subsurface Profile~~
~~Dynamic (Static + Seismic)~~	~~1150~~	~~1170~~	~~1150~~

Table 3.8.5-205 ~~Maximum Soil Dynamic Bearing Pressure Demand for CB (Unit: kPa)~~ (Deleted)

~~Site Conditions~~	~~Lower Bound Subsurface Profile~~	~~Best Estimate Subsurface Profile~~	~~Upper Bound Subsurface Profile~~
~~Dynamic (Static + Seismic)~~	~~490~~	~~510~~	~~520~~

Table 3.8.5-206 ~~Maximum Soil Dynamic Bearing Pressure Demand for FWSC (Unit: kPa)~~ (Deleted)

~~Site Conditions~~	~~Lower Bound Subsurface Profile~~	~~Best Estimate Subsurface Profile~~	~~Upper Bound Subsurface Profile~~
~~Dynamic (Static + Seismic)~~	~~420~~	~~420~~	~~400~~

Figure 3.8.4-201 (Deleted)

Figure 3.8.4-202 (Deleted)

Figure 3.8.4-203 (Deleted)

Figure 3.8.4-204 (Deleted)

Figure 3.8.4-205 (Deleted)

Figure 3.8.4-206 (Deleted)

Figure 3.8.4-207 (Deleted)

Figure 3.8.4-208 (Deleted)

3.9 Mechanical Systems and Components

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.9.2.4 Initial Startup Flow-Induced Vibration Testing of Reactor Internals

Replace the last paragraph with the following.

CWR COL 3.9.9-1-A

1. For reactor internals other than the steam dryer, the vibration assessment program, as specified in Regulatory Guide (RG) 1.20, is provided in DCD Appendix 3L and the following referenced GEH Report:

    - NEDE-33259P-A, “Reactor Internals Flow Induced Vibration Program”

    The classification of the Unit 3 reactor internals in accordance with RG 1.20 is dependent on ESBWR status; i.e., if Unit 3 is the initial ESBWR to perform testing of the reactor internals or if testing is performed at another reactor prior to Unit 3 testing. There are two different scenarios:

    a. A valid prototype for the Unit 3 reactor internals does not exist. Under this scenario, Unit 3 reactor internals classification is a prototype per RG 1.20.

    b. A valid prototype for Unit 3 reactor internals does exist. If the prototype testing is performed outside the United States, the guidance in RG 1.20, Revision 3, Regulatory Position 1.2, would need to be satisfied in order for this reactor to be considered a “valid prototype.” Assuming that Unit 3 reactor internals are substantially similar to the valid prototype and that the valid prototype does not experience inservice problems that result in component or operational modifications, Unit 3 reactor internals will be classified as non-prototype Category I. If a change to the classification for Unit 3 reactor internals is later determined to be necessary, the classification change will be addressed at the time the change is proposed with proper evaluation/justification and documented in a revision to the FSAR.

2. Specific to the steam dryer, the comprehensive vibration assessment program, as specified in RG 1.20, is provided in DCD Appendix 3L and the following referenced GEH Reports:
   - NEDE-33312P, “ESBWR Steam Dryer Acoustic Load Definition”
   - NEDE-33313P, “ESBWR Steam Dryer Structural Evaluation”
   - NEDE-33408P, “ESBWR Steam Dryer – Plant Based Load Evaluation Methodology – PBLE01 Model Description”

   The steam dryer is classified as a prototype according to RG 1.20, Revision 3. Section 10.2 of NEDE-33313P provides four elements of a steam dryer Comprehensive Vibration Assessment Program that must be addressed. The following describes the approach for the steam dryer Comprehensive Vibration Assessment Program elements, consistent with RG 1.20 and Section 10.2 of NEDE-33313P:

   a. The ESBWR steam dryer Comprehensive Vibration Assessment Program is described in DCD Section 3.9, DCD Appendix 3L, and NEDE-33313P, Section 10.0, which includes a description for preparing and submitting to the NRC a Steam Dryer Monitoring Plan no later than 90 days before initial fuel load.

   b. The detailed design of the steam dryer will follow the methodology described in DCD Appendix 3L and the incorporated engineering reports. As described in NEDE-33313P, Section 10.2(b), an example of a steam dryer predictive analysis that concludes the steam dryer will not exceed stress limits with applicable bias and uncertainties and the minimum alternating stress ratio of 2.0 is provided in NEDE-33408P. The final detailed design of the ESBWR steam dryer has not yet been completed. Therefore, the example of an as-designed steam dryer that has been subjected to the predictive analysis process and successful startup testing described in NEDE-33408P serves as the design analysis report for the steam dryer and provides sufficient information for licensing. The post-licensing commitments in ITAAC and license conditions confirm the acceptability of the ESBWR steam dryer design.

c. The startup program and associated steam dryer license conditions that include appropriate notification points during power ascension, providing data to the NRC at certain hold points and at full power, and providing to the NRC a full stress analysis report and evaluation within 90 days of reaching the full power level, are established in accordance with NEDE-33313P, Section 10.2(c).

d. Periodic steam dryer inspection during refueling outages is as described in NEDE-33313P, Section 10.2(d), and associated license conditions.

3. Summary of Reactor Internals Vibration Assessment Program

For reactor internals other than the steam dryer, the comprehensive vibration assessment program will be developed and implemented as described in DCD Appendix 3L with no departures. The vibration measurement and inspection programs will comply with the guidance specified in RG 1.20, Revision 3, consistent with the Unit 3 reactor internals classification. A summary of the vibration analysis program and description of the vibration measurement (including measurement locations and analysis predictions) and inspection phases of the comprehensive vibration inspection program will be submitted to the NRC six months prior to implementation.

For reactor internals other than the steam dryer, the preliminary and final reports (as necessary), which together summarize the results of the vibration analysis, measurement and inspection programs will be submitted to the NRC within 60 and 180 days, respectively, following the completion of the programs.

3.9.3.1 Loading Combinations, Design Transients and Stress Limits

Replace the fifth paragraph with the following.

STD COL 3.9.9-2-A The equipment stress reports identified in this DCD section will be completed within six months of completion of DCD ITAAC Table 3.1-1. The FSAR will be revised as necessary in a subsequent update to address the results of this analysis.

3.9.3.7.1(3)e Snubber Preservice and Inservice Examination and Testing

Preservice Examination and Testing

Add the following at the end of this section.

STD COL 3.9.9-4-A

A preservice thermal movement examination is also performed; during initial system heatup and cooldown, for systems whose design operating temperature exceeds 121°C (250°F), snubber thermal movement is verified.

Additionally, preservice operational readiness testing is performed on all snubbers. The operational readiness test is performed to verify the parameters of ISTD-5120. Snubbers that fail the preservice operational readiness test are evaluated to determine the cause of failure, and are retested following completion of corrective action(s).

Snubbers that are installed incorrectly or otherwise fail preservice testing requirements are re-installed correctly, adjusted, modified, repaired or replaced, as required. Preservice examination and testing is re-performed on installation-corrected, adjusted, modified, repaired or replaced snubbers as required.

The preservice inspection and testing programs for snubbers will be completed in accordance with milestones described in Section 13.4.

Inservice Examination and Testing

Add the following at the beginning of this section.

STD COL 3.9.9-4-A

Inservice examination and testing of all safety-related snubbers is conducted in accordance with the requirements of the ASME OM Code, Subsection ISTD. Inservice examination is initially performed not less than two months after attaining 5 percent reactor power operation and will be completed within 12 calendar months after attaining 5 percent reactor power. Subsequent examinations are performed at intervals defined by ISTD-4252 and Table ISTD-4252-1. Examination intervals, subsequent to the third interval, are adjusted based on the number of unacceptable snubbers identified in the then current interval.

An inservice visual examination is performed on all snubbers to identify physical damage, leakage, corrosion, degradation, indication of binding, misalignment or deformation and potential defects generic to a particular

design. Snubbers that do not meet visual examination requirements are evaluated to determine the root cause of the unacceptability, and appropriate corrective actions (e.g., snubber is adjusted, repaired, modified, or replaced) are taken. Snubbers evaluated as unacceptable during visual examination may be accepted for continued service by successful completion of an operational readiness test.

Snubbers are tested inservice to determine operational readiness during each fuel cycle, beginning no sooner than 60 days before the scheduled start of the applicable refueling outage. Snubber operational readiness tests are conducted with the snubber in the as-found condition, to the extent practical, either in place or on a test bench, to verify the test parameters of ISTD-5210. When an in-place test or bench test cannot be performed, snubber subcomponents that control the parameters to be verified are examined and tested. Preservice examinations are performed on snubbers after reinstallation when bench testing is used (ISTD-5224), or on snubbers where individual subcomponents are reinstalled after examination (ISTD-5225).

Defined test plan groups (DTPG) are established and the snubbers of each DTPG are tested according to an established sampling plan each fuel cycle. Sample plan size and composition are determined as required for the selected sample plan, with additional sampling as may be required for that sample plan based on test failures and failure modes identified. Snubbers that do not meet test requirements are evaluated to determine root cause of the failure, and are assigned to failure mode groups (FMG) based on the evaluation, unless the failure is considered unexplained or isolated. The number of unexplained snubber failures not assigned to an FMG determines the additional testing sample. Isolated failures do not require additional testing. For unacceptable snubbers, additional testing is conducted for the DTPG or FMG until the appropriate sample plan completion criteria are satisfied.

Unacceptable snubbers are adjusted, repaired, modified, or replaced. Replacement snubbers meet the requirements of ISTD-1600. Post-maintenance examination and testing, and examination and testing of repaired snubbers, is done to ensure that test parameters that may have been affected by the repair or maintenance activity are verified acceptable.

Service life for snubbers is established, monitored and adjusted as required by ISTD-6000 and the guidance of ASME OM Code Nonmandatory Appendix F.

The inservice inspection and testing programs for snubbers will be completed in accordance with milestones described in Section 13.4.

Delete the last two sentences of the last paragraph.

3.9.3.7.1(3)e Snubber Support Data

Replace the first sentence with the following.

STD COL 3.9.9-4-A For the ASME Class 1, 2, and 3 systems listed in DCD Tier 1, Section 3.1, that contain snubbers, a plant-specific table will be prepared in conjunction with the closure of the system-specific ITAAC for piping and component design and will include the following specific snubber information.

Add the following at the end of this section.

STD COL 3.9.9-4-A This information will be included in the FSAR as part of a subsequent FSAR update.

3.9.6 Inservice Testing of Pumps and Valves

Replace the last sentence of the last paragraph with the following.

STD COL 3.9.9-3-A Milestones for implementation of the ASME OM Code preservice and inservice testing programs are defined in Section 13.4.

3.9.6.1 Inservice Testing of Valves

Add the following before the last paragraph.

STD COL 3.9.9-3-A Each valve subject to inservice testing is also tested during the preservice test (PST) period. Preservice tests are conducted under conditions as near as practicable to those expected during subsequent inservice testing. Valves (or the control system) that have undergone maintenance that could affect performance, or valves that are repaired or replaced, are re-tested to verify performance parameters that could have been affected are within acceptable limits. Safety and relief valves and nonreclosing pressure relief devices are preservice tested in accordance with the requirements of the ASME OM Code, Mandatory Appendix I.

3.9.6.1.4 Valve Testing

Add the following at the end of the introduction to this section.

STD COL 3.9.9-3-A

Other specific testing requirements for power-operated valves include stroke-time testing and, as applicable, diagnostic testing to evaluate valve condition and to verify the valve will continue to function under design-basis conditions.

(1) Valve Exercise Tests

Add the following after the second sentence of the first paragraph.

NAPS COL 3.9.9-3-A

Valves are tested by full-stroke exercising, during operation at power, to the positions required to fulfill their functions.

Add the following after the third sentence of the first paragraph.

STD COL 3.9.9-3-A

If full-stroke exercising is not practicable, part-stroke exercising is performed during operation at power or during cold shutdown.

Add the following new paragraph after the first paragraph.

STD COL 3.9.9-3-A

During extended shutdowns, valves that are required to be operable must remain capable of performing their intended safety function. Exercising valves during cold shutdown commences within 48 hours of achieving cold shutdown and continues until testing is complete or the plant is ready to return to operation at power. Valve testing required to be performed during a refueling outage is completed before returning the plant to operation at power.

Add the following after the first sentence of the second paragraph.

STD COL 3.9.9-3-A

Valve testing uses reference values determined from the results of PST or IST. These tests that establish reference values are performed under conditions as near as practicable to those expected during the IST. Stroke time is measured and compared to the reference value, except for valves classified as fast-acting (e.g., solenoid-operated valves (SOVs) with stroke time less than 2 seconds), for which a stroke time limit of 2 seconds is assigned.

Add the following after the third paragraph.

STD COL 3.9.9-3-A

SOVs are tested to confirm the valves move to their energized positions and are maintained in those positions, and to confirm that the valves move to the appropriate failure mode positions when de-energized.

Pre-conditioning of valves or their associated actuators or controls prior to IST undermines the purpose of IST and is prohibited. Pre-conditioning includes manipulation, pre-testing, maintenance, lubrication, cleaning, exercising, stroking, operating, or disturbing the valve to be tested in any way, except as may occur in an unscheduled, unplanned, and unanticipated manner during normal operation.

(4) Special Tests

Add the following after the second paragraph under the second bullet.

STD COL 3.9.9-3-A

Industry and regulatory guidance is considered in development of IST program for explosively actuated valves. In addition, the IST program for explosively actuated valves incorporates lessons learned from the design and qualification process for these valves such that surveillance activities provide reasonable assurance of the operational readiness of explosively actuated valves to perform their safety functions.

3.9.6.1.5 Specific Valve Test Requirements

(1) Power-Operated Valve Tests

Replace the last paragraph with the following.

STD COL 3.9.9-3-A

Section 3.9.6.8 describes additional (non-Code) testing of power-operated valves as discussed in Regulatory Issue Summary 2000-03.

(3) Check Valve Exercise Tests

Add the following as the first sentence of the second paragraph.

STD COL 3.9.9-3-A

Check valve testing requires verification that obturator movement is in the direction required for the valve to perform its safety function.

Add the following before the last paragraph.

STD COL 3.9.9-3-A

Acceptance criteria for this testing consider the specific system design and valve application. For example, a valve's safety function may require obturator movement in both open and closed directions. A mechanical exerciser may be used to operate a check valve for testing. Where a mechanical exerciser is used, acceptance criteria are provided for the force or torque required to move the check valve's obturator. Exercise tests also detect missing, sticking, or binding obturators.

If these test methods are impractical for certain check valves, or if sufficient flow cannot be achieved or verified, a sample disassembly examination program verifies valve obturator movement. The sample disassembly examination program groups check valves by category of similar design, application, and service condition.

During the disassembly process, the full-stroke motion of the obturator is verified. Nondestructive examination is performed on the hinge pin to assess wear, and seat contact surfaces are examined to verify adequate contact. Full-stroke motion of the obturator is re-verified immediately prior to completing reassembly. At least one valve from each group is disassembled and examined at each refueling outage, and all the valves in each group are disassembled and examined at least once every eight years. Before being returned to service, valves disassembled for examination or valves that received maintenance that could affect their performance are exercised with a full- or part-stroke. Details and bases of the sampling program are documented and recorded in the test plan.

When operating conditions, valve design, valve location, or other considerations prevent direct observation or measurements by use of conventional methods to determine adequate check valve function, diagnostic equipment and nonintrusive techniques are used to monitor internal conditions. Nonintrusive tests used are dependent on system and valve configuration, valve design and materials, and include methods such as ultrasonic (acoustic), magnetic, radiography, and use of

accelerometers to measure system and valve operating parameters (e.g., fluid flow, disk position, disk movement, disk impact, and the presence or absence of cavitation and back-tapping). Nonintrusive techniques also detect valve degradation. Diagnostic equipment and techniques used for valve operability determinations are verified as effective and accurate under the PST program.

Testing is performed, to the extent practical, under normal operation, cold shutdown, or refueling conditions applicable to each check valve. Testing includes effects created by sudden starting and stopping of pumps, if applicable, or other conditions, such as flow reversal. When maintenance that could affect valve performance is performed on a valve in the IST program, post-maintenance testing is conducted prior to returning the valve to service.

Preoperational testing is performed during the initial test program (refer to Section 14.2) to verify that valves are installed in a configuration that allows correct operation, testing, and maintenance. Preoperational testing verifies that piping design features accommodate check valve testing requirements. Tests also verify disk movement to and from the seat and determine, without disassembly, that the valve disk positions correctly, fully opens or fully closes as expected, and remains stable in the open position under the full spectrum of system design-basis fluid flow conditions.

Data acquired during check valve testing and inspections, and the maintenance history of a valve or group of valves is collected and maintained in order to establish the basis for specifying inservice testing, examination, and preventive maintenance activities that will identify and/or mitigate the failure of the check valves or groups of check valves tested. This data is also used to determine if certain check valve condition monitoring tests, such as nonintrusive tests, are feasible and effective in monitoring for these identified failure mechanisms, whether periodic disassembly and examination activities would be effective in monitoring for these failure mechanisms, as well as to determine possible valve groupings to implement in a future check valve condition monitoring program as allowed by ISTC-5222, the requirements of which are described in ASME OM Code, Appendix II.

3.9.6.5 Valve Replacement, Repair and Maintenance

Add the following to the end of the paragraph.

STD COL 3.9.9-3-A

When a valve or its control system has been replaced, repaired, or has undergone maintenance that could affect valve performance, a new reference value is determined, or the previous value is reconfirmed by an inservice test. This test is performed before the valve is returned to service, or immediately if the valve is not removed from service. Deviations between the previous and new reference values are identified and analyzed. Verification that the new values represent acceptable operation is documented.

3.9.6.6 10 CFR 50.55a Relief Requests and Code Cases

Add the following at the end of the first paragraph.

STD SUP 3.9-1

No relief from or alternative to the ASME OM Code is being requested.

3.9.6.7 Inservice Testing Program Implementation

Delete the last paragraph.

3.9.6.8 Non-Code Testing of Power-Operated Valves

Replace the second sentence of the first paragraph with the following.

STD COL 3.9.9-3-A

These tests, which are typically performed under static (no flow or pressure) conditions, also document the “baseline” performance of the valves to support maintenance and trending programs.

Replace the fifth sentence of the first paragraph with the following.

STD COL 3.9.9-3-A

Uncertainties associated with performance of these tests and use of the test results (including those associated with measurement equipment and potential degradation mechanisms) are addressed appropriately.

Replace the last sentence of the first paragraph with the following.

STD COL 3.9.9-3-A

Uncertainties affecting both valve function and structural limits are addressed.

Replace the second paragraph with the following.

STD COL 3.9.9-3-A

Additional testing is performed as part of the air-operated valve (AOV) program, which includes the key elements for an AOV Program as identified in the JOG AOV program document, Joint Owners Group Air Operated Valve Program Document, Revision 1, December 13, 2000 (References 3.9.201 and 3.9.202). The AOV program incorporates the attributes for a successful power-operated valve long-term periodic verification program, as discussed in RIS 2000-03, Resolution of Generic Safety Issue 158: Performance of Safety-related Power-Operated Valves Under Design Basis Conditions, (Reference 3.9.203) by incorporating lessons learned from previous nuclear power plant operations and research programs as they apply to the periodic testing of air- and other power-operated valves included in the IST program. For example, key lessons learned addressed in the AOV program include:

- Valves are categorized according to their safety significance and risk ranking.
- Setpoints for AOVs are defined based on current vendor information or valve qualification diagnostic testing, such that the valve is capable of performing its design-basis function(s).
- Periodic static testing is performed, at a minimum on high risk (high safety significance) valves, to identify potential degradation, unless those valves are periodically cycled during normal plant operation under conditions that meet or exceed the worst case operating conditions within the licensing basis of the plant for the valve, which would provide adequate periodic demonstration of AOV capability. If required based on valve qualification or operating experience, periodic dynamic testing is performed to re-verify the capability of the valve to perform its required functions.
- Sufficient diagnostics are used to collect relevant data (e.g., valve stem thrust and torque, fluid pressure and temperature, stroke time, operating and/or control air pressure, etc.) to verify the valve meets the functional requirements of the qualification specification.
- Test frequency is specified, and is evaluated each refueling outage based on data trends as a result of testing. Frequency for periodic testing is in accordance with References 3.9.201 and 3.9.202, with a minimum of 5 years (or 3 refueling cycles) of data collected and evaluated before extending test intervals.

- Post-maintenance procedures include appropriate instructions and criteria to ensure baseline testing is re-performed as necessary when maintenance on the valve, valve repair or replacement, have the potential to affect valve functional performance.
- Guidance is included to address lessons learned from other valve programs in procedures and training specific to the AOV program.
- Documentation from AOV testing, including maintenance records and records from the corrective action program are retained and periodically evaluated as a part of the AOV program.

The attributes of the AOV testing program described above, to the extent that they apply to and can be implemented on other safety-related power-operated valves, such as electro-hydraulic valves, are applied to those other power-operated valves.

3.9.7 Risk-Informed Inservice Testing

Replace this section with the following.

STD SUP 3.9-2 Risk informed inservice testing is not being utilized.

3.9.8 Risk-Informed Inservice Inspection of Piping

Replace this section with the following.

STD SUP 3.9-3 Risk informed inservice inspection is not being utilized.

3.9.9 COL Information

3.9.9-1-A Reactor Internals Vibration Analysis, Measurement and Inspection Program

CWR COL 3.9.9-1-A This COL item is addressed in Section 3.9.2.4.

3.9.9-2-A ASME Class 2 or 3 or Quality Group D Components with 60 Year Design Life

STD COL 3.9.9-2-A This COL item is addressed in Section 3.9.3.1.

3.9.9-3-A Inservice Testing Programs

STD COL 3.9.9-3-A
NAPS COL 3.9.9-3-A This COL item is addressed in Section 3.9.6.

3.9.9-4-A Snubber Inspection and Test Program

STD COL 3.9.9-4-A This COL item is addressed in Section 3.9.3.7.1(3)e and Section 3.9.3.7.1(3)e.

3.9.10 References

3.9.201 Joint Owners Group Air Operated Valve Program Document, Revision 1, December 13, 2000.

3.9.202 USNRC, Eugene V. Imbro, letter to Mr. David J. Modeen, Nuclear Energy Institute, Comments On Joint Owners' Group Air Operated Valve Program Document, October 8, 1999.

3.9.203 Regulatory Issue Summary 2000-03, Resolution of Generic Safety Issue 158: Performance of Safety-related Power-Operated Valves Under Design Basis Conditions, March 15, 2000.

3.10 Seismic and Dynamic Qualification of Mechanical and Electrical Equipment

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.10.1.4 Dynamic Qualification Report

Replace the last paragraph with the following.

STD COL 3.10.4-1-A

An implementation schedule for completing ITAAC will be provided to the NRC no later than 1 year after issuance of the combined license or at the start of construction as defined in 10 CFR 50.10(a), whichever is later. Dominion shall submit updates to the ITAAC schedules every 6 months thereafter and, within 1 year of its scheduled date for initial loading of fuel, and shall submit updates to the ITAAC schedules every 30 days until the final notification is provided to the NRC under paragraph 10 CFR 52.99(c)(1).

The Dynamic Qualification Report and documentation that describe the seismic and dynamic qualification methods will be made available for NRC staff review, inspection, and audit. Information that verifies the seismic and dynamic qualification will be made available to the NRC to facilitate reviews, inspections, and audits throughout the process. FSAR information will be revised, as necessary, as part of a subsequent FSAR update.

STD SUP 3.10-1

Section 17.5 defines the Quality Assurance Program requirements that are applied to equipment qualification files, including requirements for handling safety-related quality records, control of purchased material, equipment and services, test control, and other quality related processes.

3.10.4 COL Information

3.10.4-1-A **Dynamic Qualification Report**

STD COL 3.10.4-1-A

This COL item is addressed in Section 3.10.1.4.

3.11 Environmental Qualification of Mechanical and Electrical Equipment

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3.11.4.4 Environmental Qualification Documentation

Replace the last paragraph with the following.

STD COL 3.11-1-A

The documentation necessary to support the continued qualification of the equipment installed in the plant that is within the Environmental Qualification (EQ) Program scope is available in accordance with 10 CFR 50 Appendix A, General Design Criterion 1. EQ files are maintained for equipment and certain post-accident monitoring devices that are subject to a harsh environment. The files are maintained for the operational life of the plant.

Central to the EQ Program is the EQ Master Equipment List (EQMEL). The EQMEL identifies the electrical and mechanical equipment or components that must be environmentally qualified for use in a harsh environment. The EQMEL consists of equipment that is essential to emergency reactor shutdown, containment isolation, reactor core cooling, or containment and reactor heat removal, or that is otherwise essential in preventing a significant release of radioactive material to the environment. This list is developed from the equipment list provided in DCD Table 3.11-1. The EQMEL and a summary of equipment qualification results are maintained as part of the equipment qualification file for the operational life of the plant.

Administrative programs are in place to control revision to the EQ files and the EQMEL. When adding or modifying components in the EQ Program, EQ files are generated or revised to support qualification. The EQMEL is revised to reflect these new components. To delete a component from the EQ Program requires a deletion justification to be prepared that demonstrates why the component can be deleted. This justification consists of an analysis of the component, an associated circuit review if appropriate, and a safety evaluation. The justification is released and/or referenced on an appropriate change document.

For changes to the EQMEL, supporting documentation is completed and approved prior to issuing the changes. This documentation includes safety reviews and new or revised EQ files. Plant modifications and

design basis changes are subject to change process reviews, e.g., reviews in accordance with 10 CFR 50.59 or the change control requirements of the ESBWR-specific appendix to 10 CFR Part 52, in accordance with appropriate plant procedures. These reviews address EQ issues associated with the activity. Any changes to the EQMEL that are not the result of a modification or design basis change are subject to a separate review that is accomplished and documented in accordance with plant procedures.

Engineering change documents or maintenance documents generated to document work performed on an EQ component are reviewed against the current revision of the EQ files for potential impact. Changes to EQ documentation may be due to, but not limited to, plant modifications, calculations, corrective maintenance, or other EQ concerns.

The operational aspects of the EQ program include:

- Evaluation of EQ results for design life to establish activities to support continued EQ
- Determination of surveillance and preventive maintenance activities based on EQ results
- Consideration of EQ maintenance recommendations from equipment vendors
- Evaluation of operating experience in developing surveillance and preventive maintenance activities for specific equipment
- Development of plant procedures that specify individual equipment identification, appropriate references, installation requirements, surveillance and maintenance requirements, post-maintenance testing requirements, condition monitoring requirements, replacement part identification, and applicable design changes and modifications
- Development of plant procedures for reviewing equipment performance and EQ operational activities, and for trending the results to incorporate lessons learned through appropriate modifications to the operational EQ program
- Development of plant procedures for the control and maintenance of EQ records

Implementation of the environmental qualification program, including development of the plant specific Environmental Qualification Document (EQD), will be in accordance with the milestone defined in Section 13.4.

3.11.7 COL Information

3.11-1-A Environmental Qualification Document

STD COL 3.11-1-A

This COL item is addressed in Section 3.11.4.4.

CWR SUP 3.12-1

3.12 Piping Design Review

Information on seismic Category I and II, and nonseismic piping analysis and their associated supports is presented in Sections 3.7, 3.9, 3D, 3K, 5.2 and 5.4.

STD SUP 3.13-1

3.13 Threaded Fasteners - ASME Code Class 1, 2, and 3

Criteria applied to the selection of materials, design, inspection and testing of threaded fasteners (i.e., threaded bolts, studs, etc.) are presented in DCD Section 3.9.3.9, with supporting information in DCD Sections 4.5.1, 5.2.3, and 6.1.1.

Appendix 3A Seismic Soil-Structure Interaction Analysis

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

3A.1 Introduction

Add at the beginning of this section.

NAPS DEP 3.7-1
~~**NAPS CDI**~~

~~The information in DCD Appendix 3A is the basis for the ESBWR Standard Plant SSI analyses. Chapters 2 and 3 contain updated site-specific information that interfaces with DCD Appendix 3A. The site-specific SSI analyses and results are provided in Sections 3.7.2, 3.8.4, and 3.8.5. Site-specific geotechnical data is described in Chapter 2. The site plan is shown in Figure 2.1-201. Where the North Anna 3 Early Site Permit site-specific information is discussed in DCD Appendix 3A, it is related to the basis for the ESBWR Standard Design SSI analyses and is updated in Chapters 2 and 3 for site-specific applicability. The information in DCD Appendix 3A is maintained and applies to the extent that it establishes methods for performing analyses and provides the basis for the ESBWR Standard Plant SSI analysis.~~

DCD Sections 3A.1 through 3A.9 provide information related to the ESBWR standard design SSI analyses for generic site conditions and analysis cases. This information provides the basis and results for the ESBWR Standard Plant SSI analyses that were performed using the CSDRS vibratory ground motion. The North Anna 3 Early Site Permit site-specific information that is discussed in Sections 3A.1 through 3A.9 is used as input to the ESBWR standard design SSI analyses.

Sections 3A.10 through 3A.19 provide information related to the Unit 3 site-specific SSI analyses. Site-specific SSI analyses were performed to evaluate the Unit 3 site conditions. The site-specific SSI analyses are described, and the results are provided, in Sections 3.7.2, 3.8.4, 3.8.5, and 3A.10 through 3A.19. Sections 3A.10 through 3A.18 correspond to DCD Sections 3A.1 through 3A.9, and Section 3A.19 summarizes the site-specific analyses results.

Both the ESBWR standard design SSI analyses and the Unit 3 site-specific SSI analyses are used in the seismic design, analyses, and qualification of plant structures, systems, and components.

3A.2 ESBWR Standard Plant Site Plan

Replace the first two sentences of the first paragraph with the following.

NAPS CDI

The site plan is shown in Figure 2.1-201. The plan orientation is denoted on the figure.

3A.9 Site Envelope Seismic Responses

Add the following at the end of this section.

NAPS DEP 3.7-1

3A.10 Unit 3 Site-Specific Seismic Soil-Structure Interaction Analysis Introduction

Sections 3A.10 through 3A.19 present the SSI analysis for Unit 3 to establish the site-specific seismic design basis for the RB, FB, CB, and FWSC for the ESBWR Standard Plant Seismic Category I structures. The RB and FB are integrated and founded on a common basemat and termed RB/FB. The FWSC is composed of two Firewater Storage Tanks (FWS) and a Fire Pump Enclosure (FPE), which are founded on a common basemat.

As described in Section 2.0, the site-specific horizontal and vertical FIRS have been compared to the corresponding CSDRS used for the design of the ESBWR standard plant. These comparisons show that there are ranges of frequencies where both the horizontal and vertical site-specific FIRS exceed the CSDRS. In accordance with the requirements of DCD Tier 1, Section 5.1, to address these exceedances, site-specific SSI analyses are performed for Unit 3 Seismic Category I RB/FB, CB, and FWSC structures using the Unit 3 site-specific ground motion and strain compatible soil properties. These analyses are supplemental to the CSDRS standard design analyses.

The ESBWR standard design based on the CSDRS remains applicable and the site-specific analyses are performed to address the Unit 3 site conditions. The site-specific SSE design ground motion at the foundation level and the dynamic properties of Unit 3 subgrade materials are described in Section 3.7.1. The site-specific SSI analysis results are presented in the form of site-enveloped seismic responses at key locations in the RB/FB, CB, and FWSC. Both the standard design analyses and the site-specific analyses results are used in the seismic design, analyses, and qualification of SSCs for Unit 3. The structural

adequacy evaluations for the RB/FB, CB, and FWSC are described in Appendix 3G.

The Unit 3 site-specific SSI analyses for the RB/FB, CB, and FWSC follow the standard design methodology described in DCD Section 3A.5.2 using the SASSI2010 computer program and, for the RB/FB, the ACS SASSI computer program. The SASSI2010 and ACS SASSI computer programs use exactly the same methodology as the SASSI2000 computer program used for the standard design SSI analyses to provide the solution for the seismic response of the structure-subgrade interaction system based on the frequency domain complex response method. The SASSI structural (HOUSE) models are developed from the standard design SASSI model described in Section 3A.16 coupled with site-specific strain compatible dynamic subsurface properties.

The site-specific SSI analyses use structural models with upper bound stiffness properties that provide conservative seismic responses for the Unit 3 rock site with high frequency design ground motion. The results of the site-specific structural stiffness variation sensitivity analyses confirm that the site-specific design basis analyses that are based on the model with upper bound stiffness properties adequately address the effects of structural stiffness variations.

The far-field models used for the site-specific SSI analyses are assigned dynamic properties of the in-situ subgrade materials that are compatible to the strains generated by the site-specific design motion. The SSI models also include near-field solid elements representing the strain compatible dynamic properties of the concrete and structural fill materials backfilled below and around the buildings. The site-specific SSI analyses consider BE, LB, and UB properties of the subgrade materials to account for the effects of the potential variability in the properties of the subgrade materials at the Unit 3 site.

The site-specific SSI analyses for the RB/FB and CB consider two different embedment configurations representing:

1. the buildings being partially embedded (PE) up to the Zone III rock nominal top elevation, and

2. the buildings being fully embedded (FE) up to the finished ground level grade elevation.

The envelope of responses obtained from the RB/FB and CB analyses of the BE, LB and UB partial and full column profiles provides the Unit 3 site-specific seismic demands used for site-specific design and evaluation of the RB/FB and CB. These enveloping site-specific seismic demands are compared with the envelopes presented in DCD Section 3A.9 to determine exceedances of the Unit 3 site-specific seismic demands relative to the corresponding seismic loads used for the standard design.

The FWSC site-specific design basis is developed based on the envelope of the results obtained from site-specific SSI analyses of FWSC stand-alone model and SSSI analyses of FWSC-CB combined model performed with the input control motion applied:

1. as a surface motion compatible to the spectra defining the site-specific design motion at the bottom of the FWSC foundation at Elevation 282 ft NAVD88 (standard design Elevation 2.15 m), and

2. as an in-column motion compatible to the spectra defining the site-specific design motion at the bottom of the concrete fill at nominal Elevation 220 ft NAVD88 (standard design Elevation -16.8 m).

Consideration of two different control motion elevations captures the effects of seismic wave propagation through concrete fill supporting the FWSC basemat and thus addresses potential deamplification of the high frequency input motion as the seismic waves propagate through the in-situ saprolite. The FWSC site-specific design basis that is developed based on the envelope of results obtained from the analyses that use these two sets of input control motions meets the intent of DC/COL-ISG-017, which specifies that the spectra defining the design motion at the surface be enveloped by deterministic seismic response analyses at the top of the considered soil columns. In accordance with

SRP 3.7.2, three subsurface profiles are considered for each input control motion, namely, a BE profile, a LB profile, and a UB profile to account for the effects of the potential variability in the properties of the soils and rock at the FWSC location of the Unit 3 site.

The results of SSI analyses are also used as the basis for the site-specific seismic design evaluations of the RB/FB, CB, and FWSC, which are described in Appendix 3G.

3A.11 Unit 3 Plant Site Plan

The Unit 3 site plan is shown in Figure 2.1-201. Table 2.5.2-209 lists the size, depth, and loading of the Unit 3 structures. The model axes in the X-direction and Y-direction represent the north-south (NS) direction and the east-west (EW) direction of the Unit 3 site, respectively. These directions are referenced to plant north. Plant north for Unit 3 is oriented 23.54 degrees east of true north. The Z-axis represents the vertical direction.

Elevations in Sections 3A.10 through 3A.19 are given in feet NAVD88 and may include the elevation in meters for comparison to specific DCD elevations. As specified in Table 2.0-201, the design plant grade elevation identified in DCD Table 3.4-1 is 4650 mm, which corresponds to 88.4 m (Elevation 290.0 ft NAVD88) for the Unit 3 site as shown in Figure 2.1-201.

3A.12 Unit 3 Site Conditions

This section describes the site conditions used in the Unit 3 site-specific SSI analyses.

3A.12.1 Generic Site Conditions

Generic site conditions are not considered in the Unit 3 site-specific SSI analyses.

3A.12.2 Unit 3 Site-Specific Conditions

The Unit 3 conditions used in the site-specific SSI analyses are described in Section 3.7.1 and discussed below.

The geology of the Unit 3 site is discussed in detail in Section 2.5.1. The subsurface materials encountered at Unit 3 and the engineering properties of these subsurface materials are discussed in detail in Section 2.5.4. To account for the variability of the subgrade properties, SSI analyses are performed for three sets (BE, UB and LB) of dynamic

properties of subgrade materials. Development of the BE, LB, and UB site-specific strain compatible dynamic subsurface material properties is discussed in Section 3.7.1.1.4. The strain compatible dynamic properties used to develop the BE, LB, and UB subsurface profiles used in the site-specific SSI analyses are provided in Tables 3.7.1-201 and 3.7.1-202 for the RB/FB, Tables 3.7.1-203 and 3.7.1-204 for the CB, and Tables 3.7.1-205 and 3.7.1-206 for the FWSC. The site-specific SSI inputs are described below for each structure.

The RB/FB and CB site-specific SSI analyses are performed for two embedment configurations representing two limiting SSI stiffness conditions:

- LB stiffness represented by partial column profiles that neglect stiffness of softer soil materials above the rock top elevation
- UB stiffness represented by full column profiles simulating full contact of embedment with below-grade exterior walls (no soil separation) and fully saturated stiffness properties of soil below the Unit 3 nominal groundwater level

To address the variability of the subgrade properties, the RB/FB and CB SSI analyses are performed for partial and full column profiles representing BE, LB, and UB dynamic properties of in-situ materials compatible to the strains generated by the site-specific design ground motion.

The RB/FB and CB SSI analyses also consider BE, LB, and UB dynamic properties for the structural and concrete fill materials placed around the RB/FB and CB. The structural fill dynamic properties are compatible to the strains generated by the site-specific design ground motion. The dynamic properties used for the concrete fill are independent of strain, reflecting the linear elastic behavior of the concrete under the small earthquake-induced strains.

The seismic response analyses of the FWSC at the Unit 3 site consider BE, LB, and UB dynamic properties of the in-situ subgrade materials at the FWSC location that are compatible with the strains generated by the site-specific design ground motion. The site-specific strain-compatible dynamic properties of the in-situ rock and saprolite subgrade materials are assigned to the far-field models and the excavated volume elements of the FWSC models. The BE, LB and BE properties assigned to the near-field elements modeling the concrete fill placed below the FWSC

foundation are independent of strain, reflecting the linear elastic behavior of the concrete under the small earthquake-induced strains. The near-field elements in the FWSC-CB combined model representing the structural fill placed in the gap between the CB and FWSC and around the concrete fill below the FWSC foundation are assigned BE, LB and UB properties that are compatible to the strains generated by the input design motion.

The P-wave velocity of the saprolite and structural fill materials below the groundwater level is set equal to or close to that of the water to capture the effect of groundwater on P-wave velocity of saturated soil. A maximum value of 0.48 is used for the Poisson's ratio of subgrade materials to ensure the numerical stability of the analyses results.

The partial and full embedment configurations considered in the RB/FB and CB SSI analyses provide responses that bound the effects of subgrade stiffness variations related to the soil separation, backfill horizontal extent, and groundwater level variations and minimize the effects of dissipation of energy in the SSI system due to damping of the embedment materials. The partially embedded models provide an LB representation of the stiffness of the RB/FB and CB embedment because they do not consider the stiffness of the subgrade materials (saprolite and the structural fill) above the Zone III rock surface. The partially embedded models also neglect the dissipation of energy in the SSI system that is due to the material damping of the saprolite and the structural fill.

The RB/FB and CB fully embedded models consider a minimum horizontal extent of the structural fill to provide a UB representation of the stiffness of the embedment because the structural fill has a lower shear wave velocity (and thus lower shear modulus and stiffness) than the surrounding in-situ saprolite. Comparisons between the shear wave velocity and damping ratio of the structural fill and in-situ saprolite, provided in Figures 3A.12.2-201 and 3A.12.2-202, show that the structural fill has lower stiffness and higher damping properties than the surrounding in-situ saprolite. The same relationships are also shown in comparisons of the average strain-compatible shear wave velocities (Vs_{ave}) and shear column frequencies (f_{sc}) of the concrete fill, structural fill and in-situ materials presented in Tables 3A.12.2-201 and 3A.12.2-202. Therefore, the consideration of the minimum horizontal extent of the structural fill reduces the dissipation of energy in the SSI

system because the material damping of the structural fill is higher than that of the in-situ saprolite.

The FWSC SSI analyses also consider BE, LB, and UB dynamic properties for the concrete fill placed below the FWSC basemat. The dynamic properties used for the concrete fill are independent of strain, reflecting the linear elastic behavior of the concrete under the small earthquake-induced strains. These dynamic properties are assigned to the near-field solid elements in the models representing the block of concrete fill below the FWSC foundation.

The site-specific SSI analyses of the FWSC neglect the effects of the structural fill placed around the FWSC basemat and the concrete fill because the strain-compatible dynamic properties of the structural fill are similar to those of the surrounding in-situ saprolite. The comparisons presented in Figure 3A.12.2-203 show that the structural fill and the in-situ saprolite have similar shear wave velocities, compression wave velocities and damping. The comparisons presented in Table 3A.12.2-203 also show that the values of the average strain-compatible shear wave velocities (Vs_{ave}) and shear column frequencies (f_{sc}) of the structural and concrete fill are very close to those of the in-situ saprolite and rock. The FWSC-CB combined model used for the FWSC-CB SSSI analyses presented in Section 3A.17.11 includes the structural fill placed in the gap between the CB and FWSC and around the concrete fill below the FWSC foundation. The inclusion of the structural fill in the combined model addresses the effects of the structural fill on the FWSC seismic response.

NAPS DEP 3.7-1

Table 3A.12.2-201 **RB/FB Comparisons of the Average Strain-Compatible Shear Wave Velocities and Shear Column Frequencies of the Fill and In-Situ Materials**

	Concrete Fill/ Zone III Rock Embedment					Structural Fill/Saprolite Embedment				
		Backfill		In-Situ			Backfill		In-Situ	
Soil Case	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**
LB	14.8	1829	30.9	979	16.5	5.2	137	6.6	228	11.0
BE		2134	36.0	1318	22.3		213	10.3	360	17.4
UB		2438	41.2	1774	30.0		331	16.0	566	27.3

NAPS DEP 3.7-1

Table 3A.12.2-202 **Average Strain-Compatible Shear Wave Velocities and Shear Column Frequencies of the Fill and In-Situ Materials at CB Location**

	Concrete Fill/ Zone III Rock Embedment					Structural Fill/Saprolite Embedment				
		Backfill		In-Situ			Backfill		In-Situ	
Soil Case	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**
LB	7.3	1829	62.8	520	17.8	7.6	147	4.8	218	7.2
UB		2438	83.7	917	31.4		347	11.4	515	16.9

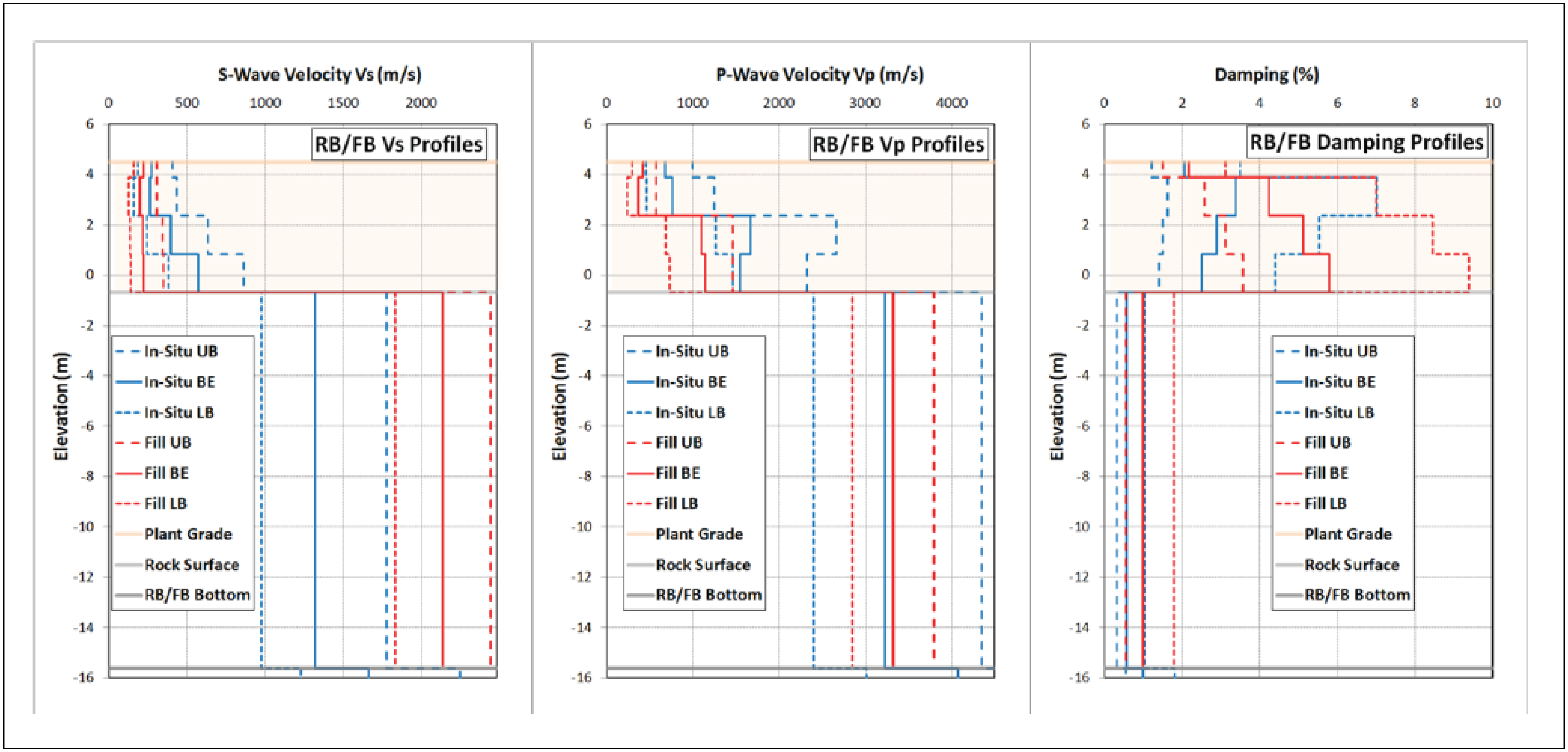
NAPS DEP 3.7-1

Table 3A.12.2-203 **FWSC Comparisons of the Average Strain-Compatible Shear Wave Velocities and Shear Column Frequencies of the In-Situ Materials**

	Zone III Rock			**Structural Fill/Saprolite**					**Full Column**				
					Backfill		**In-Situ**			**Backfill**		**In-Situ**	
Soil Case	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**Depth m**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**	**$V_{S\ ave}$ m/s**	**f_{sc} Hz**
LB		604	20.6		157	3.4	192	4.1		243	3.2	261	3.5
BE	7.3	781	26.7	11.7	252	5.4	298	6.4	19.0	383	5.1	391	5.2
UB		1010	34.5		405	8.7	461	9.9		598	7.9	584	7.7

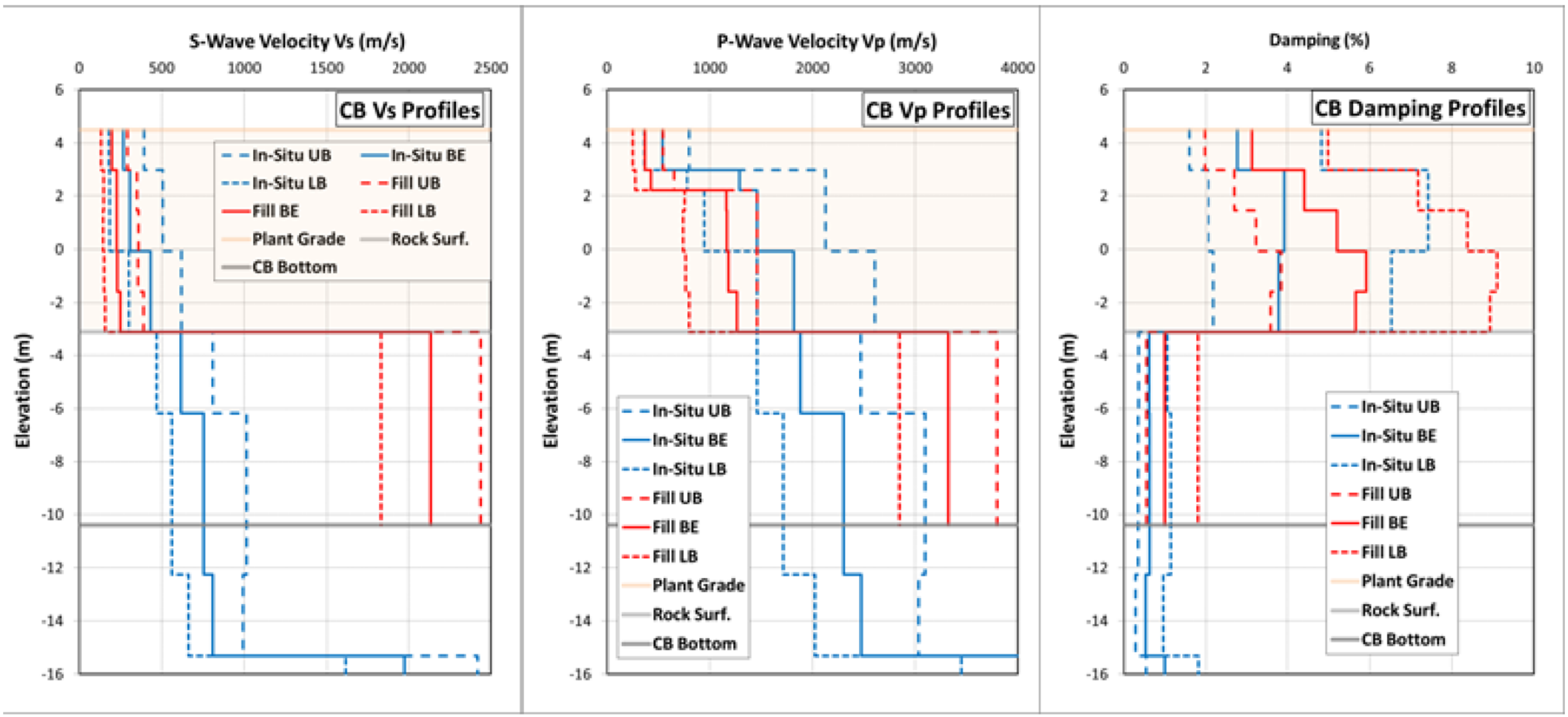
NAPS DEP 3.7-1

Figure 3A.12.2-201 **RB/FB Comparisons Between Shear Wave Velocity and Damping Ratio of Structural Fill and In-Situ Saprolite**



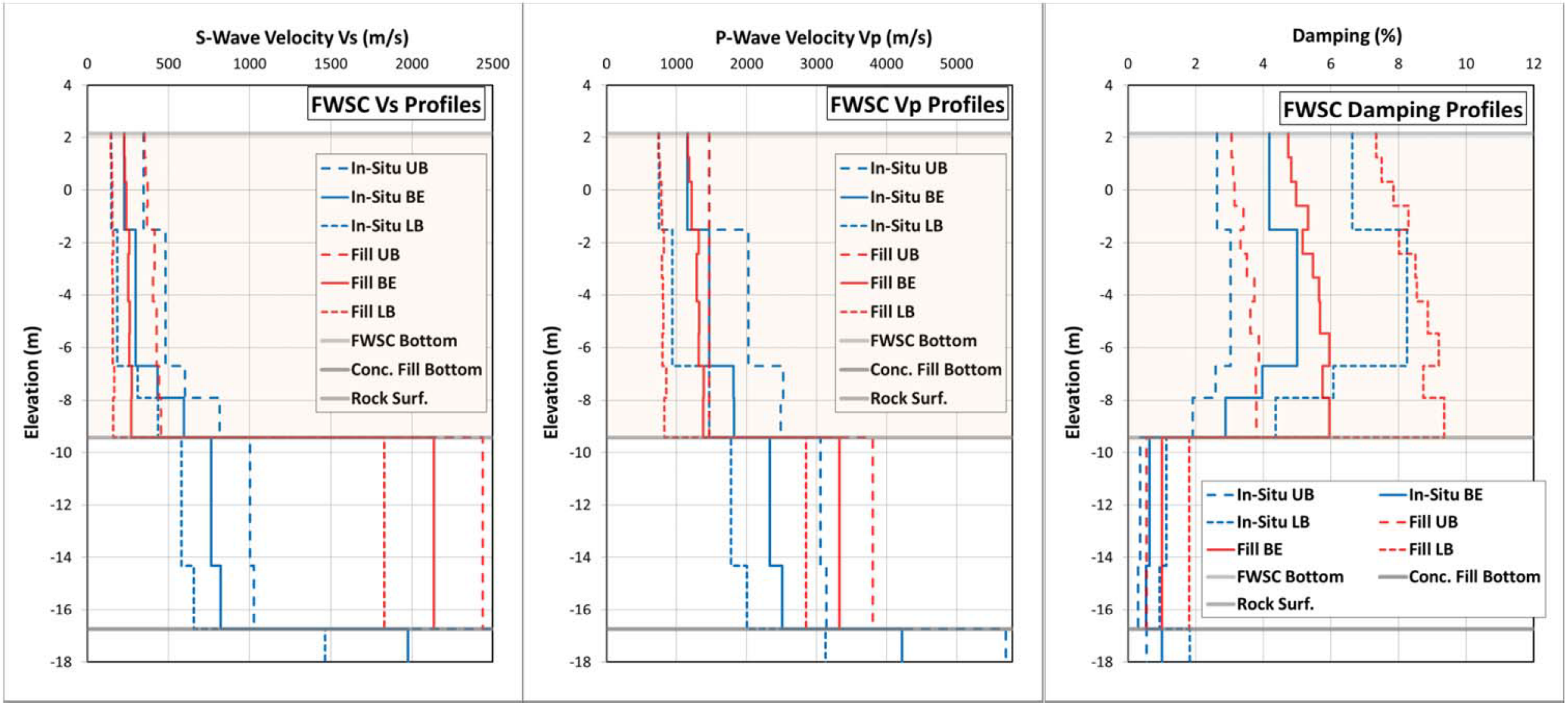
NAPS DEP 3.7-1

Figure 3A.12.2-202 **CB Comparisons Between Shear Wave Velocity and Damping Ratio of Structural Fill and In-Situ Saprolite**



NAPS DEP 3.7-1

Figure 3A.12.2-203 FWSC Comparisons Between Shear and Compression Wave Velocity and Damping Ratio of Structural Fill and In-Situ Saprolite



NAPS DEP 3.7-1

3A.13 Unit 3 Input Motion and Damping Values

3A.13.1 Unit 3 Input Motion

The time-history method is used to perform site-specific seismic response analyses. Section 3.7.1.1.5 describes development of the site-specific ground motion time histories used as input control motion in the site-specific SSI analyses.

Six sets of statistically independent ground motion time histories are used as input control motion in three orthogonal directions for the SSI analyses of the RB/FB and CB for the BE, LB, and UB partial and full column profiles. These acceleration time histories represent the in-column free-field motion at the RB/FB and CB foundation bottom elevation. The time histories are compatible with the envelope of the RB/FB FIRS and CB FIRS and the broadband spectra specified in RG 1.60 anchored at 0.1g. The in-column ground motion time histories are checked using the NEI method described in DC/COL-ISG-017. Four sets of statistically independent ground motion time histories are used as input control motion in three orthogonal directions for the FWSC site-specific seismic response analyses:

- One set of surface motion time histories compatible to the spectra defining the site-specific design motion at the bottom of the FWSC foundation at Elevation 282 ft NAVD88 (standard design Elevation 2.15 m), and
- Three sets of in-column motion time histories compatible to the spectra defining the site-specific design motion at the bottom of the concrete fill at nominal Elevation 220 ft NAVD88 (standard design Elevation -16.8 m) for analyses of BE, LB and UB subgrade profiles.

The envelope of results obtained from the FWSC analyses that use these two sets of input control motions meets the intent of DC/COL-ISG-017, which specifies that the spectra defining the design motion at the surface be enveloped by deterministic seismic response analyses at the top of the considered soil columns. The duration of time histories is 29.98 seconds and the time step is 0.005 seconds.

3A.13.2 Unit 3 Damping Values

The site-specific SSI analyses use structural models with full stiffness properties and OBE damping values to ensure conformance with guidance in SRP 3.7.2 and RG 1.61 for calculations of site-specific

in-structure response spectra (ISRS). The OBE damping values are also used for calculation of the site-specific SSE load demands on the RB/FB structures. Results obtained from the analyses of models with full (uncracked concrete) stiffness and SSE damping are used for calculations of the site-specific seismic load demands on the CB and FWSC reinforced concrete structures. The use of SSE damping for development of structural loads is adequate because the stresses obtained from models with SSE damping will remain lower than the stress limits considered by the applicable structural design codes. Responses from models with full (uncracked concrete) stiffness and SSE damping are also used for the foundation uplift and seismic stability evaluations because these analyses also consider limiting conditions that are associated with high dissipation of energy in the SSI dynamic system.

SSE damping is assigned to the models with reduced (cracked concrete) stiffness properties used for the sensitivity SSI analyses described in Section 3A.17.9 to represent the higher dissipation of energy in the structures when subjected to high stresses corresponding to the fully cracked concrete condition.

Table 3A.13.2-201 provides the damping values used for the site-specific SSI analyses. Section 3A.15 and Tables 3A.15-201 through 3A.15-206 provide details on the use of SSE damping values used in specific analysis cases.

NAPS DEP 3.7-1

Table 3A.13.2-201 **Damping Values for Dynamic Analysis**

	Percent of Critical Damping	
Components	**SSE Damping for Cracked Models**	**OBE Damping for Uncracked Models**
Reinforced concrete structures	7.0	4.0
Vent Wall/Diaphragm Floor		
• 0% Concrete Stiffness Contribution	4.0	
• 50% Concrete Stiffness Contribution	5.0	
• 100% Concrete Stiffness Contribution		3.0
Radiation Shield Wall	4.0	3.0
Reactor Pressure Vessel (RPV)	4.0	2.0
Separator/Chimney/Shroud	4.0	2.0
Fuel		
• Horizontal	6.0	6.0
• Vertical	6.0	4.0
Control Rod Drive Housing (CRDH)	2.0	1.0
RPV and Shroud Support	4.0	2.0
CRDH Support (CRD restraint)	4.0	2.0
Shroud Support	4.0	2.0

NAPS DEP 3.7-1

3A.14 Unit 3 Soil-Structure Interaction Analysis Method

Unit 3 site-specific SSI analyses are performed using the SASSI methodology used for the DCD seismic analysis of uniform and layered sites.

3A.14.1 DAC3N Analysis Method

The DAC3N analysis method is not used for site-specific SSI analyses.

3A.14.2 SASSI Analysis Method

The same SASSI methodology is used for the site-specific SSI analyses as the one presented in DCD Section 3A.5.2 to provide the solution for the seismic response of the structure-subgrade interaction system based on the frequency domain complex response method. The site-specific SASSI analyses use both the flexible volume (direct) method (DM) and the modified subtraction method (MSM) simplification. Benchmarking evaluations are used to demonstrate the accuracy of the MSM solutions relative to the corresponding solutions for the DM.

Finite elements with complex moduli are used for modeling the dynamic properties of the structures, foundations, near-field backfill materials and the excavated in-situ soil. Model details are described in Section 3A.16.

The site-specific SSI analyses are performed using a frequency step of 0.0244 Hz and a Fast Fourier Transformation (FFT) number of 8192. The computations are performed for a selected set of frequencies that are equal to or lower than the passing frequency of the analyzed SSI model, i.e., the highest frequency of seismic waves that can be transmitted through the SSI model. The seismic response of the SSI system at other frequencies is obtained by interpolation. Interpolated acceleration transfer function results are reviewed to identify numerical anomalies (e.g., sharp narrow spikes) that can potentially impact the accuracy of the frequency domain SSI analyses results. To ensure that the used set of frequencies of analysis provides sufficiently numerically accurate results, additional frequencies of analysis are used to demonstrate that the identified anomalies in the transfer function interpolation do not affect the accuracy of the results.

Analyses are performed separately for each one of the three directional components of input ground motion. The maximum co-directional seismic forces, moments, accelerations and displacements for each of the three ground motion time history components are combined using the SRSS

method. The co-directional soil/rock reactions are combined using the algebraic or absolute sum method in the time domain for sliding, soil bearing, and lateral soil pressure evaluations. The absolute sum method is a conservative alternative to performing algebraic sum for all possible combinations of the input directions.

The co-directional ISRS are combined using the SRSS method. As described in Section 3A.16, the ISRS are developed for responses at the edges of the buildings by taking into account coupling effects between vertical and rocking and between lateral and torsion motions. The site-specific SSI analyses use cut-off-frequencies that ensure the calculated site-specific design ISRS are adequate for design and qualification of components and equipment for frequencies up to 50 Hz, per the guidelines of NRC DC/COL-ISG-01.

3A.14.2.1 **SASSI Analysis Method for RB/FB**

The site-specific SSI analyses of the RB/FB are performed using the SASSI2010 computer program with the exception of the sensitivity analyses of the fully embedded RB/FB models with reduced stiffness properties described in Section 3A.17.9, which use the ACS SASSI computer program. A verification study demonstrates that the two programs provide virtually identical numerical results for the SSI response of the RB/FB at the Unit 3 site.

Analyses of the partially embedded RB/FB model use the flexible volume (direct) method (DM) where all nodes of the excavated volume are specified as interaction nodes to calculate the SSI system impedance matrix. Analyses of the large fully embedded RB/FB model use the modified subtraction method (MSM) simplification where the impedance calculations are performed for a selected set of nodes within the excavated volume.

3A.14.2.2 **SASSI Analysis Method for CB**

The site-specific SSI analyses of the CB are performed using the SASSI2010 computer program. The analyses of the partially embedded CB model use the flexible volume DM where all nodes of the excavated volume are specified as interaction nodes to calculate the SSI system impedance matrix. The analyses of the large fully embedded CB model use the MSM simplification where the impedance calculations are performed for a selected set of nodes within the excavated volume, and the two horizontal planes within the excavated volume.

3A.14.2.3 SASSI Analysis Method for FWSC

The site-specific SSI analyses of the FWSC half-models with symmetry and asymmetry boundary conditions are performed using the SASSI2010 computer program. However, unlike the SSI analysis performed for the standard design that considers the FWSC basemat resting on the surface of the supporting subgrade with infinite horizontal layering, the site-specific SSI analysis uses the structural model that explicitly includes the concrete fill below the FWSC foundation as a block embedded in the in-situ soil and rock. For the site-specific SSI analysis, a refined mesh is used for the excavated volume portion of the model to ensure it can pass high frequency waves. Because this results in a total number of interaction nodes that can exceed the program limitation for impedance calculations if the DM is used, the simplified MSM is employed instead where only selected nodes of the excavated volume are specified as interaction nodes. Specifically, interaction nodes are defined at the five exterior sides of the excavated volume (excluding the plane of symmetry), and two additional horizontal planes within the excavated volume.

3A.14.2.4 SASSI Analysis Method for Structure-Soil-Structure Interaction Analyses

The SSSI analyses of the CB-RB/FB, CB-FWSC, and FWSC-CB combined models are performed using the SASSI2010 computer program with the MSM where only selected nodes of the excavated volume are specified as interaction nodes. Section 3A.17.11 describes the site-specific evaluations for assessing the effects of SSSI on the seismic response of Seismic Category I structures at the Unit 3 site.

3A.15 Site-Specific Soil-Structure Interaction Analysis Cases

The site-specific SSI analyses cases for the RB/FB, CB, and FWSC are summarized below.

The RB/FB site-specific cases listed in Table 3A.15-201 are as follows:

- Design bases analyses Cases 1 through 6 use the SASSI2010 computer program and the structural models with UB stiffness properties.
- Sensitivity analyses Cases S1 through S12 use the SASSI2010 and ACS SASSI computer programs and the structural models with reduced stiffness properties to evaluate the effects of structural stiffness variations as described in Section 3A.17.9.

RB/FB acceleration transfer function results obtained from each analysis case are reviewed to ensure that selected frequencies of analysis provide sufficient numerically accurate results. Values of cut-off frequencies of analysis are used that are equal to or lower than the passing frequency of the SSI model (the highest frequency of seismic waves that can be transmitted through the SSI model). The selected values of cut-off-frequencies of analysis ensure that the SSI analyses provide RB/FB site-specific design ISRS that, per the guidelines of NRC DC/COL-ISG-01, are adequate for design and qualification of components and equipment for frequencies up to 50 Hz. The cut-off frequencies of analysis for the RB/FB are equal to or higher than 50 Hz for all analyses cases with the exception of the analysis of the fully embedded RB/FB model for the LB full column profile for which a cut-off frequency of 32 Hz is used.

Results of the RB/FB SSI analyses provided in Section 3A.17 show that the site-specific SSI analyses adequately capture the response of the RB/FB structures at high frequencies. The results show that the analyses of UB subgrade profiles govern the responses of the RB/FB at high frequencies. The analyses of the LB and BE profiles only provide bounding results for the ISRS envelopes at frequencies below 21 Hz. This is at least 10 Hz lower than the lowest cut-off frequency value of 32 Hz used for the analyses of the LB full column profile. The cut-off frequencies used for analyses of the UB partial and full column profiles that govern the design at high frequencies capture virtually all (approximately 99 percent) of the input motion energy. The other analyses cases use cut-off frequencies that enable capturing of at least 82 percent of the input motion energy.

The CB site-specific cases listed in Table 3A.15-202 are as follows:

- Design bases analyses Cases 1 through 6 are analyzed utilizing the structural models with full (uncracked concrete) stiffness properties and OBE damping to develop the Unit 3 site-specific seismic design basis ISRS
- Design bases analyses Cases 7 through 12 are analyzed utilizing the structural models with full (uncracked concrete) stiffness properties and SSE damping to develop the Unit 3 site-specific seismic design basis load demands on the CB structure and calculate base reactions for the CB stability evaluations

- Sensitivity Cases S1 through S6 are analyzed utilizing the CB structural models with reduced (cracked) stiffness properties and SSE damping to provide responses for the concrete cracking effects sensitivity evaluation described in Section 3A.17.9.

Acceleration transfer function results obtained from each analysis case are inspected to ensure that selected frequencies of analyses provide numerically accurate results. Values of cut-off frequencies of analyses are used that are equal to or lower than the passing frequency of the SSI model (the highest frequency of seismic waves that can be transmitted through the SSI model). The selected values of cut-off-frequencies of analysis ensure that the SSI analysis provide CB site-specific design ISRS that, per requirements of DC/COL-ISG-01, are adequate for design and qualification of components and equipment for frequencies up to 50 Hz. The cut-off frequencies of analyses are equal to or higher than 50 Hz for all analysis cases with the exception of the analysis of the LB full column profile for which cut-off frequency of 34 Hz is used.

Results of the CB SSI analyses presented in Section 3A.17 show that the analysis of the UB subgrade profiles govern the responses of the CB at high frequencies. The cut-off frequencies used for the analyses of the UB partial and full column profiles capture virtually all (approximately 99 percent) of the input motion energy. The other analysis cases use cut-off frequencies that enable capturing of at least 82 percent of the input motion energy. The UB partial column profile also provides bounding ISRS results for frequencies higher than 31 Hz. The LB and BE full column profiles can only affect the enveloping ISRS for the response at the CB basemat top at frequencies below 18 Hz, which is at least 16 Hz lower than the lowest value of 34 Hz used as cut-off frequency for the CB site-specific SSI analyses of the LB full column profile.

The FWSC site-specific cases listed in Table 3A.15-203 are as follows:

- Design bases Cases 1 through 6 use the FWSC structural model with full (uncracked concrete) stiffness properties and OBE damping to develop the site-specific seismic design basis ISRS and to calculate maximum displacements of the FWSC structures relative to free-field ground motion. Cases 1 through 3 are performed using the surface control motion input at the bottom of the FWSC basemat Elevation 282 ft NAVD88. Cases 4 through 6 are performed using the in-column control motions input at the bottom of the concrete fill Elevation 220 ft NAVD88.

- Cases 7 through 9 are analyzed utilizing the FWSC structural model with full (uncracked concrete) stiffness properties and SSE damping and in-column control motions input at the bottom of the concrete fill Elevation 220 ft NAVD88 to develop the Unit 3 site-specific seismic design basis load demands on the FWSC structures and to calculate base reactions for the FWSC stability and bearing pressure evaluations.
- Sensitivity Cases S1 through S4 are analyzed utilizing the FWSC structural model with reduced (cracked) stiffness properties and SSE damping supported on the LB and UB subgrade profiles to provide responses to the concrete cracking sensitivity evaluation described in Section 3A.17.9. Cases S1 through S4 are performed using in-column control motion inputs at the bottom of the FWSC basemat and at the bottom of the concrete fill, respectively.

FWSC acceleration transfer function results obtained from each analysis case are inspected to ensure that selected frequencies of analyses provide numerically accurate results. Values of cut-off frequencies of analyses are used that are equal to or lower than the passing frequency of the SSI model (the highest frequency of seismic waves that can be transmitted through the SSI model). The selected values of cut-off-frequencies of analysis ensure that the SSI analysis provides FWSC site-specific design ISRS that, per requirements of DC/COL-ISG-01, is adequate for the design and qualification of components and equipment for frequencies up to 50 Hz. The cut-off frequencies of analyses are equal to or higher than 50 Hz for all analysis cases with the exception of the analysis of the LB subgrade profile (analysis Cases 1, 4, 7, S1, and S2) for which a cut-off frequency of 36 Hz is used.

Results of the FWSC SSI analyses presented in Section 3A.17 show that the analyses of the UB subgrade profile govern the FWSC responses at high frequencies. The cut-off frequencies used for the analyses of the UB subgrade profiles capture virtually all (approximately 99 percent) of the input motion energy. The analyses of the BE and LB subgrade profiles use cut-off frequencies that enable the capture of at least 95 percent and 81 percent of the input motion energy, respectively. The comparisons in Section 3A.17 of the results obtained from the set of six analysis of the FWSC model with full stiffness properties and OBE damping for the different subgrade profiles show that the analyses of the UB and BE

profiles provide results for maximum accelerations, member forces, and base reactions that bound the results obtained from the analyses of the LB subgrade profile. The UB and BE profiles also provide bounding ISRS results for frequencies higher than 25 Hz, which is at least 10 Hz lower than the lowest value of 36 Hz used as the cut-off frequency for the FWSC site-specific SSI analyses of the LB subgrade profile.

The SSSI analysis cases for the RB/FB effects on the CB are listed in Table 3A.15-204. Section 3A.17.11 describes the site-specific SSSI analyses performed to address effects of structure to structure interaction of the RB/FB on the CB seismic response. Values of cut-off frequencies of analysis are equal to or close to the passing frequency of the CB-RB/FB model (the highest frequency of seismic waves that can be transmitted through the model). The SSSI analyses of the CB-RB/FB combined models that are performed for the three cases listed in Table 3A.15-204 use cut-off frequencies of analysis that are identical to those used for the corresponding SSI analyses to ensure that the energy content of the input motion captured by the SSSI analyses and the reference SSI analyses is the same and does not affect the SSSI evaluations.

Section 3A.17.11 describes the site-specific SSSI analyses performed to address effects of structure to structure interaction on the seismic response of the CB and FWSC at the Unit 3 site. Table 3A.15-205 presents the two analysis cases considered for the site-specific evaluation of the SSSI effects of the FWSC on the CB (CB-FWSC) seismic response. Table 3A.15-206 presents the nine analysis cases performed to include the SSSI effects of the CB on the FWSC (FWSC-CB) seismic response in the FWSC site-specific design basis.

The SSSI analyses of the CB-FWSC combined model that are performed for the two bounding subgrade stiffness conditions use cut-off frequencies of analysis that are identical to those used for the corresponding SSI analyses. This ensures that the energy content of the input motion captured by the SSSI analyses and the reference SSI analyses is the same and does not affect the SSSI evaluations.

The SSSI analyses of the FWSC-CB combined model are performed for the same set of input subgrade properties and ground motion time histories as those used for the SSI analyses of the FWSC stand-alone model. This ensures that the results of the FWSC-CB analyses that are used to develop the FWSC site-specific design basis completely capture

the effects of the subgrade properties variations. The cut-off frequency used for the FWSC-CB SSSI analysis of the UB subgrade profile is 70 Hz, which is identical to the cut-off frequency used for the corresponding SSI analyses of the FWSC stand-alone model for the UB soil profile. As shown in Table 3A.15-206, the analyses of the UB profiles capture virtually all (approximately 99 percent) of the input motion energy.

The FWSC-CB SSSI analyses of the BE and LB profiles use cut-off frequencies of 47 Hz and 30 Hz, respectively, and can capture at least 80 percent of the input motion energy. These cut-off frequencies are slightly lower than the cut-off frequencies of 36 Hz and 55 Hz used for the corresponding SSI analyses of the FWSC stand-alone model for the LB and BE soil profiles.

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Table 3A.15-201 **Site-Specific SSI Cases – RB/FB**

Case No.	Computer Program	Structural Model Properties*	Subgrade Profile		Method	Control Motion El.	Frequency (Hz) Passing	Frequency (Hz) Cut-off	Captured Motion Energy X	Captured Motion Energy Y	Captured Motion Energy Z
1	SASSI 2010	UC100	Partial Column	LB	DM	224 ft	62	62	99%	98%	96%
2				BE			83	70	100%	100%	100%
3				UB			112	70	99%	99%	100%
4			Full Column	LB	MSM		32	32	82%	82%	88%
5				BE			50	50	96%	96%	94%
6				UB			78	70	99%	99%	100%
Sensitivity Analyses for Evaluations of Stiffness Variation Effects											
S1	SASSI 2010	CR50	Partial Column	LB	DM	224 ft	62	62	99%	98%	96%
S2				BE			83	70	100%	100%	100%
S3				UB			112	70	99%	99%	100%
S4			Full Column	LB	MSM		32	32	82%	82%	88%
S5	ACS SASSI			BE			50	50	96%	96%	94%
S6				UB			78	70	99%	99%	100%
S7	SASSI 2010	CR00	Partial Column	LB	DM	224 ft	62	62	99%	98%	96%
S8				BE			83	70	100%	100%	100%
S9				UB			112	70	99%	99%	100%
S10			Full Column	LB	MSM		32	32	82%	82%	88%
S11	ACS SASSI			BE			50	50	96%	96%	94%
S12				UB			78	70	99%	99%	100%

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Table 3A.15-201 **Site-Specific SSI Cases – RB/FB** *(continued)*

*Structural Properties:

UC100 – Uncracked reinforced concrete and 100% in-fill concrete contribution to the stiffness of concrete-filled steel structures and OBE damping values

CR50 – Cracked reinforced concrete and 50% in-fill concrete contribution to the stiffness of concrete-filled steel structures and SSE damping values

CR00 – Cracked reinforced concrete and 0% in-fill concrete contribution to stiffness of concrete-filled steel structures and SSE damping values

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Table 3A.15-202 **Site-Specific SSI Analysis Cases – CB**

Case No.	Structural Model Properties*)	Subgrade Profile		Method	Control Motion El.	Frequency (Hz) Passing	Frequency (Hz) Cut-off	Captured Motion Energy X (NS)	Captured Motion Energy Y (EW)	Captured Motion Energy Z (Vert.)
1	UC_{OBE}	Partial Column	LB	DM	241 ft	50	50	97%	96%	99%
2	UC_{OBE}	Partial Column	BE	DM	241 ft	66	66	100%	99%	100%
3	UC_{OBE}	Partial Column	UB	DM	241 ft	86	70	99%	99%	99%
4	UC_{OBE}	Full Column	LB	MSM	241 ft	34	34	83%	82%	86%
5	UC_{OBE}	Full Column	BE	MSM	241 ft	51	50	96%	95%	92%
6	UC_{OBE}	Full Column	UB	MSM	241 ft	77	70	99%	99%	99%
7	UC_{SSE}	Partial Column	LB	DM	241 ft	50	50	97%	96%	99%
8	UC_{SSE}	Partial Column	BE	DM	241 ft	66	66	100%	99%	100%
9	UC_{SSE}	Partial Column	UB	DM	241 ft	86	70	99%	99%	99%
10	UC_{SSE}	Full Column	LB	MSM	241 ft	34	34	83%	82%	86%
11	UC_{SSE}	Full Column	BE	MSM	241 ft	51	50	96%	95%	92%
12	UC_{SSE}	Full Column	UB	MSM	241 ft	77	70	99%	99%	99%
Sensitivity Analyses for Evaluations of Stiffness Variation Effects										
S1	CR_{SSE}	Partial Column	LB	DM	241 ft	50	50	97%	96%	99%
S2	CR_{SSE}	Partial Column	BE	DM	241 ft	66	66	100%	99%	100%
S3	CR_{SSE}	Partial Column	UB	DM	241 ft	86	70	99%	99%	99%
S4	CR_{SSE}	Full Column	LB	MSM	241 ft	34	34	83%	82%	86%
S5	CR_{SSE}	Full Column	BE	MSM	241 ft	51	50	96%	95%	92%
S6	CR_{SSE}	Full Column	UB	MSM	241 ft	77	70	99%	99%	99%

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Table 3A.15-202 **Site-Specific SSI Analysis Cases – CB** *(continued)*

*) Structural Properties:

UC_{OBE} – Uncracked reinforced concrete and OBE damping values

UC_{SSE} – Uncracked reinforced concrete and SSE damping values

CR_{SSE} – Cracked reinforced concrete with 50% reduced shear and bending stiffness and SSE damping values

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Table 3A.15-203 **Site-Specific SSI Cases – FWSC**

Case No.	Structural Model Properties*)	Subgrade Profile	Method	Control Motion El.	Frequency (Hz) Passing	Frequency (Hz) Cut-off	Captured Motion Energy X (NS)	Captured Motion Energy Y (EW)	Captured Motion Energy Z (Vert.)
1	UC_{OBE}	LB	MSM	282 ft	36	36	89%	87%	83%
2		BE			55	55	97%	97%	95%
3		UB			84	70	100%	100%	99%
4		LB	MSM	220 ft	36	36	87%	81%	84%
5		BE			55	55	98%	96%	95%
6		UB			84	70	100%	99%	99%
7	UC_{SSE}	LB	MSM	220 ft	36	36	87%	81%	84%
8		BE			55	55	98%	96%	95%
9		UB			84	70	100%	99%	99%
Sensitivity Analyses for Evaluations of Stiffness Variation Effects									
S1	CR_{SSE}	LB	MSM	282 ft	36	36	89%	87%	83%
S2		BE	MSM	220 ft	36	36	87%	81%	84%
S3		UB	MSM	282 ft	84	70	100%	100%	99%
S4		LB	MSM	220 ft	84	70	100%	99%	99%

*) Structural Properties:
UC_{OBE} – Uncracked reinforced concrete and OBE damping values
UC_{SSE} – Uncracked reinforced concrete and SSE damping values
CR_{SSE} – Cracked reinforced concrete with 50% reduced shear and bending stiffness and SSE damping values

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Table 3A.15-204 **Site-Specific SSSI Cases – CB-RB/FB**

Case No.	Subgrade Profile		Method	Control Motion El.	Passing Freq. (Hz)	Cut-off Freq. (Hz)	Captured Motion Energy		
							X (NS)	Y (EW)	Z (Vert.)
CR1	Partial Column	LB	MSM	241 ft	55	50	97%	97%	99%
CR2	Partial Column	UB	MSM	241 ft	85	70	100%	100%	99%
CR3	Full Column	UB	MSM	241 ft	65	70*	96%	95%	98%

* The cut-off frequency is taken as 70 Hz in order to use the same cut-off frequency as the SSI analysis of the stand-alone model for comparison.

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Table 3A.15-205 **Site-Specific SSSI Cases – CB-FWSC**

Case No.	Subgrade Profile		Method	Control Motion El.	Passing Freq. (Hz)	Cut-off Freq. (Hz)	Captured Motion Energy X (NS)	Captured Motion Energy Y (EW)	Captured Motion Energy Z (Vert.)
CF1	Full Column	LB	MSM	241 ft	34	34	83%	82%	86%
CF2	Full Column	UB	MSM	241 ft	74	70	99%	99%	99%

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Table 3A.15-206 Site-Specific SSSI Cases – FWSC-CB

(a) FWSC-CB Combined Model with OBE Damping

Case No.	Subgrade Profile	Method	Control Motion El.	Passing Freq. (Hz)	Cut-off Freq. (Hz)	Captured Motion Energy X (NS)	Captured Motion Energy Y (EW)	Captured Motion Energy Z (Vert.)
FC1	LB	MSM	282 ft	30	30	84%	81%	75%
FC2	BE			47	47	98%	97%	95%
FC3	UB			72	70	99%	99%	99%
FC4	LB		220 ft	30	30	80%	72%	72%
FC5	BE			47	47	98%	96%	95%
FC6	UB			72	70	99%	99%	99%

(b) FWSC-CB Combined Model with SSE Damping

Case No.	Subgrade Profile	Method	Control Motion El.	Passing Freq. (Hz)	Cut-off Freq. (Hz)	Captured Motion Energy X (NS)	Captured Motion Energy Y (EW)	Captured Motion Energy Z (Vert.)
FC7	LB	MSM	220 ft	30	30	80%	72%	72%
FC8	BE			47	47	98%	96%	95%
FC9	UB			72	70	99%	99%	99%

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3A.16 Site-Specific SSI Analysis Models

Following the standard design methodology described in DCD Section 3A.7, models used for the site-specific seismic response analyses are based on three-dimensional lumped mass-beam models that consider shear, bending, torsion, and axial deformations. Single-Degree-of-Freedom (SDOF) oscillators are attached to lumped-mass beam models to capture local out-of-plane responses of slabs and walls for the flexible modes of vibration with frequencies up to 50 Hz.

The models used for the site-specific SSI analyses differ from the models used for the standard design SASSI analyses in that:

- The lower OBE damping value is assigned to the models to calculate ISRS
- The meshing of the below-grade portion of the models is modified to fit the layering and stiffness properties of the Unit 3 subgrade
- Rigid outriggers are installed at each floor elevation to facilitate calculation of ISRS and displacements at floor edges
- Near-field subgrade elements are included in the models to represent the fill materials

3A.16.1 Method of Dynamic Structural Model Development

The Unit 3 site-specific models are based on the models used for the DCD analyses, as described in DCD Section 3A.7, including the method of dynamic structural model development discussed in DCD Section 3A.7.1.

For the Unit 3 site-specific seismic demands, sensitivity studies are performed to capture the effect of concrete cracking on out-of-plane vibrations of flexible walls and slabs. Additional SDOF oscillators in the reduced stiffness models used for the RB/FB, CB, and FWSC structural stiffness variation effects sensitivity studies are used to capture local out-of-plane response of cracked slabs and walls. These SDOF oscillators are developed following the same methodology as the one used in the standard design.

3A.16.2 Lumped Mass-Beam Stick Model for SSI Analysis

As described further below, the site-specific models are based on the standard design lumped mass-beam stick models described in DCD Section 3A.7.2.

3A.16.3 SSI Model for SASSI2010 Analysis

The site-specific SSI structural models for the RB/FB, CB, and FWSC are constructed from the lumped mass-stick beam models coupled with the foundation finite element model following the methodology described in DCD Section 3A.7.3. The RB/FB, CB, and FWSC lumped mass-stick models (LMSMs) are shown in DCD Figures 3A.7-4, 3A.7-6, and 3A.7-7, respectively. Details of the models for the RB/FB, CB, and FWSC are described in the following sections.

3A.16.3.1 SSI Model for SASSI2010 Analysis – RB/FB

The model used for the site-specific SSI analysis of the RB/FB is based on the LMSMs shown in DCD Figure 3A.7-4 and designated in DCD Table 3A.6-1 as the base model including wall oscillators. This model was considered for RU-7 and RL-6 standard design SSI analyses.

Stand-alone RB/FB SSI models are developed for the site-specific RB/FB SSI analyses for the two embedment configurations as follows:

- Partially embedded RB/FB model used for analyses of BE, LB, and UB partial column subgrade profiles
- Fully embedded RB/FB model used for analyses of BE, LB, and UB full column profiles

A minimum value of 10.27 ft (3.13 m) is used for the lateral extent of the near-field concrete and structural fill elements at all four sides of the RB/FB model. This value represents the smaller of one-half of the distance between RB/FB and adjacent buildings and the gap between the building and the inside face of sheet piling at the sides without adjacent buildings, as described in Section 2.5.4.5.1.

The models used for RB/FB site-specific SSI analyses are shown in Figures 3A.16.3-201 through 3A.16.3-204b for the partially embedded model and in Figures 3A.16.3-205 through 3A.16.3-208b for the fully embedded model. The model axes are as described in Section 3A.11.

Exterior walls below grade and the foundation basemat are modeled using plate (flat shell) elements the same as the SASSI model used for

the standard design RB/FB SSI analyses. The basemat plate elements are shown in Figures 3A.16.3-201 and 3A.16.3-205.

The UB stiffness properties and OBE damping values provided in Table 3A.13.2-201 are assigned to the structural members in the RB/FB structural models representative of:

- Full (uncracked) stiffness of the reinforced concrete structures
- Full (100 percent) stiffness contribution of the in-fill concrete to the concrete-filled steel structures

The OBE damping values reflect the lower dissipation of energy in the structures experiencing lower stress levels associated with the uncracked concrete condition.

The use of UB stiffness properties and OBE damping values ensure conformance with SRP 3.7.2 and RG 1.61 for calculations of site-specific ISRS. The use of UB stiffness properties and OBE damping values provides conservative results for the site-specific SSE load demands on the RB/FB structures for the Unit 3 rock site and high frequency design motion. As described in Section 3A.17.9, sensitivity analyses are performed to evaluate the effects of structural stiffness variations on the site-specific SSI response of the Unit 3 RB/FB using the RB/FB structural models with reduced stiffness properties and SSE damping values. These models are representative of dynamic properties of RB/FB reinforced concrete structures under the fully cracked condition (i.e., the condition when all of the concrete structural members are cracked). Consistent with the approach used for the standard design to address the effects of in-fill concrete on the stiffness of the Vent Wall (VW) and Diaphragm Floor (D/F), the site-specific sensitivity analyses consider a 50 percent and 0 percent stiffness contribution of the in-fill concrete to the concrete-filled steel structures. The models used for the sensitivity study on the structural stiffness variation effects also have SDOF oscillators that represent the local out-of-plane vibrations of the RB/FB slabs and walls under cracked concrete conditions.

For the partially embedded model, the excavated volume is modeled from the top of the Zone III rock at Elevation -0.68 m (Elevation 273.0 ft NAVD88) to the bottom of the excavation at Elevation -15.5 m (Elevation 224.4 ft NAVD88). For the fully embedded model, the excavated volume is extended upward to the finished ground level grade at Elevation 4.5 m (Elevation 290.0 ft NAVD88). The mesh size of the

excavated volume elements is set to ensure conformance with SASSI2010 criteria that the maximum element size in all three directions does not exceed 20 percent of the shear wave length of the excavated soil at the highest (cut-off) frequency of analysis. The mesh of the near-field solid elements of the backfill surrounding the RB/FB is consistent with the mesh of the excavated volume and the mesh of the plate elements of the basemat and exterior walls.

In the partially embedded model, the excavated volume is modeled using a uniform and regular mesh. However, the top layers of the soft soil medium and the structural fill added to the fully embedded model between the Zone III rock and the finished ground level grade elevation require a more refined mesh in order to pass seismic waves with high frequencies. The fully embedded model also requires a coarser mesh of the rock and concrete fill below the Zone III rock level to ensure the overall model size does not exceed the program limitations. Therefore, in the transitional layers below the rock surface elevation, the fully embedded models use non-uniform mesh with triangular shell elements, prism, tetrahedral, and pyramid solid elements. An adequate value for the point-load radius used as input in the computation of the site impedance matrix is determined based on the results of a benchmarking study.

The passing and cut-off frequencies of analysis for the models used for the SSI analysis of the RB/FB are provided in Table 3A.15-201. The passing frequencies are calculated based on both the maximum horizontal and vertical dimensions of the excavated volume mesh and the near-field mesh. The model maximum passing frequencies for all subsurface profiles are no smaller than the cut-off frequency of analysis.

The maximum aspect ratios of the regular 3-D thin shell elements in the RB/FB partially and fully embedded models are 1:1.6 and 1:1.8, respectively. The maximum aspect ratios of the regular 3-D solid brick elements in the partially and fully embedded models are 1:1.6 and 1:3.5, respectively. Accuracy of the SASSI2010 program has been verified and validated for models with a maximum aspect ratio of 1:4 for both the 3-D thin shell and 3-D solid brick finite elements. Additionally, the accuracy of using non-uniform irregular elements is also demonstrated by results of a comparative study.

The site profiles used for the site-specific SSI analyses of the RB/FB (Tables 3A.16.3-201 through 3A.16.3-206) consist of 38 and 41 layers on

top of the half-space for the partial and full columns, respectively. The maximum value of Poisson's ratio of all materials in the RB/FB model is 0.48, which is within the range to which the accuracy of the SASSI2010 program has been verified and validated. The shear and compression wave velocities are adjusted using the equivalent wave travel time procedure as shown below.

$$Vs_{ave} = \frac{H}{\sum_i d_i / Vs_i} \quad Vp_{ave} = \frac{H}{\sum_i d_i / Vp_i}$$

where H is the thickness of the adjusted layer and d_i, V_{si} and V_{pi} are the thickness, shear wave and compression wave velocities of the layers in the original site profiles, respectively (see "Original data" in the note on Tables 3A.16.3.201 through 3A.16.3-206). The unit weight and damping ratios of the adjusted layers are determined as weighted averages with respect to the layer thickness.

The top of the half-space in the RB/FB models is established at Elevation -168.0 m (125 ft NAVD88). The half-space simulation consists of an additional ten layers with viscous dashpots added at the base of the site finite element model to account for the dissipation of energy at the model lower boundary. The total depth of the site model used for the SSI analyses of the RB/FB is more than 223 m, which exceeds two times the footprint dimension of the RB/FB. Results of a sensitivity study demonstrate that the depth of the lower boundary of the site model does not affect the results of SSI analysis.

As indicated in Figure 3A.16.3-209, the RB/FB stick model is connected to the side walls at floor Elevations -11.5 m, -6.4 m, -0.68 m, and 4.5 m by a set of rigid beams. At the base of the model at Elevation -15.5 m, a rigid link is used to connect all the stick models to the center of the basemat. Figure 3A.16.3-209 also shows the connection between the RB/FB stick model and foundation.

Figures 3A.16.3-202, 3A.16.3-206a, and 3A.16.3-206b show the 3-D spring elements established at the RB/FB exterior wall/backfill interfaces as shown in Figure 3A.16.3-202 for the partially embedded model and Figures 3A.16.3-206a and 3A.16.3-206b for the fully embedded model. Springs are also established at the bottom of the structural model to calculate reactions at the basemat interface with the underlying Zone III-IV rock. These spring elements are assigned global stiffness

properties high enough to ensure they do not affect the dynamic response of the analyzed SSI system, but not so high that they cause numerical sensitivity resulting from significant digit saturation truncation. The interface spring elements provide spring force results that serve as input for calculation of the site-specific wall lateral pressure and foundation bearing pressure demands. The spring force results also serve as input for calculation of seismic driving forces for the site-specific stability evaluations.

3A.16.3.2 SSI Model for SASSI2010 Analysis – CB

The model used for the site-specific SSI analysis of the CB is based on the three-dimensional LMSM shown in DCD Figure 3A.7-6 and designated in DCD Table 3A.6-1 as the base model.

Stand-alone CB SSI models are developed for the site-specific CB SSI analyses for the two embedment configurations as follows:

- Partially embedded CB model used for analyses of LB, BE and UB partial column subgrade profiles
- Fully embedded CB model used for analyses of LB, BE and UB full column profiles

A minimum value of 12.24 ft (3.73 m) is used for the lateral extent of near-field concrete and structural fill elements at all four sides of the CB model. This value is the smaller of one-half of the distance between CB and adjacent buildings and the actual width of the gap between the building and the inside face of sheet piling at the sides without adjacent buildings, as described in Section 2.5.4.5.1.

The models used for CB site-specific SSI analyses are shown in Figures 3A.16.3-210 through 3A.16.3-213b for the partially embedded model and in Figures 3A.16.3-214 through 3A.16.3-217b for the fully embedded model. The model axes are as described in Section 3A.11.

Exterior walls below grade and the foundation basemat are modeled using plate elements the same as the SASSI model used for standard design CB SSI analyses. However, because the soil medium between the top elevation of the finished ground level grade or Zone III rock and the foundation basemat are modeled in the Unit 3 site-specific SSI analysis, the vertical spacing of the wall nodes is adjusted to match the site-specific subsurface layers. The sizes of the plate elements are adjusted to satisfy the SASSI2010 requirement for the mesh size that

limits the size of the elements to not more than 20 percent of the length of the shear wave passing through the soil material. The basemat plate elements are shown in Figures 3A.16.3-210 and 3A.16.3-214.

The full (uncracked concrete) stiffness is assigned to the structural members in the CB structural model to ensure that the SSI analysis yields conservative results for Unit 3 rock site and high frequency design motion. In accordance with the RG 1.61 and SRP 3.7.2 guidance, the dynamic models used for the development of the CB site-specific ISRS are assigned OBE structural damping. The use of lower OBE structural damping values ensures that the ISRS peaks envelope the condition when the stresses and the dissipation of energy in the structures are low (RG 1.61, Section C.1.2).

The development of site-specific seismic structural load demands, foundation uplift, and stability evaluations are based on responses obtained from CB models with upper bound (uncracked concrete) stiffness properties in conjunction with SSE structural damping. Per RG 1.61, Section C.1.2, the use of SSE damping for development of structural loads is adequate because the stresses obtained from models with SSE damping will remain lower than the stress limits considered by the applicable structural design codes. The use of SSE damping for the foundation uplift and stability evaluations is adequate because these analyses also consider limiting conditions that are associated with high dissipation of energy in the SSI dynamic system.

Section 3A.17.9 discusses evaluation of the effects of concrete cracking on the SSI response of the CB that is based on the results of the sensitivity analyses of the CB dynamic models with reduced structural stiffness properties representing the fully cracked concrete condition, i.e., conditions where all of the concrete structural members are cracked. The models used for the sensitivity study on the concrete cracking effects also have SDOF oscillators that represent the local out-of-plane vibrations of the CB slabs under cracked concrete conditions.

For the partially embedded model, the excavated volume is modeled from the top of the Zone III rock at Elevation -3.12 m (Elevation 265.0 ft NAVD88) to the bottom of the excavation at Elevation -15.31 m (Elevation 225.0 ft NAVD88). For the FE model, the excavated volume is further extended upward to the finished ground level grade at Elevation 4.5 m (Elevation 290.0 ft NAVD88). The mesh size of the excavated volume elements is set to ensure that per SASSI2010 criteria,

the maximum element size in all three directions does not exceed 20 percent of the shear wave length of the excavated soil at the highest (cut-off) frequency of analysis. The mesh of the near-field soil solid elements of the backfill surrounding the CB is consistent with the mesh of the excavated volume and the mesh of the plate elements of the basemat and exterior walls.

In the partially embedded model, the excavated volume is modeled using a uniform and regular mesh. However, the top layers of soft soil medium and structural fill added to the fully embedded model between the Zone III rock and the finished ground level grade require a refined mesh in order to capture sufficient input motion energy. The fully embedded model also requires a courser mesh of the rock and concrete fill below the Zone III rock level to ensure the overall model size does not exceed the program limitations. Therefore, in the transitional layers below the rock surface elevation, the fully embedded models use non-uniform mesh with irregular triangular shell elements, prism, tetrahedral and pyramid solid elements. A benchmarking study determines an adequate value for the point radius used as input in the SASSI2010 computation of the site impedance matrix.

The passing and cut-off frequencies of analysis for the models used for SSI analysis of CB are shown in Table 3A.15-202. The passing frequencies are calculated on the basis of both the maximum horizontal and vertical dimensions of the excavated volume mesh and near-field mesh. The table shows that the model maximum passing frequencies for all subsurface profiles are no smaller than the cut-off frequency of analysis. The last three columns of Table 3A.15-202 show the percentage of input motion energy captured by the CB SSI analysis that are obtained as the values at the cut-off frequencies of analyses of the input motion cumulative power densities. The table shows that the SSI analyses of UB subgrade profiles that govern the response of the building at high frequency capture 99 percent of the input motion energy. The analyses of LB and BE profiles capture at least 82 percent and 92 percent of the input motion energy, respectively.

The maximum aspect ratios of the regular 3-D thin shell elements in the CB partially and fully embedded models are 1:1.7 and 1:1.6, respectively. The maximum aspect ratios of the regular 3-D solid brick elements in the partially and fully embedded models are 1:1.5 and 1:1.9, respectively. The accuracy of SASSI2010 program has been verified and validated for

models with a maximum aspect ratio of 1:4 for both the 3-D thin shell and 3-D solid brick finite elements. Additionally, the accuracy of using non-uniform irregular elements is also demonstrated by the results of a comparative study.

Site profiles used for the site-specific SSI analysis of the CB are presented in Tables 3A.16.3-207 through 3A.16.3-209 for the partial column profiles, and in Tables 3A.16.3-210 through 3A.16.3-212 for the full column profiles. They consist of 17 and 28 layers on top of half space for the partial and full columns, respectively. The maximum value of Poisson's ratio of all materials in the CB model is 0.48, which is within the range to which the accuracy of SASSI2010 program has been verified and validated. Layering and strain compatible properties of site profiles that were developed from the results of the site response analysis are adjusted using the procedure described in Section 3A.16.3.1.

The top of the half-space in the CB models is established at DCD Elevation -90.8 m (125 ft NAVD88). Consistent with SASSI manual recommendations, the half-space simulation consists of an additional ten layers with viscous dashpots added at the base of the site finite element model to account for the dissipation of energy at the model lower boundary. The total depth of the site model used for the SSI analyses of CB is more than 88 m, which exceeds two times the CB maximum footprint dimension of 30.3 m. Results of a sensitivity study demonstrate that the depth of the lower boundary of the site model used for the site-specific SSI analyses does not affect the results of the analysis.

As indicated in Figure 3A.16.3-218, the CB stick model is connected to the side walls at floor Elevation -10.4 m, -7.4 m, -2.0 m, and 4.5 m by a set of rigid beams. At the base of the model at Elevation -7.4 m, a rigid beam is used to connect the stick model to the center of the basemat.

Three-dimensional (3-D) spring elements are established at the CB exterior wall/backfill interfaces and concrete fill under the CB basemat as shown in Figures 3A.16.3-211, and 3A.16.3-215a and 3A.16.3-215b for the partially embedded and fully embedded models, respectively. Springs are also established at the bottom of the CB structural model to calculate reactions at the concrete fill interface with the underlying Zone III/IV rock. These spring elements are assigned global stiffness properties high enough to ensure they do not affect the dynamic response of the analyzed SSI system. The interface spring elements provide spring force results that serve as input for calculation of the site-specific wall lateral

pressure and foundation bearing pressure demands. The spring forces results also serve as input for calculation of seismic driving forces for the site-specific stability evaluations.

3A.16.3.3 SSI Model for SASSI2010 Analysis – FWSC

The model used for the site-specific SSI analysis of the FWSC is a half model with symmetry (asymmetry) boundary conditions based on the three-dimensional LMSM shown in DCD Figure 3A.7-7 and is designated in DCD Table 3A.6-1 as the base model. The stiffness of the structural elements of the FWSC is represented by a single set of stick elements that consider the vertical and horizontal eccentricity.

In addition to the differences in the SSI models identified in Section 3A.16, the site-specific FWSC SSI model also differs from the standard design base model in that a block of near-field solid elements embedded in the in-situ soil and rock is used to model the concrete fill placed below the FWSC basemat

The FWSC SSI analyses use the FWSC (UC_{OBE}) model with full (uncracked concrete) stiffness properties and OBE damping values for the reinforced concrete structures. Additional cases are performed using full (uncracked) concrete stiffness properties and SSE damping values for the development of site-specific load demands on the FWSC structures and the base reaction time histories for the foundation uplift and stability evaluations.

The foundation basemat is modeled using plate elements in the same manner as the DCD SASSI analysis model. The size of the plate element does not exceed 20 percent of the length of the shear wave passing through the soil material.

Full (uncracked concrete) stiffness properties are assigned to the FWSC structural model, which yields conservative results for the Unit 3 rock site and high frequency design motion. In accordance with RG 1.61 and SRP 3.7.2 guidance, the dynamic UC_{OBE} model used for the development of the FWSC site-specific ISRS is assigned OBE structural damping. The use of lower OBE structural damping values ensures that the ISRS peaks envelope the condition when the stresses and dissipation of energy in the structures are low (RG 1.61, Section C.1.2).

The development of site-specific seismic structural load demands, foundation uplift, and stability evaluations are based on responses obtained from the FWSC UC_{SSE} model with upper bound (uncracked

concrete) stiffness properties in conjunction with SSE structural damping. Per RG 1.61, Section C.1.2, the use of SSE damping for the development of structural loads is adequate because the stresses obtained from models with SSE damping will remain lower than the stress limits considered by the applicable structural design codes. The use of SSE damping for the foundation uplift and stability evaluations is adequate because these analyses generate foundation reaction forces and moments at the bottom of basemat that are consistent with the seismic structural load demands generated from the FWSC UC_{SSE} model.

Evaluation of the effects of concrete cracking on the SSI response of the Unit 3 FWSC is based on the results of the sensitivity analyses of the FWSC CR_{SSE} dynamic model with reduced structural stiffness properties representing the fully cracked concrete condition, i.e., conditions where all of the concrete structural members are cracked. The CR_{SSE} model used for the sensitivity study on the concrete cracking effects also has SDOF oscillators that represent the local out-of-plane vibrations of the FWS roof under cracked concrete conditions.

The models used for FWSC site-specific SSI analyses are shown on Figures 3A.16.3-219 through 3A.16.3-221b. The model axes are as described in Section 3A.11.

The structural model also includes the concrete fill block resting on top of the Zone III-IV rock that supports the FWSC foundation. The concrete fill that is embedded in the in-situ saprolite and Zone III rock is modeled using brick solid elements. As shown in Figures 3A.16.3-219 through 3A.16.3-221b, the concrete fill block, as well as the excavated volume, are modeled from the top of the Zone III-IV rock at Elevation -16.84 m (Elevation 220.0 ft NAVD88) to the bottom of the basemat foundation at Elevation 2.15 m (Elevation 282.3 ft NAVD88). The size of the elements in all directions for the excavated volume is determined, in accordance with SASSI2010 requirements, not to exceed 20 percent of the length of the shear wave passing through the soil material in order to capture sufficient input motion energy. The passing and cut-off frequencies of analysis are shown in Table 3A.15-203. The passing frequencies are calculated on the basis of both the maximum horizontal and vertical dimensions of the excavated volume and backfill mesh. The table shows that the model maximum passing frequencies for

all subsurface profiles are no smaller than the cut-off frequency of analysis.

The excavated volume in the FWSC Unit 3 site-specific SSI model has a uniform mesh. The maximum aspect ratio of the 3-D thin shell elements for the basemat is 1:1.4. The maximum aspect ratio of the 3-D solid brick elements is 1:2.9. Accuracy of the SASSI2010 program is verified and validated for models with a maximum aspect ratio of 1:4 for both the 3-D thin shell and 3-D solid brick finite elements.

The soil properties used for the site-specific FWSC SSI analysis shown in Tables 3A.16.3-213 through 3A.16.3-215 consist of 50 layers on top of the half-space. The maximum value of soil Poisson's ratio considered in the site models is 0.48, which is within the range of accuracy that the SASSI2010 program has been verified and validated. The shear and compression wave velocities, unit weights, and damping ratios are not adjusted from the original strain iterated soil profiles. Instead, layering of the profiles of strain-compatible properties that are developed from the results of the site response analysis are adjusted and some layers are divided so that the site models used for site-specific SSI analyses can meet passing frequency requirements.

The top of the half-space in the FWSC models is established at the DCD Elevation -120.8 m (126 ft NAVD88). Consistent with SASSI manual recommendations, the half-space simulation consists of an additional ten layers with viscous dashpots added at the base of the site finite element model to account for the dissipation of energy at the model lower boundary. The total depth of the site model used for SSI analyses of FWSC is more than 123 m, which exceeds two times the maximum footprint dimension of the FWSC basemat of 52 m. Results of a sensitivity study demonstrate that the depth of the lower boundary of the site model used for the site-specific SSI analyses does not affect the results of the analysis.

The FWSC stick models are connected to the basemat at Elevation 2.15 m (282.3 ft NAVD88) by a set of rigid beams without mass along the footprint of walls of the FWS and the FPE.

The 3-D spring elements are established at the interface between the concrete fill solid elements and the FWSC basemat shell elements as well as the interface between the concrete fill elements and the surrounding in-situ soil. These spring elements are assigned global

stiffness properties high enough to ensure that they do not affect the dynamic properties of the analyzed SSI system. The spring elements at the FWSC basemat and concrete fill interface provide spring force results that serve as input for the calculation of base contact pressures and foundation bearing pressure demands. The spring forces results also serve as input for the calculation of seismic driving forces for the site-specific stability evaluations.

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Table 3A.16.3-201 **Subsurface Properties for SSI Analysis of RB/FB PE (BE profile)**

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-0.68 / -3.54	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-3.54 / -6.40	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-6.40 / -8.95	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-8.95 / -11.50	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-11.50 / -13.50	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-13.50 / -15.50	2.32	1318	3229	0.58	82.8	0.400	2.32	2134	3325	1.00	134.2	0.150
-15.50 / -18.45	2.60	1644	4025	0.98	111.4	0.400	…← See Note					
-18.45 / -21.41	2.61	1661	4068	1.00	112.2	0.400						
-21.41 / -24.61	2.61	1578	3865	1.00	98.6	0.400						
-24.61 / -27.81	2.61	1578	3865	1.00	98.6	0.400						

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Table 3A.16.3-201 **Subsurface Properties for SSI Analysis of RB/FB PE (BE profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-27.81 -32.78	2.63	2682	4779	1.00	107.9	0.270						
-32.78 -37.76	2.63	2682	4779	1.00	107.7	0.270						
-37.76 -42.74	2.63	2682	4779	1.00	107.7	0.270						
-42.74 -	2.63	2804	4996	1.00		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

-15.50 -15.62	2.32	1318	3229	0.58		0.400						
-15.62 -18.45	2.61	1661	4068	1.00		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.

Example: 1644=(18.45-15.5)/(0.12/1318+2.83/1661)

Damping ratio is determined using the thickness weighted average procedure.

Example: 0.98=(0.58x(15.62-15.50)+1.00x(18.45-15.62))/(0.12+2.83)

NAPS DEP 3.7-1

Table 3A.16.3-202 **Subsurface Properties for SSI Analysis of RB/FB PE (LB profile)**

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-0.68 / -3.54	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-3.54 / -6.40	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-6.40 / -8.95	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-8.95 / -11.50	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-11.50 / -13.50	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-13.50 / -15.50	2.32	979	2398	1.02	61.5	0.400	2.32	1829	2850	1.80	115.0	0.150
-15.50 / -18.45	2.60	1217	2983	1.79	82.5	0.400	…← See Note					
-18.45 / -21.41	2.61	1230	3014	1.82	83.1	0.400						
-21.41 / -24.61	2.61	1058	2591	1.82	66.1	0.400						
-24.61 / -27.81	2.61	1058	2591	1.82	66.1	0.400						

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Table 3A.16.3-202 **Subsurface Properties for SSI Analysis of RB/FB PE (LB profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-27.81 / -32.78	2.63	2190	3902	1.82	88.1	0.270						
-32.78 / -37.76	2.63	2190	3902	1.82	87.9	0.270						
-37.76 / -42.74	2.63	2190	3902	1.82	87.9	0.270						
-42.74 / -	2.63	2290	4079	1.82		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

-15.50 / -15.62	2.32	979	2398	1.02		0.400						
-15.62 / -18.45	2.61	1230	3014	1.82		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.
Example: 1217=(18.45-15.5)/(0.12/979+2.83/1230)
Damping ratio is determined using the thickness weighted average procedure.
Example: 1.79=(1.02x(15.62-15.50)+1.82x(18.45-15.62))/(0.12+2.83)

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Table 3A.16.3-203 **Subsurface Properties for SSI Analysis of RB/FB PE (UB profile)**

EL (m)	Soil Unit Weight (t/m^3)	Soil Vs (m/sec)	Soil Vp (m/sec)	Soil Damping (%)	Soil Highest Frequency (Hz)	Soil Poisson's ratio	Backfill Unit Weight (t/m^3)	Backfill Vs (m/sec)	Backfill Vp (m/sec)	Backfill Damping (%)	Backfill Highest Frequency (Hz)	Backfill Poisson's ratio
-0.68 / -3.54	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-3.54 / -6.40	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-6.40 / -8.95	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-8.95 / -11.50	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-11.50 / -13.50	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-13.50 / -15.50	2.32	1774	4346	0.33	111.5	0.400	2.32	2438	3800	0.55	153.3	0.150
-15.50 / -18.45	2.60	2218	5434	0.54	150.3	0.400	…← See Note					
-18.45 / -21.41	2.61	2242	5492	0.55	151.4	0.400						
-21.41 / -24.61	2.61	2354	5767	0.55	147.1	0.400						
-24.61 / -27.81	2.61	2354	5767	0.55	147.1	0.400						

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Table 3A.16.3-203 **Subsurface Properties for SSI Analysis of RB/FB PE (UB profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-27.81 / -32.78	2.63	3285	5852	0.55	132.1	0.270						
-32.78 / -37.76	2.63	3285	5852	0.55	131.9	0.270						
-37.76 / -42.74	2.63	3285	5852	0.55	131.9	0.270						
-42.74 / -	2.63	3434	6119	0.55		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.50 / -15.62	2.32	1774	4346	0.33		0.400						
-15.62 / -18.45	2.61	2242	5492	0.55		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.
Example: 2218=(18.45-15.5)/(0.12/1774+2.83/2242)
Damping ratio is determined using the thickness weighted average procedure.
Example: 0.54=(0.33x(15.62-15.50)+0.55x(18.45-15.62))/(0.12+2.83)

NAPS DEP 3.7-1

Table 3A.16.3-204 **Subsurface Properties for SSI Analysis of RB/FB FE (BE profile)**

EL (m)	Soil Unit Weight (t/m3)	Soil Vs (m/sec)	Soil Vp (m/sec)	Soil Damping (%)	Soil Highest Frequency (Hz)	Soil Poisson's ratio	Backfill Unit Weight (t/m3)	Backfill Vs (m/sec)	Backfill Vp (m/sec)	Backfill Damping (%)	Backfill Highest Frequency (Hz)	Backfill Poisson's ratio
4.50 3.89	2.00	277	678	2.07	56.9	0.400	2.08	224	418	2.18	56.9	0.299
3.89 3.13	2.00	267	761	3.38	54.9	0.430	2.08	198	370	4.24	50.3	0.299
3.13 2.37	2.00	267	761	3.38	54.9	0.430	2.08	198	370	4.24	50.3	0.299
2.37 1.61	2.08	397	1668	2.89	81.6	0.470	2.08	216	1103	5.13	54.9	0.480
1.61 0.84	2.08	397	1668	2.89	81.6	0.470	2.08	216	1103	5.13	54.9	0.480
0.84 0.08	2.08	575	1549	2.50	118.2	0.420	2.08	224	1144	5.80	56.9	0.480
0.08 -0.68	2.08	575	1549	2.50	118.2	0.420	2.08	224	1144	5.80	56.9	0.480
-0.68 -2.00	2.32	1318	3229	0.58	135.5	0.400	2.32	2134	3325	1.00	272.7	0.150
-2.00 -4.50	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150
-4.50 -6.40	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150

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Table 3A.16.3-204 **Subsurface Properties for SSI Analysis of RB/FB FE (BE profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-6.40 -8.40	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150
-8.40 -10.40	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150
-10.40 -11.50	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150
-11.50 -13.50	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	01.50
-13.50 -15.50	2.32	1318	3229	0.58	67.7	0.400	2.32	2134	3325	1.00	136.3	0.150
-15.50 -18.45	2.60	1644	4025	0.98	84.5	0.400	…← See Note					
-18.45 -21.41	2.61	1661	4068	1.00	85.4	0.400						
-21.41 -23.41	2.61	1578	3865	1.00	81.1	0.400						
-23.41 -25.61	2.61	1578	3865	1.00	81.1	0.400						
-25.61 -27.81	2.61	1578	3865	1.00	81.1	0.400						

NAPS DEP 3.7-1

Table 3A.16.3-204 **Subsurface Properties for SSI Analysis of RB/FB FE (BE profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-27.81 / -32.78	2.63	2682	4779	1.00	107.9	0.270						
-32.78 / -37.76	2.63	2682	4779	1.00	107.7	0.270						
-37.76 / -42.74	2.63	2682	4779	1.00	107.7	0.270						
-42.74 / -	2.63	2804	4996	1.00		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

-15.50 / -15.62	2.32	1318	3229	0.58		0.400						
-15.62 / -18.45	2.61	1661	4068	1.00		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.
Example: 1644=(18.45-15.5)/(0.12/1318+2.83/1661)
Damping ratio is determined using the thickness weighted average procedure.
Example: 0.98=(0.58x(15.62-15.50)+1.00x(18.45-15.62))/(0.12+2.83)

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Table 3A.16.3-205 **Subsurface Properties for SSI Analysis of RB/FB FE (LB profile)**

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
4.50 / 3.89	2.00	188	460	3.50	38.6	0.400	2.08	162	303	3.12	41.1	0.300
3.89 / 3.13	2.00	163	464	7.02	33.5	0.430	2.08	127	238	7.00	32.2	0.301
3.13 / 2.37	2.00	163	464	7.02	33.5	0.430	2.08	127	238	7.00	32.2	0.301
2.37 / 1.61	2.08	248	1266	5.52	51.0	0.480	2.08	135	689	8.45	34.3	0.480
1.61 / 0.84	2.08	248	1266	5.52	51.0	0.480	2.08	135	689	8.45	34.3	0.480
0.84 / 0.08	2.08	384	1463	4.40	78.9	0.463	2.08	143	729	9.40	36.3	0.480
0.08 / -0.68	2.08	384	1463	4.40	78.9	0.463	2.08	143	729	9.40	36.3	0.480
-0.68 / -2.00	2.32	979	2398	1.02	100.6	0.400	2.32	1829	2850	1.80	233.7	0.150
-2.00 / -4.50	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-4.50 / -6.40	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150

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Table 3A.16.3-205 **Subsurface Properties for SSI Analysis of RB/FB FE (LB profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-6.40 / -8.40	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-8.40 / -10.40	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-10.40 / -11.50	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-11.50 / -13.50	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-13.50 / -15.50	2.32	979	2398	1.02	50.3	0.400	2.32	1829	2850	1.80	116.8	0.150
-15.50 / -18.45	2.60	1217	2983	1.79	62.6	0.400	…← See Note					
-18.45 / -21.41	2.61	1230	3014	1.82	63.2	0.400						
-21.41 / -23.41	2.61	1058	2591	1.82	54.4	0.400						
-23.41 / -25.61	2.61	1058	2591	1.82	54.4	0.400						
-25.61 / -27.81	2.61	1058	2591	1.82	54.4	0.400						

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Table 3A.16.3-205 **Subsurface Properties for SSI Analysis of RB/FB FE (LB profile)** *(continued)*

EL (m)	Soil: Unit Weight (t/m3)	Soil: Vs (m/sec)	Soil: Vp (m/sec)	Soil: Damping (%)	Soil: Highest Frequency (Hz)	Soil: Poisson's ratio	Backfill: Unit Weight (t/m3)	Backfill: Vs (m/sec)	Backfill: Vp (m/sec)	Backfill: Damping (%)	Backfill: Highest Frequency (Hz)	Backfill: Poisson's ratio
-27.81 / -32.78	2.63	2190	3902	1.82	88.1	0.270						
-32.78 / -37.76	2.63	2190	3902	1.82	87.9	0.270						
-37.76 / -42.74	2.63	2190	3902	1.82	87.9	0.270						
-42.74 / -	2.63	2190	4079	1.82		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio						
-15.50 / -15.62	2.32	979	2398	1.02		0.400						
-15.62 / -18.45	2.61	1230	3014	1.82		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.
Example: 1217=(18.45-15.5)/(0.12/979+2.83/1230)
Damping ratio is determined using the thickness weighted average procedure.
Example: 1.79=(1.02x(15.62-15.50)+1.82x(18.45-15.62))/(0.12+2.83)

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Table 3A.16.3-206 **Subsurface Properties for SSI Analysis of RB/FB FE (UB profile)**

	Soil						Backfill					
EL (m)	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
4.50 3.89	2.00	408	999	1.22	83.9	0.400	2.08	309	578	1.52	78.5	0.300
3.89 3.13	2.00	437	1248	1.63	89.8	0.430	2.08	307	574	2.57	78.0	0.300
3.13 2.37	2.00	437	1248	1.63	89.8	0.430	2.08	307	574	2.57	78.0	0.300
2.37 1.61	2.08	634	2665	1.52	130.4	0.470	2.08	346	1436	3.11	87.9	0.470
1.61 0.84	2.08	634	2665	1.52	130.4	0.470	2.08	346	1436	3.11	87.9	0.470
0.84 0.08	2.08	862	2321	1.42	177.3	0.420	2.08	352	1463	3.58	89.5	0.469
0.08 -0.68	2.08	862	2321	1.42	177.3	0.420	2.08	352	1463	3.58	89.5	0.469
-0.68 -2.00	2.32	1774	4346	0.33	182.4	0.400	2.32	2438	3800	0.55	311.5	0.150
-2.00 -4.50	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-4.50 -6.40	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150

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Table 3A.16.3-206 **Subsurface Properties for SSI Analysis of RB/FB FE (UB profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-6.40 / -8.40	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-8.40 / -10.40	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-10.40 / -11.50	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-11.50 / -13.50	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-13.50 / -15.50	2.32	1774	4346	0.33	91.2	0.400	2.32	2438	3800	0.55	155.7	0.150
-15.50 / -18.45	2.60	2218	5434	0.54	114.0	0.400	...← See Note					
-18.45 / -21.41	2.61	2242	5492	0.55	115.2	0.400						
-21.41 / -23.41	2.61	2354	5767	0.55	121.0	0.400						
-23.41 / -25.61	2.61	2354	5767	0.55	121.0	0.400						
-25.61 / -27.81	2.61	2354	5767	0.55	121.0	0.400						

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Table 3A.16.3-206 **Subsurface Properties for SSI Analysis of RB/FB FE (UB profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-27.81 -32.78	2.63	3285	5852	0.55	132.1	0.270						
-32.78 -37.76	2.63	3285	5852	0.55	131.9	0.270						
-37.76 -42.74	2.63	3285	5852	0.55	131.9	0.270						
-42.74 -	2.63	3434	6119	0.55		0.270						

Note: The soil properties of the adjusted layer, shown in the red box, are evaluated from the original properties shown below.

Original data

-15.50 -15.62	2.32	1774	4346	0.33		0.400						
-15.62 -18.45	2.61	2242	5492	0.55		0.400						

Vs and Vp are determined using the equivalent wave travel time procedure.
Example: 2218=(18.45-15.5)/(0.12/1774+2.83/2242)
Damping ratio is determined using the thickness weighted average procedure.
Example: 0.54=(0.33x(15.62-15.50)+0.55x(18.45-15.62))/(0.12+2.83)

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Table 3A.16.3-207 **Subsurface Properties for SSI Analysis of CB PE (BE Profile)**

	Soil						**Backfill**					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-3.120 -4.645	2.32	617	1885	0.62	72.5	0.440	2.32	2134	3325	1.00	251.0	0.150
-4.645 -6.170	2.32	617	1885	0.62	72.5	0.440	2.32	2134	3325	1.00	251.0	0.150
-6.170 -7.400	2.32	754	2305	0.63	88.7	0.440	2.32	2134	3325	1.00	251.0	0.150
-7.400 -8.900	2.32	754	2305	0.63	88.7	0.440	2.32	2134	3325	1.00	251.0	0.150
-8.900 -10.400	2.32	754	2305	0.63	88.7	0.440	2.32	2134	3325	1.00	251.0	0.150
-10.400 -12.260	2.32	754	2305	0.63	81.0	0.440	2.32	2134	3325	1.00	229.4	0.150
-12.260 -13.785	2.32	811	2477	0.53	95.4	0.440	2.32	2134	3325	1.00	251.0	0.150
-13.785 -15.310	2.32	811	2477	0.53	95.4	0.440	2.32	2134	3325	1.00	251.0	0.150

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Table 3A.16.3-207 **Subsurface Properties for SSI Analysis of CB PE (BE Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.310 -18.360	2.61	1976	4225	1.00	129.5	0.360						
-18.360 -24.460	2.61	2128	4273	1.00	69.7	0.335						
-24.460 -27.500	2.61	2421	4613	1.00	159.2	0.310						
-27.500 -30.550	2.63	2638	4772	1.00	172.9	0.280						
-30.550 -33.600	2.63	2512	4787	1.00	164.7	0.310						
-33.600 -36.650	2.63	2639	4937	1.00	173.0	0.300						
-36.650 -39.700	2.63	2689	4722	1.00	176.3	0.260						
-39.700 -42.740	2.63	2847	5150	1.00	187.3	0.280						
-42.740 -45.790	2.63	2804	5245	1.00	183.8	0.300						
-45.790 -	2.63	2804	4794	1.00		0.240						

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Table 3A.16.3-208 **Subsurface Properties for SSI Analysis of CB PE (LB Profile)**

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-3.120 -4.645	2.32	471	1463	1.06	55.4	0.442	2.32	1829	2850	1.80	215.1	0.150
-4.645 -6.170	2.32	471	1463	1.06	55.4	0.442	2.32	1829	2850	1.80	215.1	0.150
-6.170 -7.400	2.32	561	1715	1.14	66.0	0.440	2.32	1829	2850	1.80	215.1	0.150
-7.400 -8.900	2.32	561	1715	1.14	66.0	0.440	2.32	1829	2850	1.80	215.1	0.150
-8.900 -10.400	2.32	561	1715	1.14	66.0	0.440	2.32	1829	2850	1.80	215.1	0.150
-10.400 -12.260	2.32	561	1715	1.14	60.3	0.440	2.32	1829	2850	1.80	196.6	0.150
-12.260 -13.785	2.32	662	2022	0.96	77.8	0.440	2.32	1829	2850	1.80	215.1	0.150
-13.785 -15.310	2.32	662	2022	0.96	77.8	0.440	2.32	1829	2850	1.80	215.1	0.150

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Table 3A.16.3-208 **Subsurface Properties for SSI Analysis of CB PE (LB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.310 -18.360	2.61	1613	3450	1.82	105.7	0.360						
-18.360 -24.460	2.61	1738	3488	1.82	56.9	0.335						
-24.460 -27.500	2.61	1977	3767	1.82	130.0	0.310						
-27.500 -30.550	2.63	2154	3897	1.82	141.2	0.280						
-30.550 -33.600	2.63	2051	3909	1.82	134.4	0.310						
-33.600 -36.650	2.63	2155	4031	1.82	141.3	0.300						
-36.650 -39.700	2.63	2195	3855	1.82	143.9	0.260						
-39.700 -42.740	2.63	2324	4205	1.82	152.8	0.280						
-42.740 -45.790	2.63	2289	4282	1.82	150.0	0.300						
-45.790 -	2.63	2290	3915	1.82		0.240						

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Table 3A.16.3-209 **Subsurface Properties for SSI Analysis of CB PE (UB Profile)**

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-3.120 -4.645	2.32	808	2469	0.36	95.0	0.440	2.32	2438	3800	0.55	286.8	0.150
-4.645 -6.170	2.32	808	2469	0.36	95.0	0.440	2.32	2438	3800	0.55	286.8	0.150
-6.170 -7.400	2.32	1014	3098	0.35	119.2	0.440	2.32	2438	3800	0.55	286.8	0.150
-7.400 -8.900	2.32	1014	3098	0.35	119.2	0.440	2.32	2438	3800	0.55	286.8	0.150
-8.900 -10.400	2.32	1014	3098	0.35	119.2	0.440	2.32	2438	3800	0.55	286.8	0.150
-10.400 -12.260	2.32	1014	3098	0.35	109.0	0.440	2.32	2438	3800	0.55	262.1	0.150
-12.260 -13.785	2.32	993	3034	0.29	116.8	0.440	2.32	2438	3800	0.55	286.8	0.150
-13.785 -15.310	2.32	993	3034	0.29	116.8	0.440	2.32	2438	3800	0.55	286.8	0.150

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Table 3A.16.3-209 **Subsurface Properties for SSI Analysis of CB PE (UB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.310 -18.360	2.61	2420	5174	0.55	158.6	0.360						
-18.360 -24.460	2.61	2607	5233	0.55	85.4	0.335						
-24.460 -27.500	2.61	2965	5650	0.55	195.0	0.310						
-27.500 -30.550	2.63	3231	5845	0.55	211.8	0.280						
-30.550 -33.600	2.63	3077	5863	0.55	201.7	0.310						
-33.600 -36.650	2.63	3232	6047	0.55	211.9	0.300						
-36.650 -39.700	2.63	3293	5783	0.55	215.9	0.260						
-39.700 -42.740	2.63	3487	6308	0.55	229.4	0.280						
-42.740 -45.790	2.63	3434	6424	0.55	225.1	0.300						
-45.790 -	2.63	3434	5872	0.55		0.240						

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Table 3A.16.3-210 **Subsurface Properties for SSI Analysis of CB FE (BE Profile)**

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
4.500 / 3.740	2.00	266	540	2.78	53.6	0.340	2.08	198	370	3.14	49.9	0.299
3.740 / 2.980	2.00	266	540	2.78	53.6	0.340	2.08	198	370	3.14	49.9	0.299
2.980 / 2.215	2.00	307	1290	3.93	61.8	0.470	2.08	228	426	4.41	57.5	0.299
2.215 / 1.450	2.00	307	1463	3.93	61.8	0.477	2.08	228	1161	4.41	57.5	0.480
1.450 / 0.690	2.00	307	1463	3.93	61.8	0.477	2.08	229	1168	5.21	57.7	0.480
0.690 / -0.070	2.00	307	1463	3.93	61.8	0.477	2.08	229	1168	5.21	57.7	0.480
-0.070 / -0.720	2.08	433	1818	3.78	87.2	0.470	2.08	231	1180	5.92	58.2	0.480
-0.720 / -1.370	2.08	433	1818	3.78	87.2	0.470	2.08	231	1180	5.92	58.2	0.480
See Note → -1.370 / -2.000	2.08	433	1818	3.78	87.2	0.470	2.08	242	1232	5.76	60.9	0.480
-2.000 / -2.560	2.08	433	1818	3.78	87.2	0.470	2.08	248	1264	5.67	62.5	0.480

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Table 3A.16.3-210 **Subsurface Properties for SSI Analysis of CB FE (BE Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-2.560 / -3.120	2.08	433	1818	3.78	87.2	0.470	2.08	248	1264	5.67	62.5	0.480
-3.120 / -4.645	2.32	617	1885	0.62	62.2	0.440	2.32	2134	3325	1.00	215.2	0.150
-4.645 / -6.170	2.32	617	1885	0.62	62.2	0.440	2.32	2134	3325	1.00	215.2	0.150
-6.170 / -7.400	2.32	754	2305	0.63	76.0	0.440	2.32	2134	3325	1.00	215.2	0.150
-7.400 / -8.900	2.32	754	2305	0.63	76.0	0.440	2.32	2134	3325	1.00	215.2	0.150
-8.900 / -10.400	2.32	754	2305	0.63	76.0	0.440	2.32	2134	3325	1.00	215.2	0.150
-10.400 / -12.260	2.32	754	2305	0.63	76.0	0.440	2.32	2134	3325	1.00	215.2	0.150
-12.260 / -13.785	2.32	811	2477	0.53	81.7	0.440	2.32	2134	3325	1.00	215.2	0.150
-13.785 / -15.310	2.32	811	2477	0.53	81.7	0.440	2.32	2134	3325	1.00	215.2	0.150

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Table 3A.16.3-210 **Subsurface Properties for SSI Analysis of CB FE (BE Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.310 -18.360	2.61	1976	4225	1.00	129.5	0.360						
-18.360 -24.460	2.61	2128	4273	1.00	69.7	0.335						
-24.460 -27.500	2.61	2421	4613	1.00	159.2	0.310						
-27.500 -30.550	2.63	2638	4772	1.00	172.9	0.280						
-30.550 -33.600	2.63	2512	4787	1.00	164.7	0.310						
-33.600 -36.650	2.63	2639	4937	1.00	173.0	0.300						
-36.650 -39.700	2.63	2689	4722	1.00	176.3	0.260						
-39.700 -42.740	2.63	2847	5150	1.00	187.3	0.280						
-42.740 -45.790	2.63	2804	5245	1.00	183.8	0.300						
-45.790 -	2.63	2804	4794	1.00		0.240						

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Table 3A.16.3-210 **Subsurface Properties for SSI Analysis of CB FE (BE Profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**

Note: The soil properties of adjusted layer, shown red box, are evaluated from original properties shown below.

Original data

-1.370 -1.600	2.08	433	1818	3.78		0.470	2.08	231	1180	5.92		0.480
-1.600 -2.000	2.08	433	1818	3.78		0.470	2.08	248	1264	5.67		0.480

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Table 3A.16.3-211 Subsurface Properties for SSI Analysis of CB FE (LB Profile)

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
4.500 / 3.740	2.00	180	365	4.83	36.2	0.339	2.08	134	251	4.98	33.7	0.301
3.740 / 2.980	2.00	180	365	4.83	36.2	0.339	2.08	134	251	4.98	33.7	0.301
2.980 / 2.215	2.00	186	782	7.42	37.5	0.470	2.08	148	277	7.17	37.3	0.300
2.215 / 1.450	2.00	186	949	7.42	37.5	0.480	2.08	148	755	7.17	37.3	0.480
1.450 / 0.690	2.00	186	949	7.42	37.5	0.480	2.08	146	742	8.38	36.8	0.480
0.690 / -0.070	2.00	186	949	7.42	37.5	0.480	2.08	146	742	8.38	36.8	0.480
-0.070 / -0.720	2.08	302	1463	6.53	60.8	0.478	2.08	150	764	9.11	37.8	0.480
-0.720 / -1.370	2.08	302	1463	6.53	60.8	0.478	2.08	150	764	9.11	37.8	0.480
See Note → -1.370 / -2.000	2.08	302	1463	6.53	60.8	0.478	2.08	154	786	9.00	38.9	0.480
-2.000 / -2.560	2.08	302	1463	6.53	60.8	0.478	2.08	157	800	8.93	39.5	0.480

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Table 3A.16.3-211 **Subsurface Properties for SSI Analysis of CB FE (LB Profile)** *(continued)*

	Soil						**Backfill**					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-2.560 -3.120	2.08	302	1463	6.53	60.8	0.478	2.08	157	800	8.93	39.5	0.480
-3.120 -4.645	2.32	471	1463	1.06	47.5	0.442	2.32	1829	2850	1.80	184.4	0.150
-4.645 -6.170	2.32	471	1463	1.06	47.5	0.442	2.32	1829	2850	1.80	184.4	0.150
-6.170 -7.400	2.32	561	1715	1.14	56.5	0.440	2.32	1829	2850	1.80	184.4	0.150
-7.400 -8.900	2.32	561	1715	1.14	56.5	0.440	2.32	1829	2850	1.80	184.4	0.150
-8.900 -10.400	2.32	561	1715	1.14	56.5	0.440	2.32	1829	2850	1.80	184.4	0.150
-10.400 -12.260	2.32	561	1715	1.14	56.5	0.440	2.32	1829	2850	1.80	184.4	0.150
-12.260 -13.785	2.32	662	2022	0.96	66.7	0.440	2.32	1829	2850	1.80	184.4	0.150
-13.785 -15.310	2.32	662	2022	0.96	66.7	0.440	2.32	1829	2850	1.80	184.4	0.150

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Table 3A.16.3-211 **Subsurface Properties for SSI Analysis of CB FE (LB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-15.310 / -18.360	2.61	1613	3450	1.82	105.7	0.360						
-18.360 / -24.460	2.61	1738	3488	1.82	56.9	0.335						
-24.460 / -27.500	2.61	1977	3767	1.82	130.0	0.310						
-27.500 / -30.550	2.63	2154	3897	1.82	141.2	0.280						
-30.550 / -33.600	2.63	2051	3909	1.82	134.4	0.310						
-33.600 / -36.650	2.63	2155	4031	1.82	141.3	0.300						
-36.650 / -39.700	2.63	2195	3855	1.82	143.9	0.260						
-39.700 / -42.740	2.63	2324	4205	1.82	152.8	0.280						
-42.740 / -45.790	2.63	2289	4282	1.82	150.0	0.300						
-45.790 / -	2.63	2290	3915	1.82		0.240						

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Table 3A.16.3-211 **Subsurface Properties for SSI Analysis of CB FE (LB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
Note: The soil properties of adjusted layer, shown red box, are evaluated from original properties shown below.												
Original data												
-1.370 -1.600	2.08	302	1463	6.53		0.478	2.08	150	764	9.11		0.480
-1.600	2.08	302	1463	6.53		0.478	2.08	157	800	8.93		0.480

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Table 3A.16.3-212 Subsurface Properties for SSI Analysis of CB FE (UB Profile)

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
4.500 – 3.740	2.00	394	800	1.60	79.4	0.340	2.08	292	546	1.98	73.6	0.300
3.740 – 2.980	2.00	394	800	1.60	79.4	0.340	2.08	292	546	1.98	73.6	0.300
2.980 – 2.215	2.00	506	2126	2.08	102.0	0.470	2.08	350	655	2.72	88.2	0.300
2.215 – 1.450	2.00	506	2126	2.08	102.0	0.470	2.08	350	1463	2.72	88.2	0.470
1.450 – 0.690	2.00	506	2126	2.08	102.0	0.470	2.08	360	1463	3.24	90.7	0.468
0.690 – -0.070	2.00	506	2126	2.08	102.0	0.470	2.08	360	1463	3.24	90.7	0.468
-0.070 – -0.720	2.08	621	2608	2.19	125.2	0.470	2.08	358	1463	3.84	90.2	0.468
-0.720 – -1.370	2.08	621	2608	2.19	125.2	0.470	2.08	358	1463	3.84	90.2	0.468
See Note → -1.370 – -2.000	2.08	621	2608	2.19	125.2	0.470	2.08	378	1463	3.69	95.4	0.464
-2.000 – -2.560	2.08	621	2608	2.19	125.2	0.470	2.08	391	1463	3.60	98.6	0.462

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Table 3A.16.3-212 **Subsurface Properties for SSI Analysis of CB FE (UB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-2.560 / -3.120	2.08	621	2608	2.19	125.2	0.470	2.08	391	1463	3.60	98.6	0.462
-3.120 / -4.645	2.32	808	2469	0.36	81.4	0.440	2.32	2438	3800	0.55	245.8	0.150
-4.645 / -6.170	2.32	808	2469	0.36	81.4	0.440	2.32	2438	3800	0.55	245.8	0.150
-6.170 / -7.400	2.32	1014	3098	0.35	102.2	0.440	2.32	2438	3800	0.55	245.8	0.150
-7.400 / -8.900	2.32	1014	3098	0.35	102.2	0.440	2.32	2438	3800	0.55	245.8	0.150
-8.900 / -10.400	2.32	1014	3098	0.35	102.2	0.440	2.32	2438	3800	0.55	245.8	0.150
-10.400 / -12.260	2.32	1014	3098	0.35	102.2	0.440	2.32	2438	3800	0.55	245.8	0.150
-12.260 / -13.785	2.32	993	3034	0.29	100.1	0.440	2.32	2438	3800	0.55	245.8	0.150
-13.785 / -15.310	2.32	993	3034	0.29	100.1	0.440	2.32	2438	3800	0.55	245.8	0.150

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Table 3A.16.3-212 **Subsurface Properties for SSI Analysis of CB FE (UB Profile)** *(continued)*

	Soil						**Backfill**					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-15.310 -18.360	2.61	2420	5174	0.55	158.6	0.360						
-18.360 -24.460	2.61	2607	5233	0.55	85.4	0.335						
-24.460 -27.500	2.61	2965	5650	0.55	195.0	0.310						
-27.500 -30.550	2.63	3231	5845	0.55	211.8	0.280						
-30.550 -33.600	2.63	3077	5863	0.55	201.7	0.310						
-33.600 -36.650	2.63	3232	6047	0.55	211.9	0.300						
-36.650 -39.700	2.63	3293	5783	0.55	215.9	0.260						
-39.700 -42.740	2.63	3487	6308	0.55	229.4	0.280						
-42.740 -45.790	2.63	3434	6424	0.55	225.1	0.300						
-45.790 -	2.63	3434	5872	0.55		0.240						

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Table 3A.16.3-212 **Subsurface Properties for SSI Analysis of CB FE (UB Profile)** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m³)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m³)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
Note: The soil properties of adjusted layer, shown red box, are evaluated from original properties shown below.												
Original data												
-1.370 -1.600	2.08	621	2608	2.19		0.470	2.08	358	1463	3.84		0.468
-1.600 -2.000	2.08	621	2608	2.19		0.470	2.08	391	1463	3.60		0.462

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Table 3A.16.3-213 **Site Model Properties for SSI Analysis of BE Profile - FWSC**

	Soil						Backfill					
EL (m)	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
2.15 / 1.40	2.00	226	1153	4.18	54.7	0.480	2.32	2134	3325	1.00	517.3	0.150
1.40 / 0.65	2.00	226	1153	4.18	54.7	0.480	2.32	2134	3325	1.00	517.3	0.150
0.65 / -0.10	2.00	226	1153	4.18	54.7	0.480	2.32	2134	3325	1.00	517.3	0.150
-0.10 / -0.85	2.00	226	1153	4.18	54.7	0.480	2.32	2134	3325	1.00	517.3	0.150
-0.85 / -1.60	2.00	226	1153	4.18	54.7	0.480	2.32	2134	3325	1.00	517.3	0.150
-1.60 / -2.46	2.00	298	1463	5.00	69.3	0.478	2.32	2134	3325	1.00	496.2	0.150
-2.46 / -3.32	2.00	298	1463	5.00	69.3	0.478	2.32	2134	3325	1.00	496.2	0.150
-3.32 / -4.18	2.00	298	1463	5.00	69.3	0.478	2.32	2134	3325	1.00	496.2	0.150

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Table 3A.16.3-213 **Site Model Properties for SSI Analysis of BE Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-4.18 -5.04	2.00	298	1463	5.00	69.3	0.478	2.32	2134	3325	1.00	496.2	0.150
-5.04 -5.91	2.00	298	1463	5.00	68.5	0.478	2.32	2134	3325	1.00	490.5	0.150
-5.91 -6.78	2.00	298	1463	5.00	68.5	0.478	2.32	2134	3325	1.00	490.5	0.150
-6.78 -8.000	2.08	432	1814	3.97	70.8	0.470	2.32	2134	3325	1.00	349.8	0.150
-8.000 -9.520	2.08	596	1821	2.89	78.4	0.440	2.32	2134	3325	1.00	280.7	0.150
-9.520 -11.140	2.32	763	2331	0.64	94.1	0.440	2.32	2134	3325	1.00	263.4	0.150
-11.140 -12.770	2.32	763	2331	0.64	93.6	0.440	2.32	2134	3325	1.00	261.8	0.150
-12.770 -14.400	2.32	763	2331	0.64	93.6	0.440	2.32	2134	3325	1.00	261.8	0.150

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Table 3A.16.3-213 **Site Model Properties for SSI Analysis of BE Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-14.400 / -15.620	2.32	821	2508	0.53	134.5	0.440	2.32	2134	3325	1.00	349.8	0.150
-15.620 / -16.840	2.32	821	2508	0.53	134.5	0.440	2.32	2134	3325	1.00	349.8	0.150

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Table 3A.16.3-213 **Site Model Properties for SSI Analysis of BE Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-16.840 -18.670	2.61	1976	4225	1.00	215.9	0.360						
-18.670 -22.020	2.61	2128	4273	1.00	127.0	0.335						
-22.020 -25.37	2.63	2421	4613	1.00	144.5	0.310						
-25.37 -28.72	2.63	2638	4772	1.00	157.4	0.280						
-28.72 -32.08	2.63	2512	4787	1.00	149.5	0.310						
-32.08 -35.43	2.63	2639	4937	1.00	157.5	0.300						
-35.43 -38.78	2.63	2689	4722	1.00	160.5	0.260						
-38.78 -42.14	2.63	2847	5150	1.00	169.4	0.280						
-42.14 -45.49	2.63	2804	5245	1.00	167.4	0.300						
-45.49 -	2.63	2804	4794	1.00	-	0.240						

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Table 3A.16.3-214 **Site Model Properties for SSI Analysis of LB Profile - FWSC**

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
2.15 / 1.40	2.00	147	750	6.63	35.6	0.480	2.32	1829	2850	1.82	443.3	0.150
1.40 / 0.65	2.00	147	750	6.63	35.6	0.480	2.32	1829	2850	1.82	443.3	0.150
0.65 / -0.10	2.00	147	750	6.63	35.6	0.480	2.32	1829	2850	1.82	443.3	0.150
-0.10 / -0.85	2.00	147	750	6.63	35.6	0.480	2.32	1829	2850	1.82	443.3	0.150
-0.85 / -1.60	2.00	147	750	6.63	35.6	0.480	2.32	1829	2850	1.82	443.3	0.150
-1.60 / -2.46	2.00	184	940	8.25	42.7	0.480	2.32	1829	2850	1.82	425.3	0.150
-2.46 / -3.32	2.00	184	940	8.25	42.7	0.480	2.32	1829	2850	1.82	425.3	0.150
-3.32 / -4.18	2.00	184	940	8.25	42.7	0.480	2.32	1829	2850	1.82	425.3	0.150

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Table 3A.16.3-214 **Site Model Properties for SSI Analysis of LB Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-4.18 / -5.04	2.00	184	940	8.25	42.7	0.480	2.32	1829	2850	1.82	425.3	0.150
-5.04 / -5.91	2.00	184	940	8.25	42.2	0.480	2.32	1829	2850	1.82	420.4	0.150
-5.91 / -6.78	2.00	184	940	8.25	42.2	0.480	2.32	1829	2850	1.82	420.4	0.150
-6.78 / -8.000	2.08	310	1463	6.08	50.8	0.476	2.32	1829	2850	1.82	299.8	0.150
-8.000 / -9.520	2.08	436	1463	4.37	57.3	0.451	2.32	1829	2850	1.82	240.6	0.150
-9.520 / -11.140	2.32	581	1775	1.14	71.7	0.440	2.32	1829	2850	1.82	225.8	0.150
-11.140 / -12.770	2.32	581	1775	1.14	71.2	0.440	2.32	1829	2850	1.82	224.4	0.150
-12.770 / -14.400	2.32	581	1775	1.14	71.2	0.440	2.32	1829	2850	1.82	224.4	0.150

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Table 3A.16.3-214 **Site Model Properties for SSI Analysis of LB Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m³)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-14.400 -15.620	2.32	655	2002	0.94	107.3	0.440	2.32	1829	2850	1.82	299.8	0.150
-15.620 -16.840	2.32	655	2002	0.94	107.3	0.440	2.32	1829	2850	1.82	299.8	0.150

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Table 3A.16.3-214 **Site Model Properties for SSI Analysis of LB Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-16.840 -18.670	2.61	1464	3130	1.82	160.0	0.360						
-18.670 -22.020	2.61	1738	3488	1.82	103.7	0.335						
-22.020 -25.37	2.63	1977	3767	1.82	118.0	0.310						
-25.37 -28.72	2.63	2154	3897	1.82	128.5	0.280						
-28.72 -32.08	2.63	2051	3909	1.82	122.0	0.310						
-32.08 -35.43	2.63	2155	4031	1.82	128.6	0.300						
-35.43 -38.78	2.63	2195	3855	1.82	131.0	0.260						
-38.78 -42.14	2.63	2324	4205	1.82	138.3	0.280						
-42.14 -45.49	2.63	2289	4282	1.82	136.6	0.300						
-45.49 -	2.63	2290	3915	1.82	-	0.240						

NAPS DEP 3.7-1

Table 3A.16.3-215 **Site Model Properties for SSI Analysis of UB Profile - FWSC**

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
2.15 / 1.40	2.00	348	1463	2.64	84.3	0.470	2.32	2438	3800	0.55	591.0	0.150
1.40 / 0.65	2.00	348	1463	2.64	84.3	0.470	2.32	2438	3800	0.55	591.0	0.150
0.65 / -0.10	2.00	348	1463	2.64	84.3	0.470	2.32	2438	3800	0.55	591.0	0.150
-0.10 / -0.85	2.00	348	1463	2.64	84.3	0.470	2.32	2438	3800	0.55	591.0	0.150
-0.85 / -1.60	2.00	348	1463	2.64	84.3	0.470	2.32	2438	3800	0.55	591.0	0.150
-1.60 / -2.46	2.00	483	2030	3.03	112.3	0.470	2.32	2438	3800	0.55	566.9	0.150
-2.46 / -3.32	2.00	483	2030	3.03	112.3	0.470	2.32	2438	3800	0.55	566.9	0.150
-3.32 / -4.18	2.00	483	2030	3.03	112.3	0.470	2.32	2438	3800	0.55	566.9	0.150

NAPS DEP 3.7-1

Table 3A.16.3-215 **Site Model Properties for SSI Analysis of UB Profile - FWSC** *(continued)*

EL (m)	Soil: Unit Weight (t/m^3)	Soil: Vs (m/sec)	Soil: Vp (m/sec)	Soil: Damping (%)	Soil: Highest Frequency (Hz)	Soil: Poisson's ratio	Backfill: Unit Weight (t/m^3)	Backfill: Vs (m/sec)	Backfill: Vp (m/sec)	Backfill: Damping (%)	Backfill: Highest Frequency (Hz)	Backfill: Poisson's ratio
-4.18 / -5.04	2.00	483	2030	3.03	112.3	0.470	2.32	2438	3800	0.55	566.9	0.150
-5.04 / -5.91	2.00	483	2030	3.03	111.0	0.470	2.32	2438	3800	0.55	560.4	0.150
-5.91 / -6.78	2.00	483	2030	3.03	111.0	0.470	2.32	2438	3800	0.55	560.4	0.150
-6.78 / -8.000	2.08	600	2524	2.60	98.3	0.470	2.32	2438	3800	0.55	399.6	0.150
-8.000 / -9.520	2.08	814	2488	1.92	107.1	0.440	2.32	2438	3800	0.55	320.7	0.150
-9.520 / -11.140	2.32	1002	3060	0.36	123.7	0.440	2.32	2438	3800	0.55	300.9	0.150
-11.140 / -12.770	2.32	1002	3060	0.36	122.9	0.440	2.32	2438	3800	0.55	299.1	0.150
-12.770 / -14.400	2.32	1002	3060	0.36	122.9	0.440	2.32	2438	3800	0.55	299.1	0.150

NAPS DEP 3.7-1

Table 3A.16.3-215 **Site Model Properties for SSI Analysis of UB Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**	**Unit Weight (t/m^3)**	**Vs (m/sec)**	**Vp (m/sec)**	**Damping (%)**	**Highest Frequency (Hz)**	**Poisson's ratio**
-14.400 -15.620	2.32	1028	3141	0.30	168.5	0.440	2.32	2438	3800	0.55	399.6	0.150
-15.620 -16.840	2.32	1028	3141	0.30	168.5	0.440	2.32	2438	3800	0.55	399.6	0.150

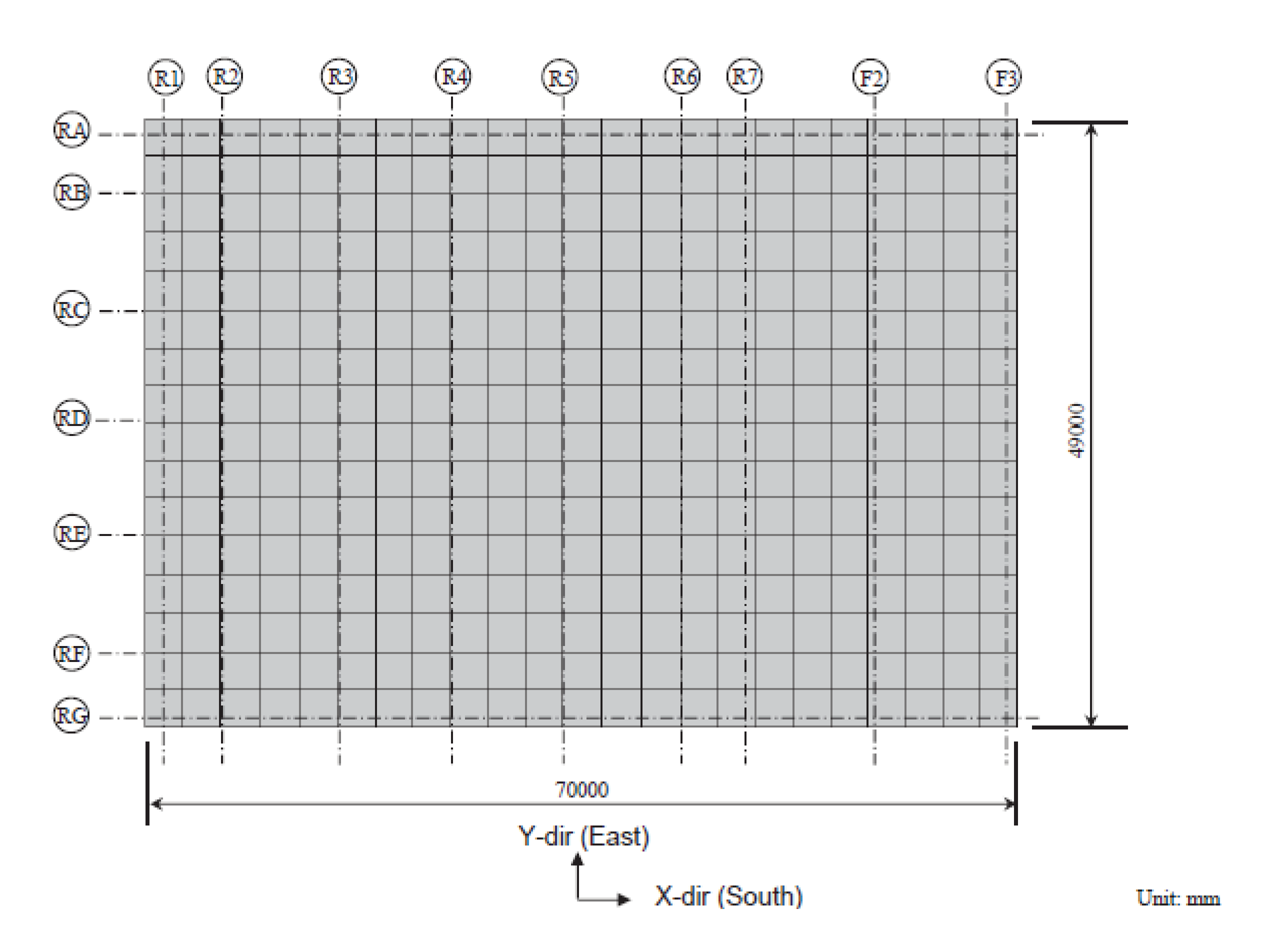
NAPS DEP 3.7-1

Table 3A.16.3-215 **Site Model Properties for SSI Analysis of UB Profile - FWSC** *(continued)*

	Soil						Backfill					
EL (m)	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio	Unit Weight (t/m^3)	Vs (m/sec)	Vp (m/sec)	Damping (%)	Highest Frequency (Hz)	Poisson's ratio
-16.840 -18.670	2.61	2667	5703	0.55	291.4	0.360						
-18.670 -22.020	2.61	2607	5233	0.55	155.6	0.335						
-22.020 -25.37	2.63	2965	5650	0.55	177.0	0.310						
-25.37 -28.72	2.63	3231	5845	0.55	192.8	0.280						
-28.72 -32.08	2.63	3077	5863	0.55	183.1	0.310						
-32.08 -35.43	2.63	3232	6047	0.55	192.9	0.300						
-35.43 -38.78	2.63	3293	5783	0.55	196.5	0.260						
-38.78 -42.14	2.63	3487	6308	0.55	207.5	0.280						
-42.14 -45.49	2.63	3434	6424	0.55	205.0	0.300						
-45.49 -	2.63	3434	5872	0.55	-	0.240						

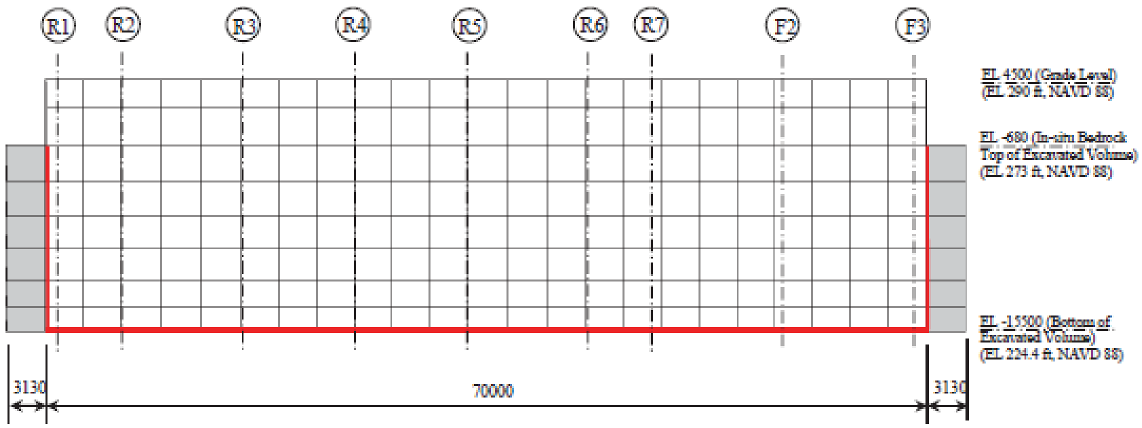
NAPS DEP 3.7-1

Figure 3A.16.3-201 **SASSI2010 Plate Elements for RB/FB Basemat in Partially Embedded Model**



NAPS DEP 3.7-1

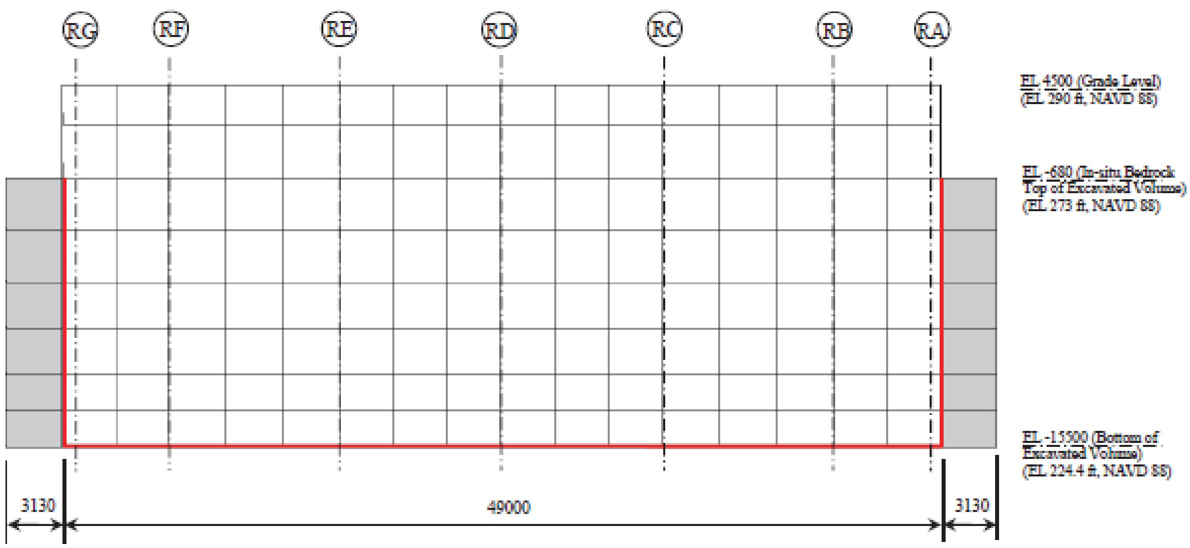
Figure 3A.16.3-202 **SASSI2010 Plate Elements for RB/FB Exterior Walls in the Partially Embedded Model**



(a) Walls on Column Rows RA and RG

Unit: mm

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.



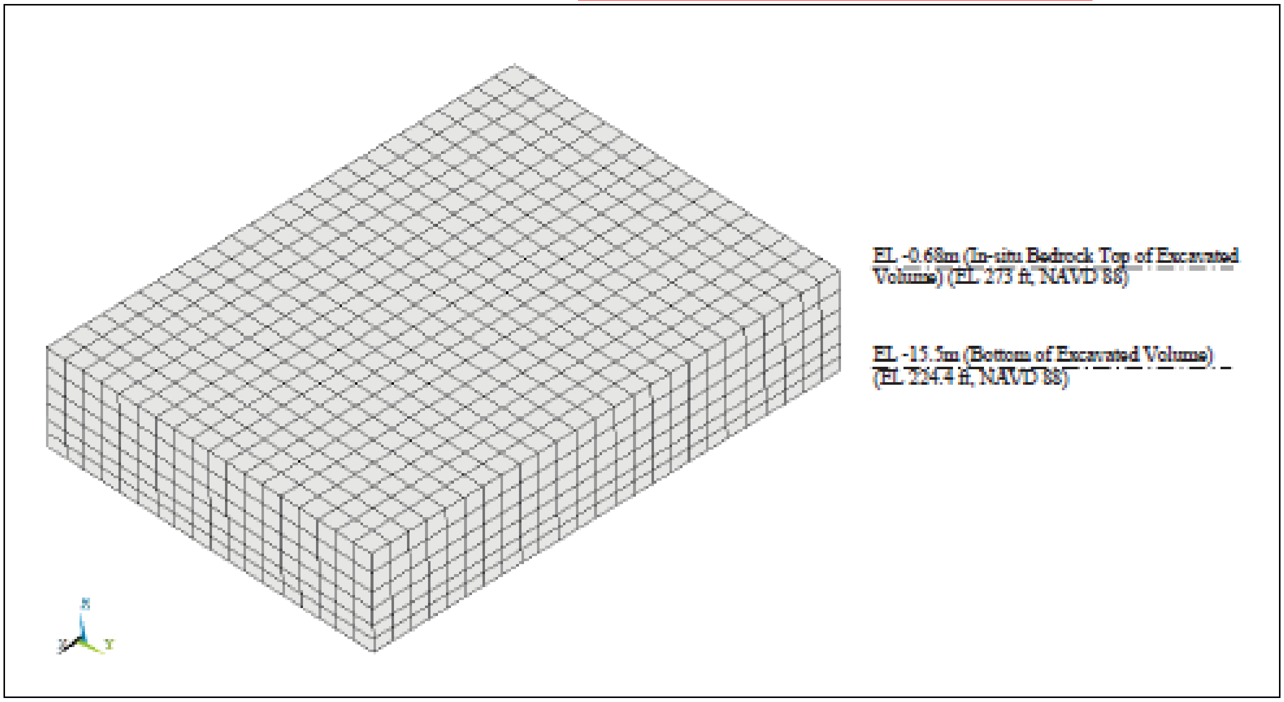
(b) Walls on Column Rows R1 and F3

Unit: mm

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

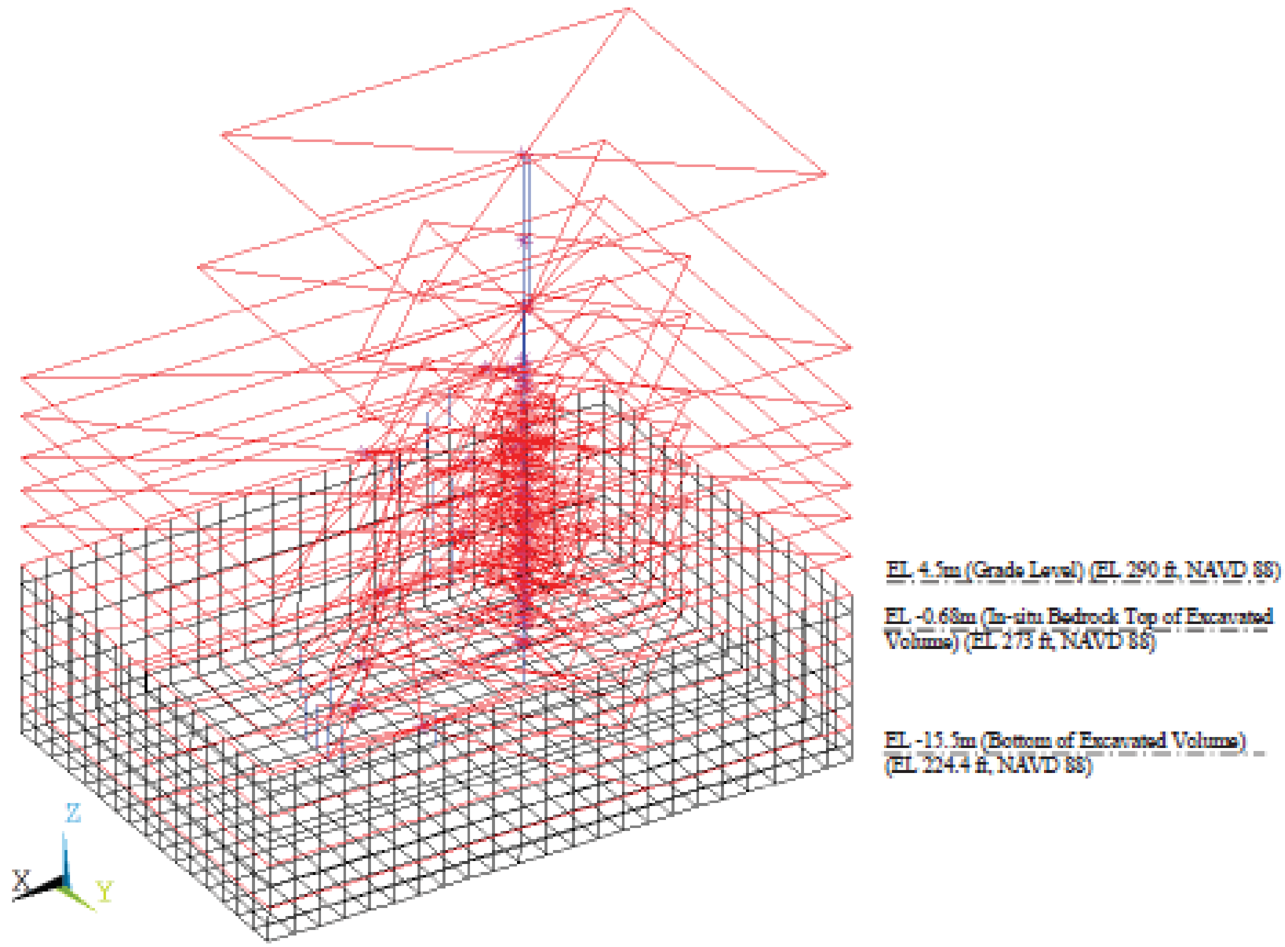
NAPS DEP 3.7-1

Figure 3A.16.3-203 **SASSI2010 Excavated Volume Solid Elements for the RB/FB Partially Embedded Model**



NAPS DEP 3.7-1

Figure 3A.16.3-204a **Overview of SASSI2010 SSI RB/FB Partially Embedded Model**

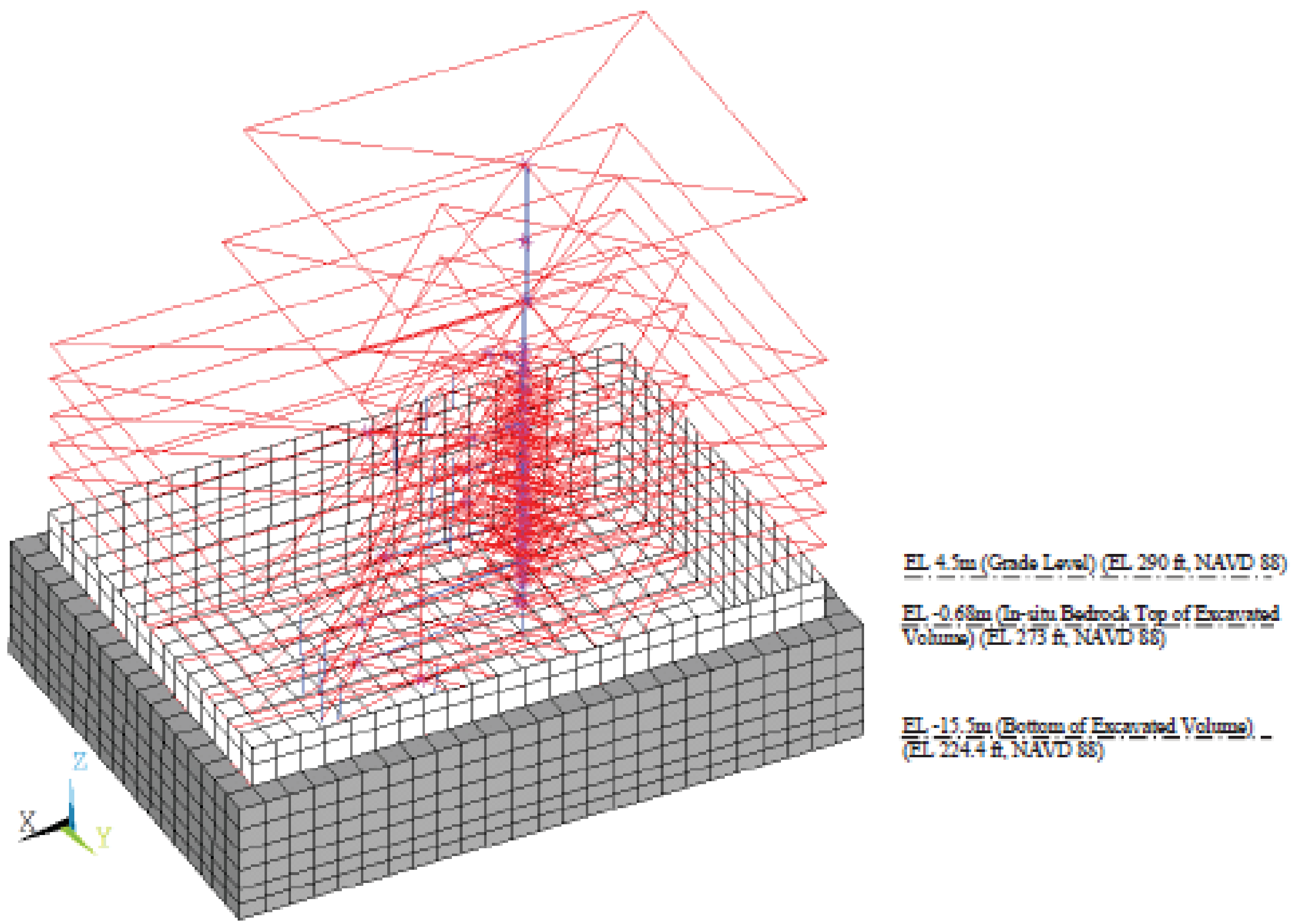


(a) Overview without Concrete Fill and Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams indicated in red are installed at the floor levels.

NAPS DEP 3.7-1

Figure 3A.16.3-204b **Overview of SASSI2010 SSI RB/FB Partially Embedded Model**

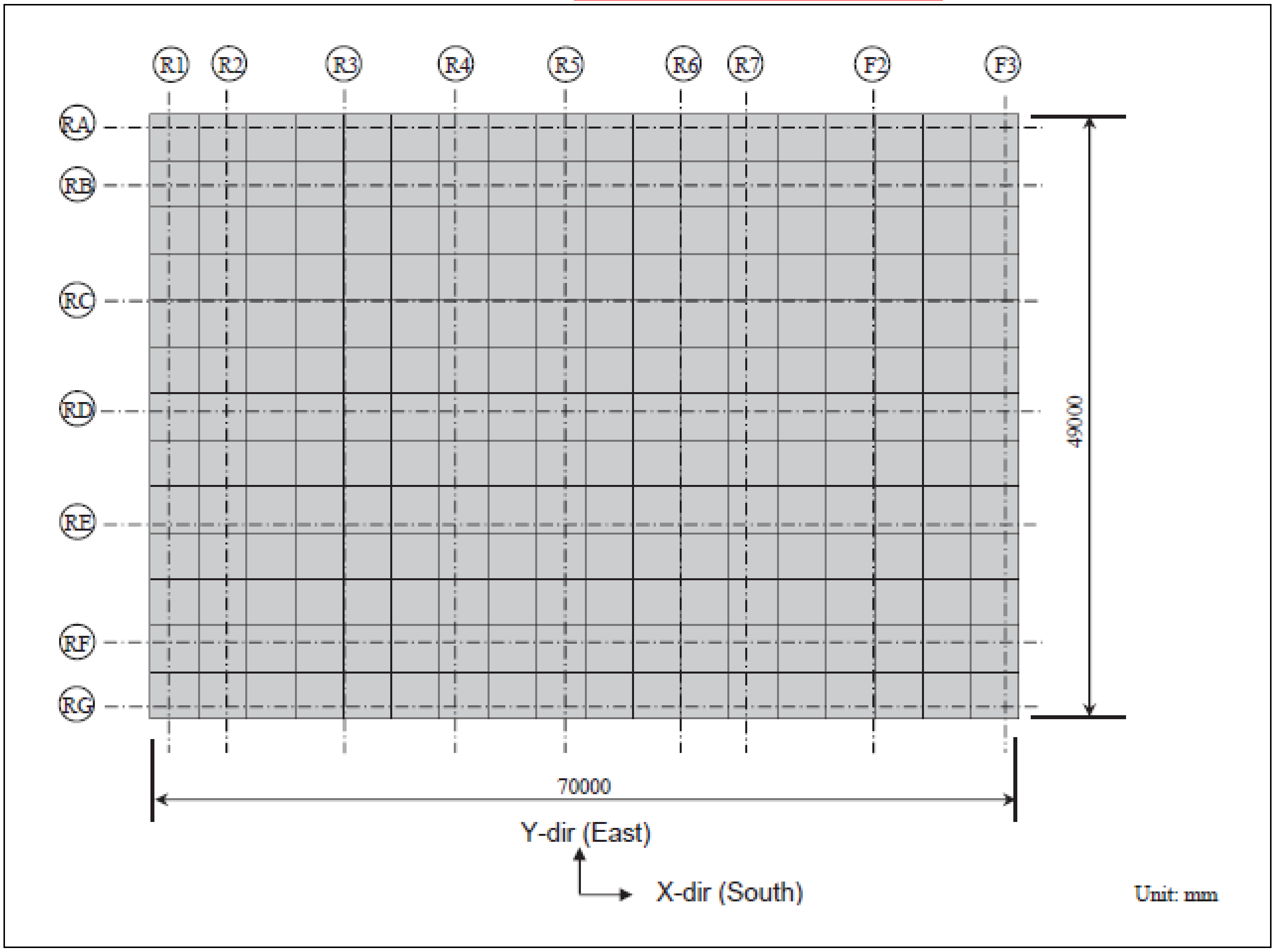


(b) Overview with Concrete Fill and without Excavated Volume

Note:
1) Wall and basemat are modeled with shell elements.
2) Rigid beams indicated in red are installed at the floor levels.

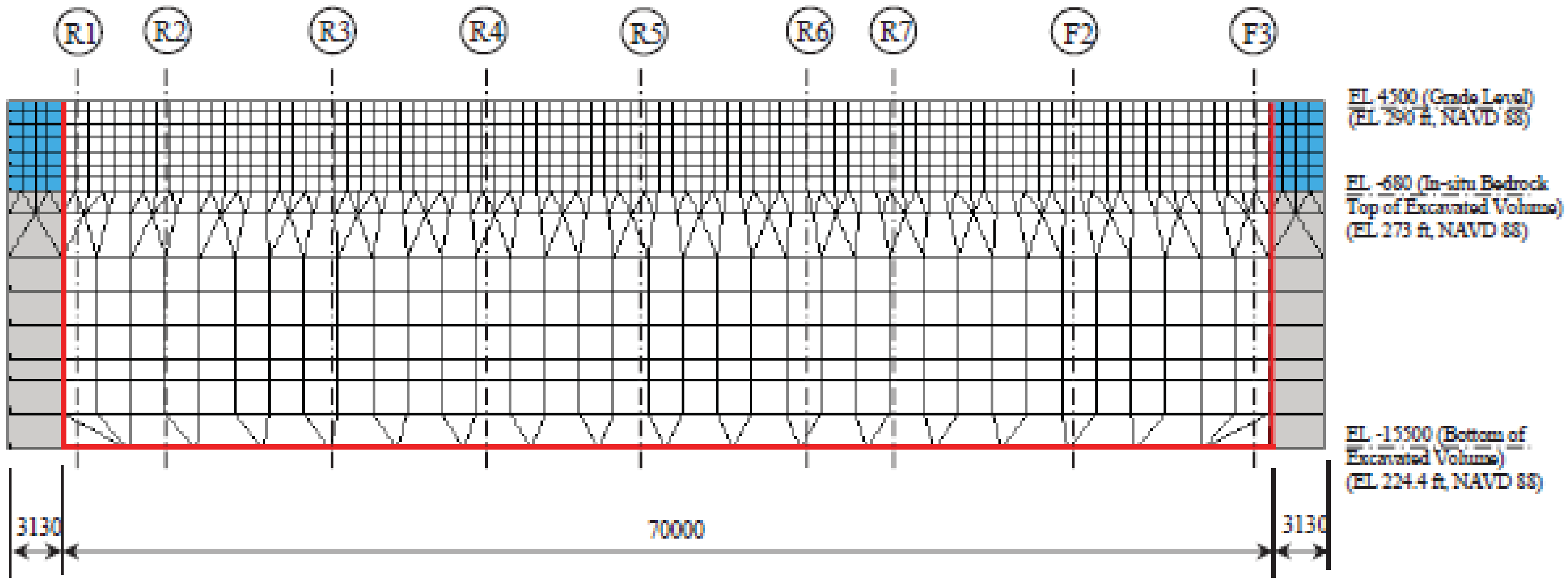
NAPS DEP 3.7-1

Figure 3A.16.3-205 **SASSI2010 Plate Elements for RB/FB Basemat in the Fully Embedded Model**



NAPS DEP 3.7-1

Figure 3A.16.3-206a **SASSI2010 Plate Elements for RB/FB Exterior Walls in the Fully Embedded Model**



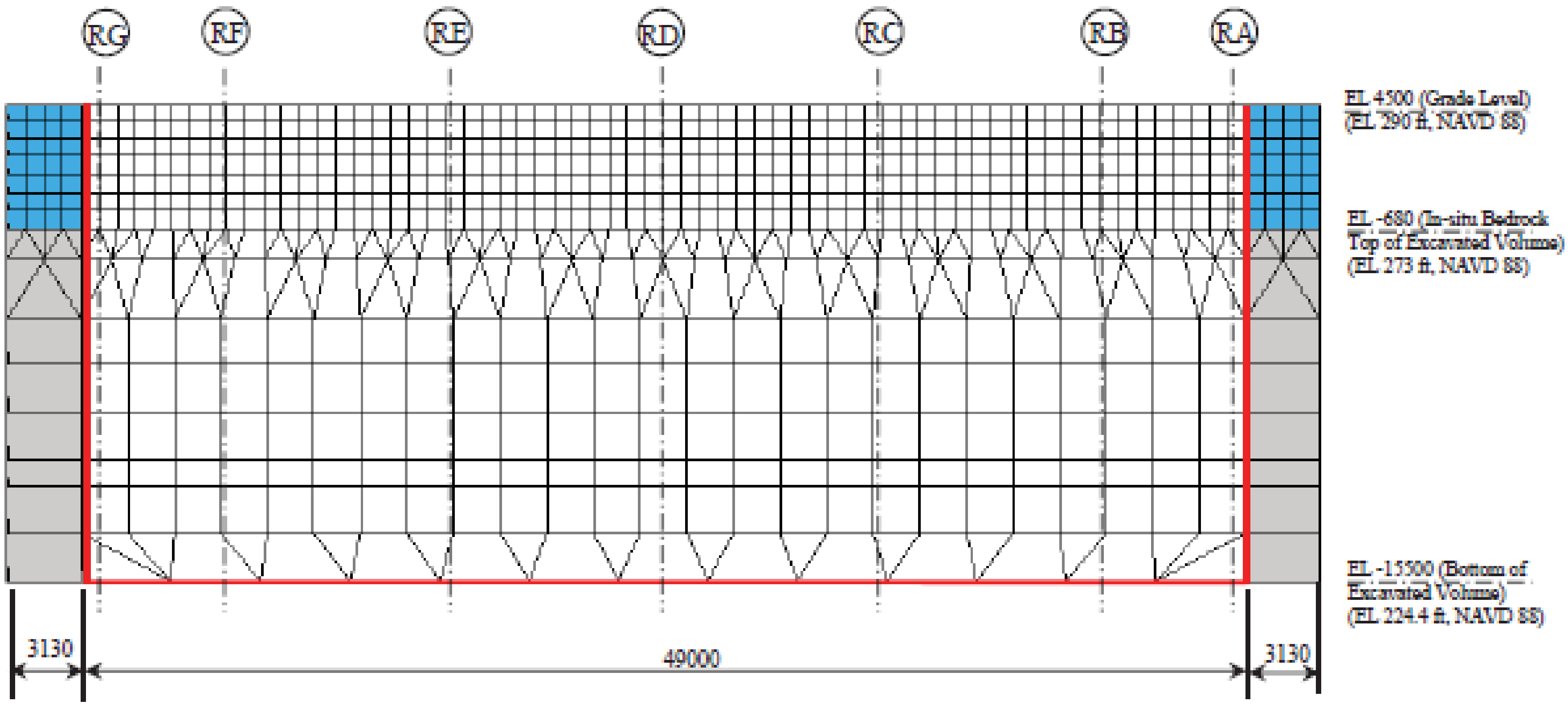
(a) Walls on Column Rows RA and RG

Unit: mm

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

NAPS DEP 3.7-1

Figure 3A.16.3-206b **SASSI2010 Plate Elements for RB/FB Exterior Walls in the Fully Embedded Model**



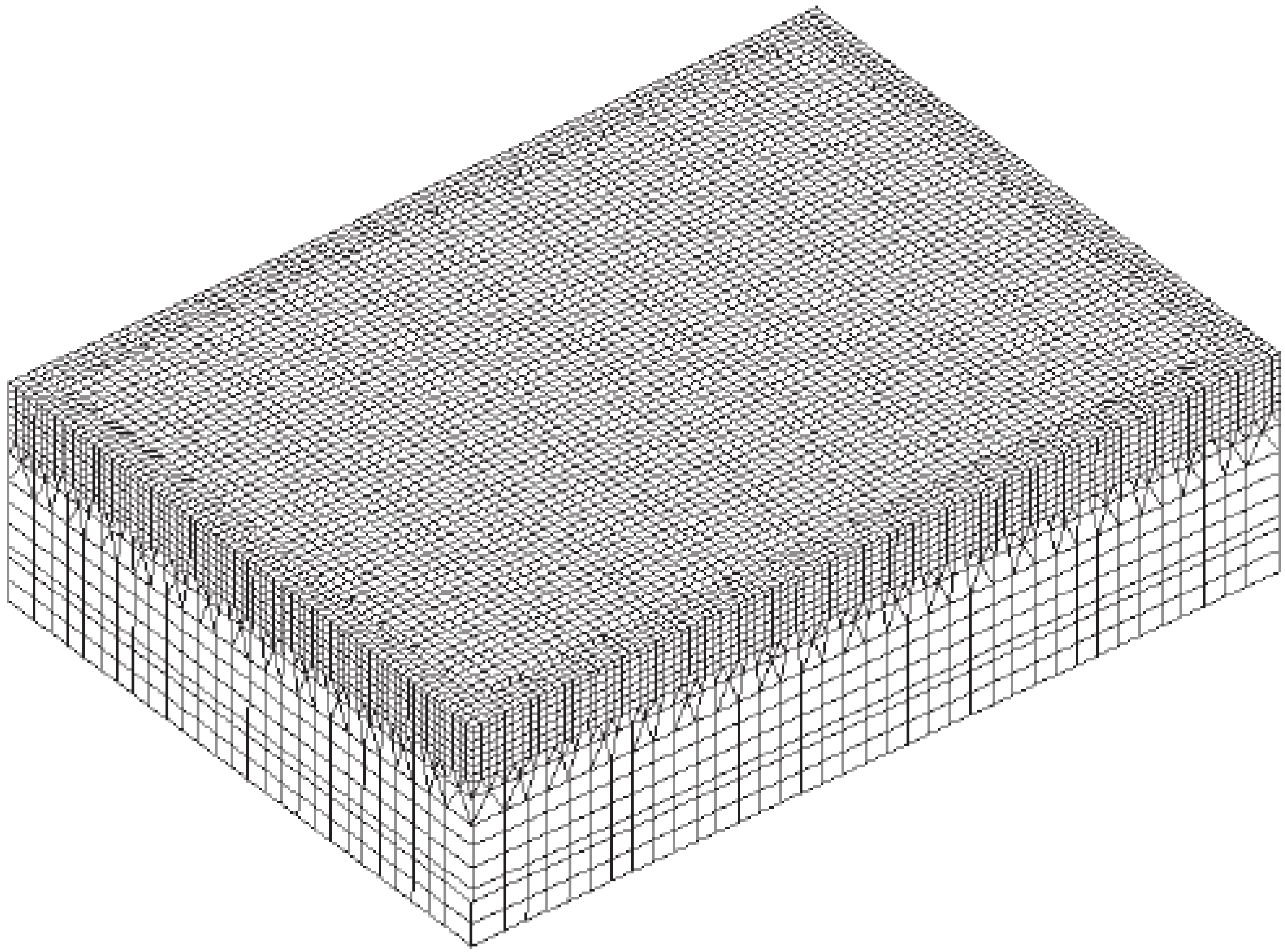
(b) Walls on Column Rows R1 and F3

Unit: mm

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

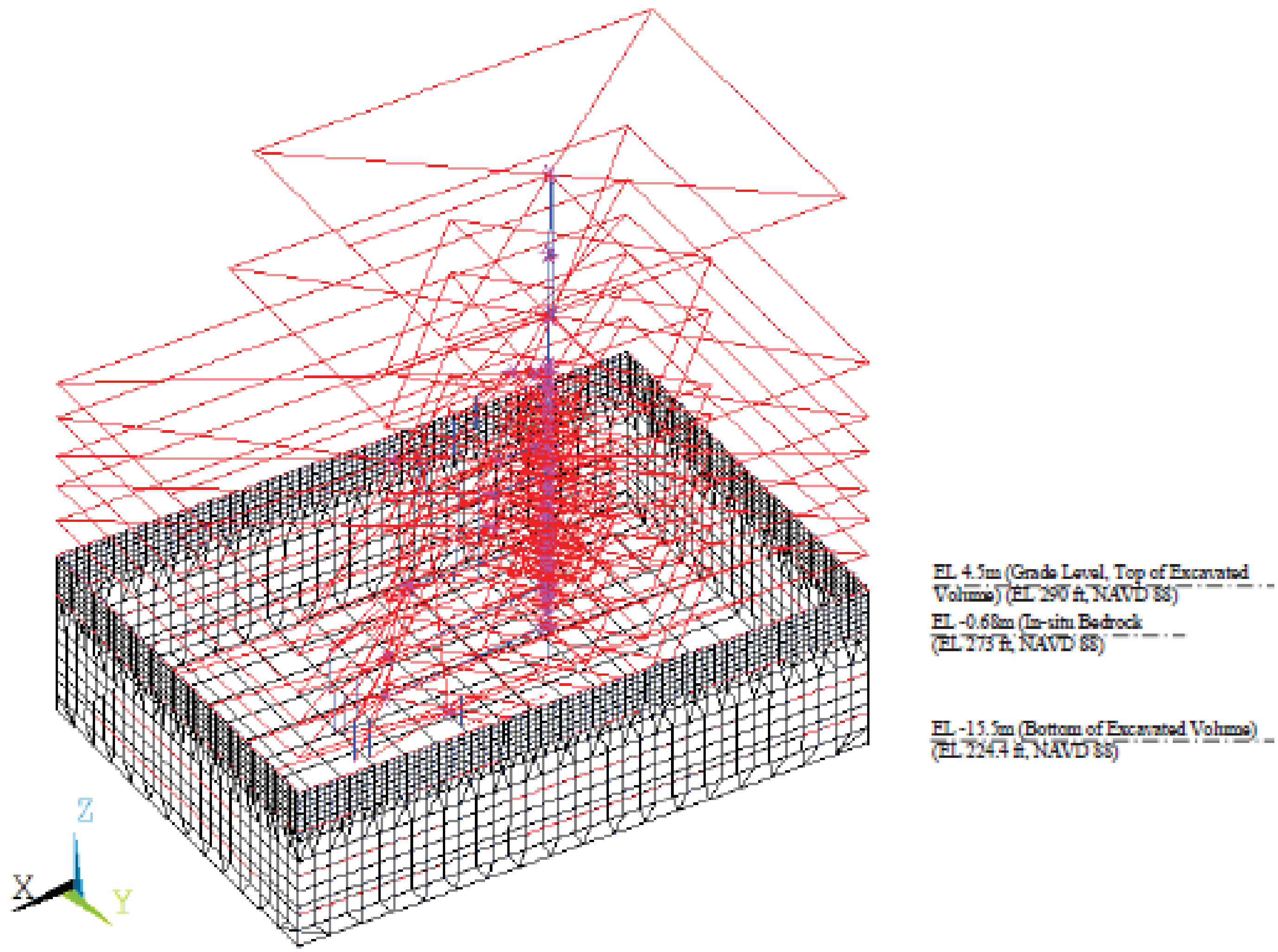
NAPS DEP 3.7-1

Figure 3A.16.3-207 **SASSI2010 Excavated Volume Solid Elements for the RB/FB Fully Embedded Model**



NAPS DEP 3.7-1

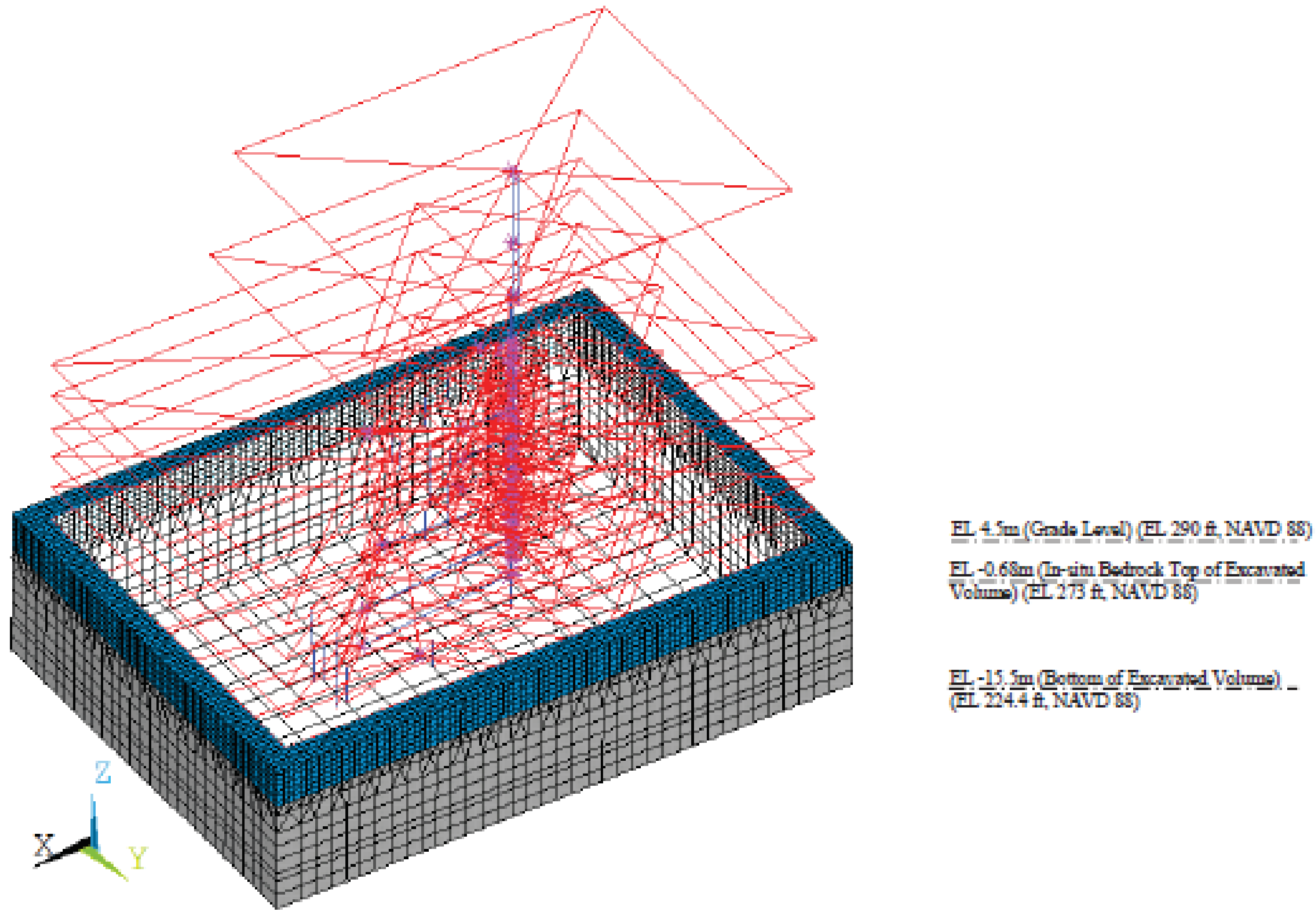
Figure 3A.16.3-208a **Overview of SASSI2010 SSI RB/FB Fully Embedded Model**



(a) Overview without Concrete Fill and Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams indicated in red are installed at the floor levels.

NAPS DEP 3.7-1 Figure 3A.16.3-208b **Overview of SASSI2010 SSI RB/FB Fully Embedded Model**

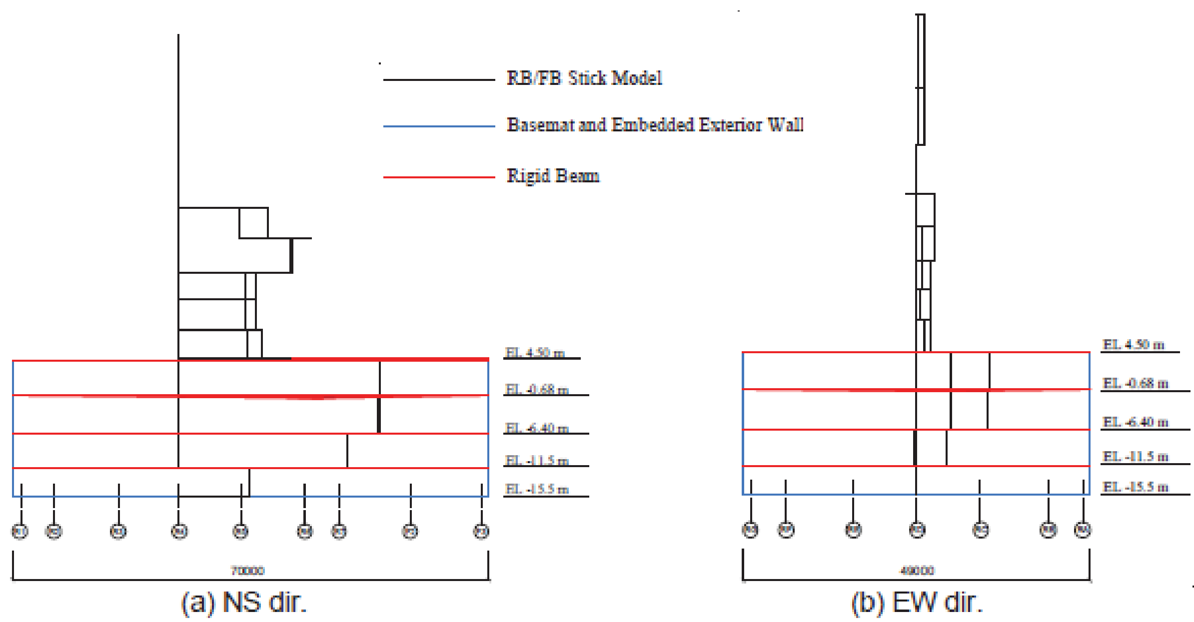


(b) Overview with Concrete Fill and without Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams indicated in red are installed at the floor levels.

NAPS DEP 3.7-1

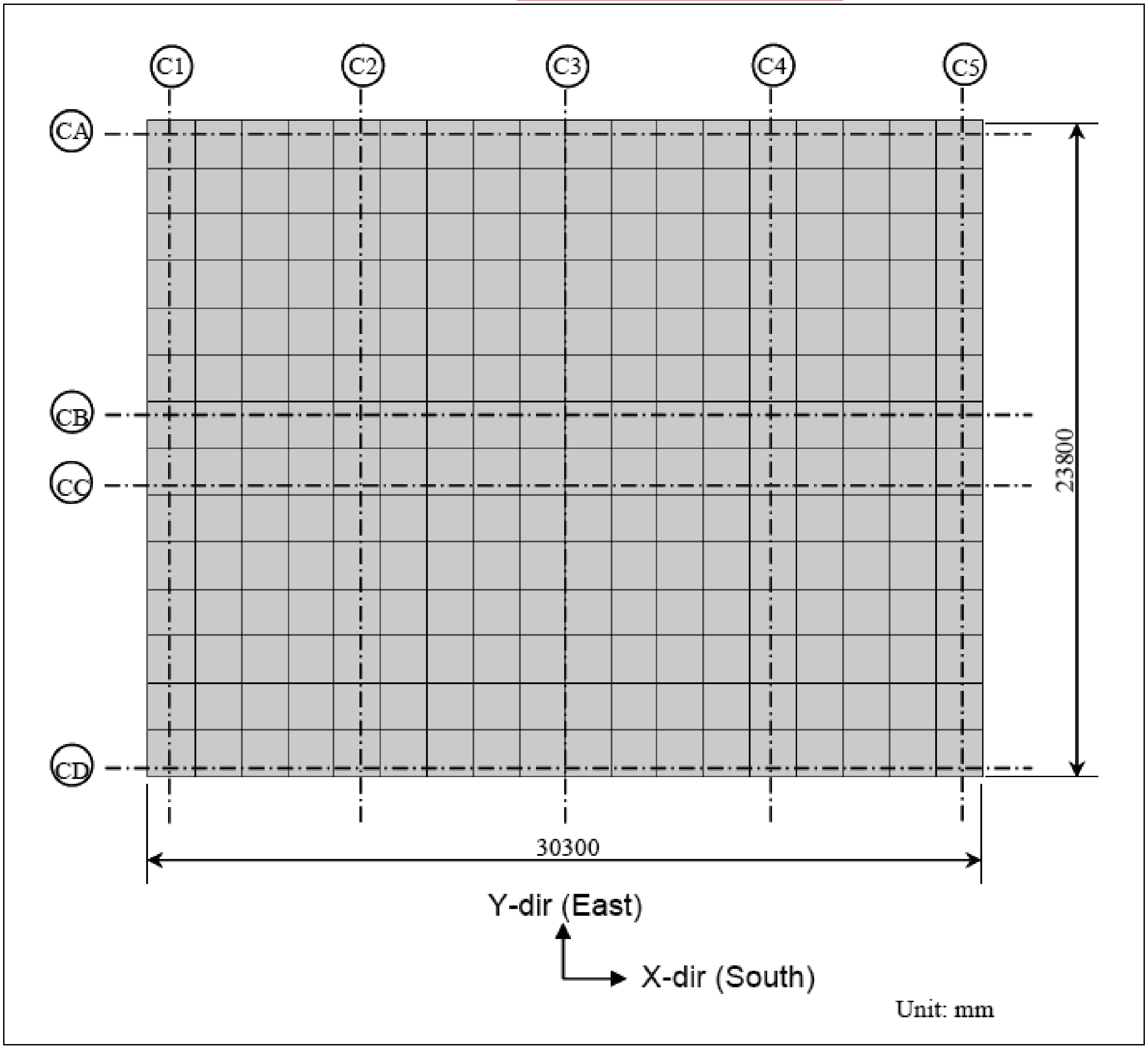
Figure 3A.16.3-209 Connection Between RB/FB Stick Model and Foundation



Note: Outriggers installed to the RB/FB stick are not shown in the figures.

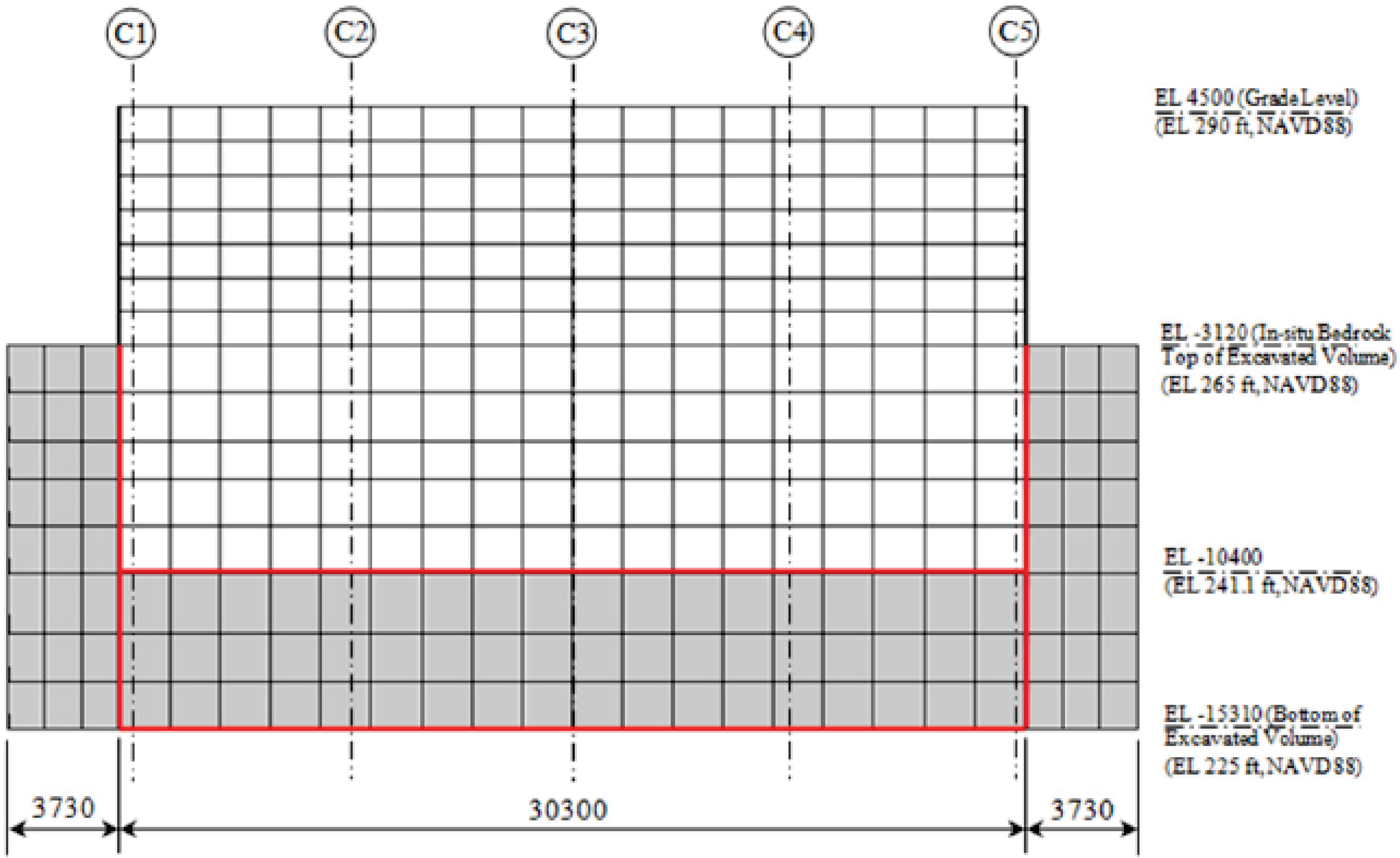
NAPS DEP 3.7-1

Figure 3A.16.3-210 **SASSI2010 Plate Elements for CB Basemat in the Partially Embedded Model**



NAPS DEP 3.7-1

Figure 3A.16.3-211 **SASSI2010 Plate Elements for CB Exterior Walls in the Partially Embedded Model**

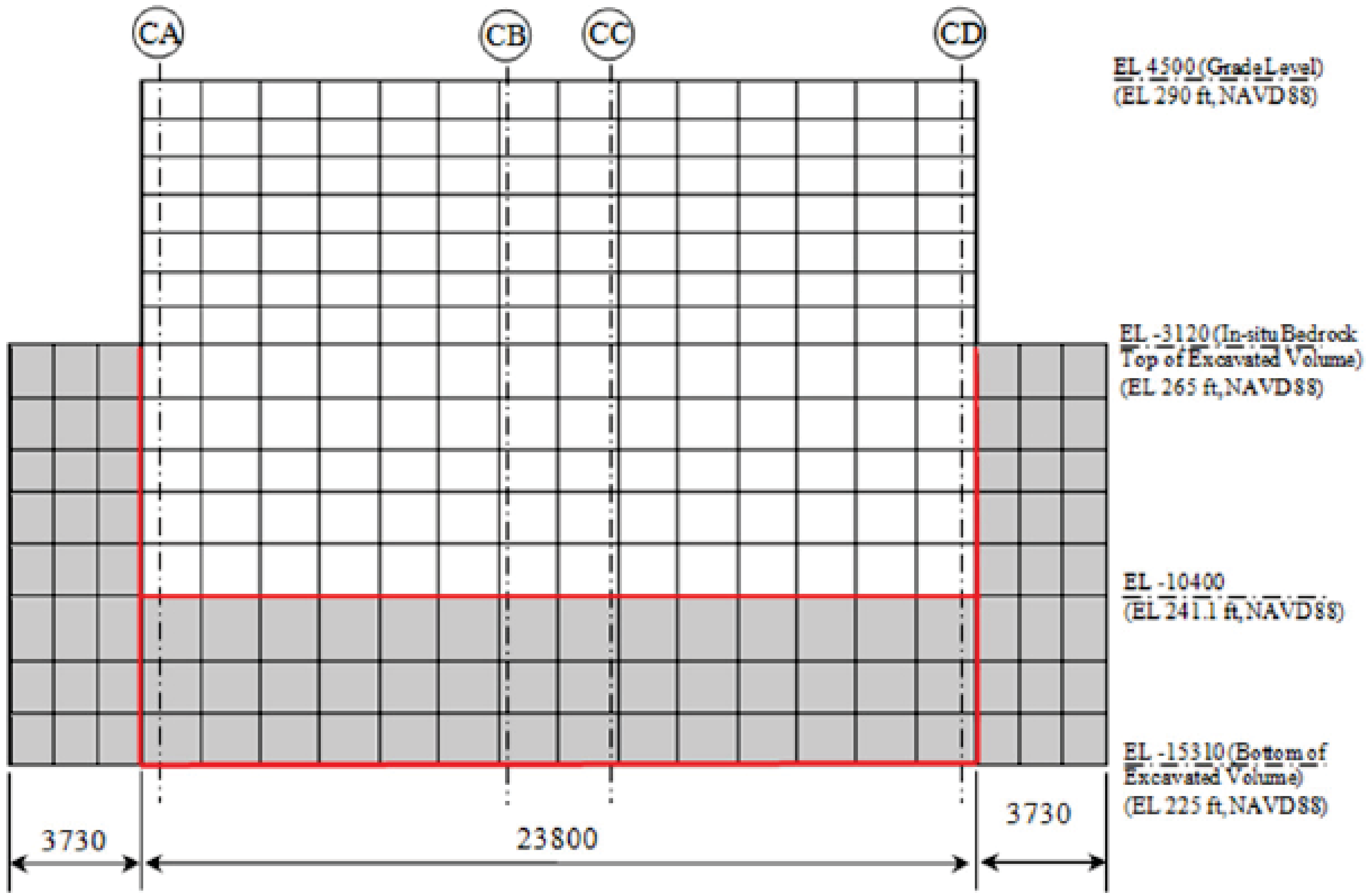


(a) Walls on Column Rows CA and CD (Unit: mm)

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

NAPS DEP 3.7-1

Figure 3A.16.3-211 **SASSI2010 Plate Elements for CB Exterior Walls in the Partially Embedded Model** *(continued)*

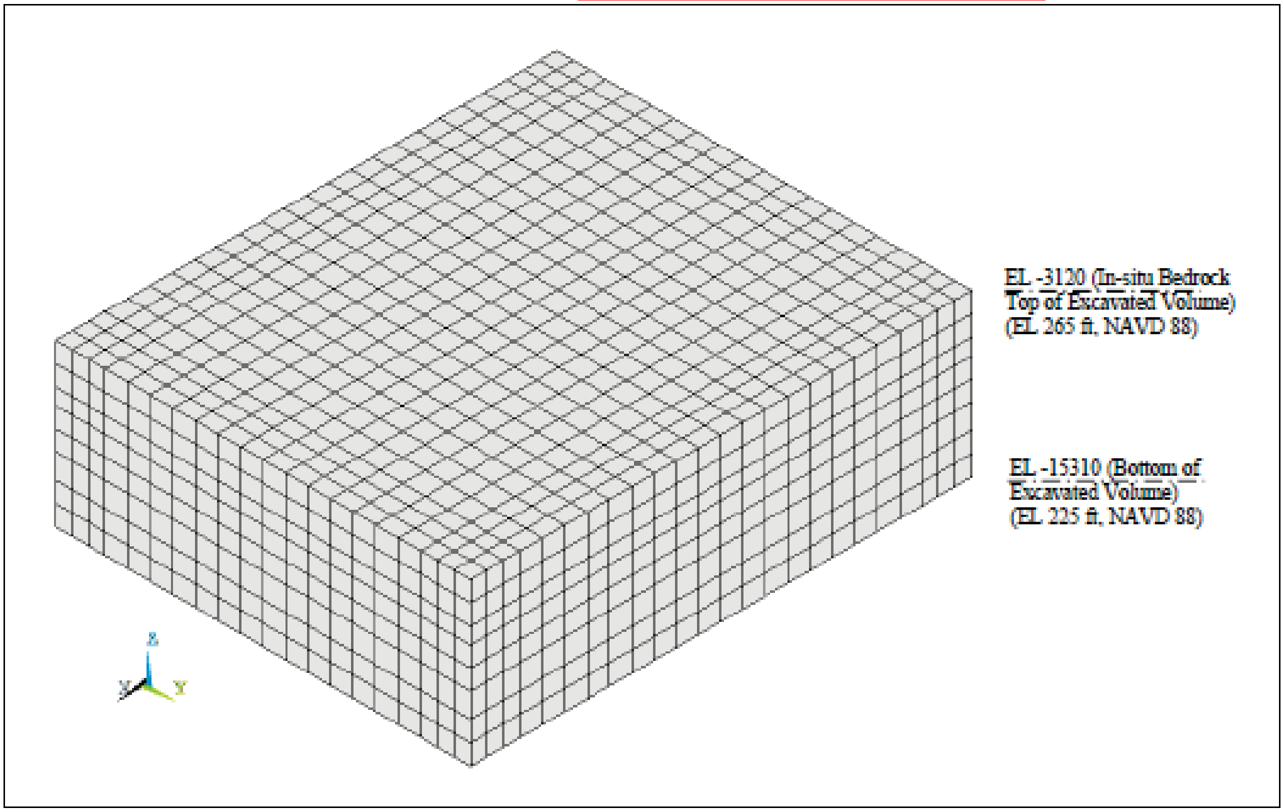


(b) Walls on Column Rows C1 and C5 (Unit: mm)

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

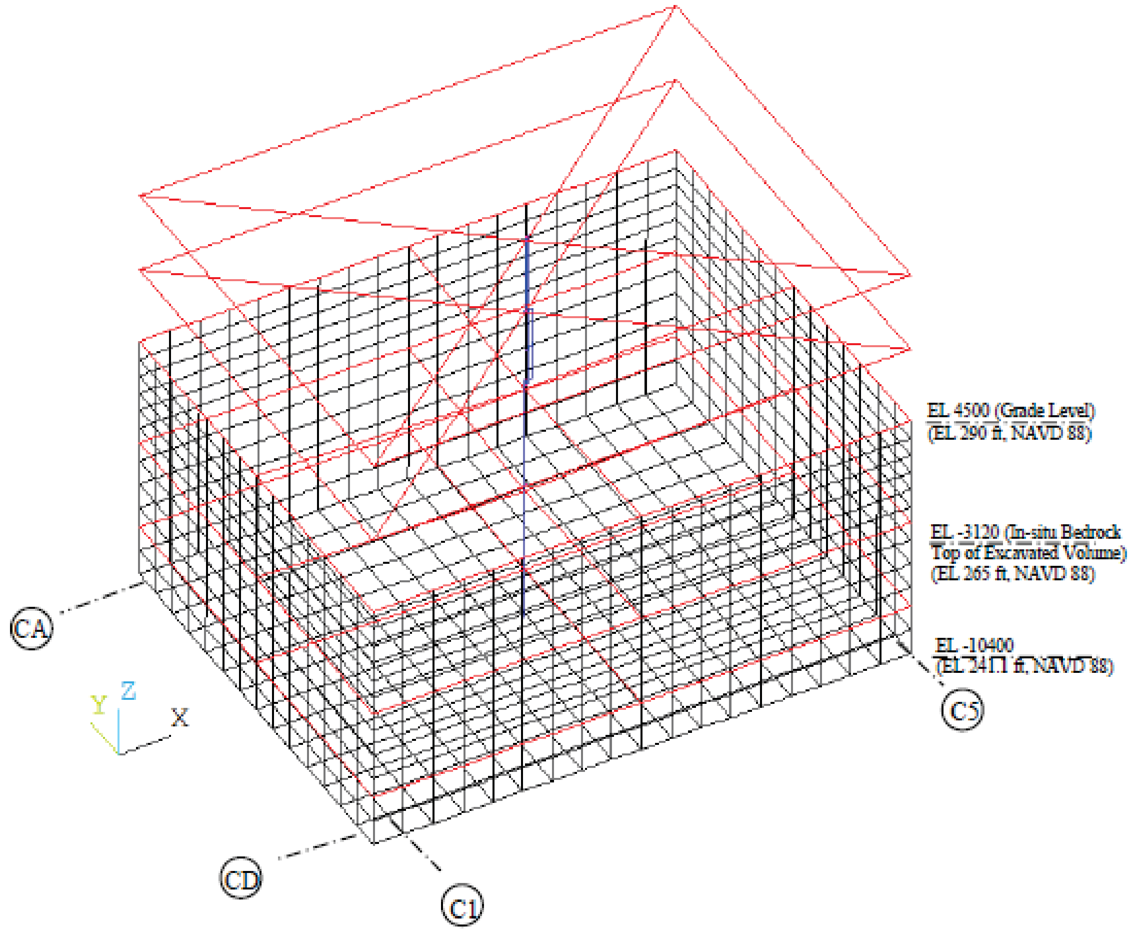
NAPS DEP 3.7-1

Figure 3A.16.3-212 **SASSI2010 Excavated Volume Solid Elements for the CB Partially Embedded Model**



NAPS DEP 3.7-1

Figure 3A.16.3-213a **Overview of SASSI2010 SSI CB Partially Embedded Model**

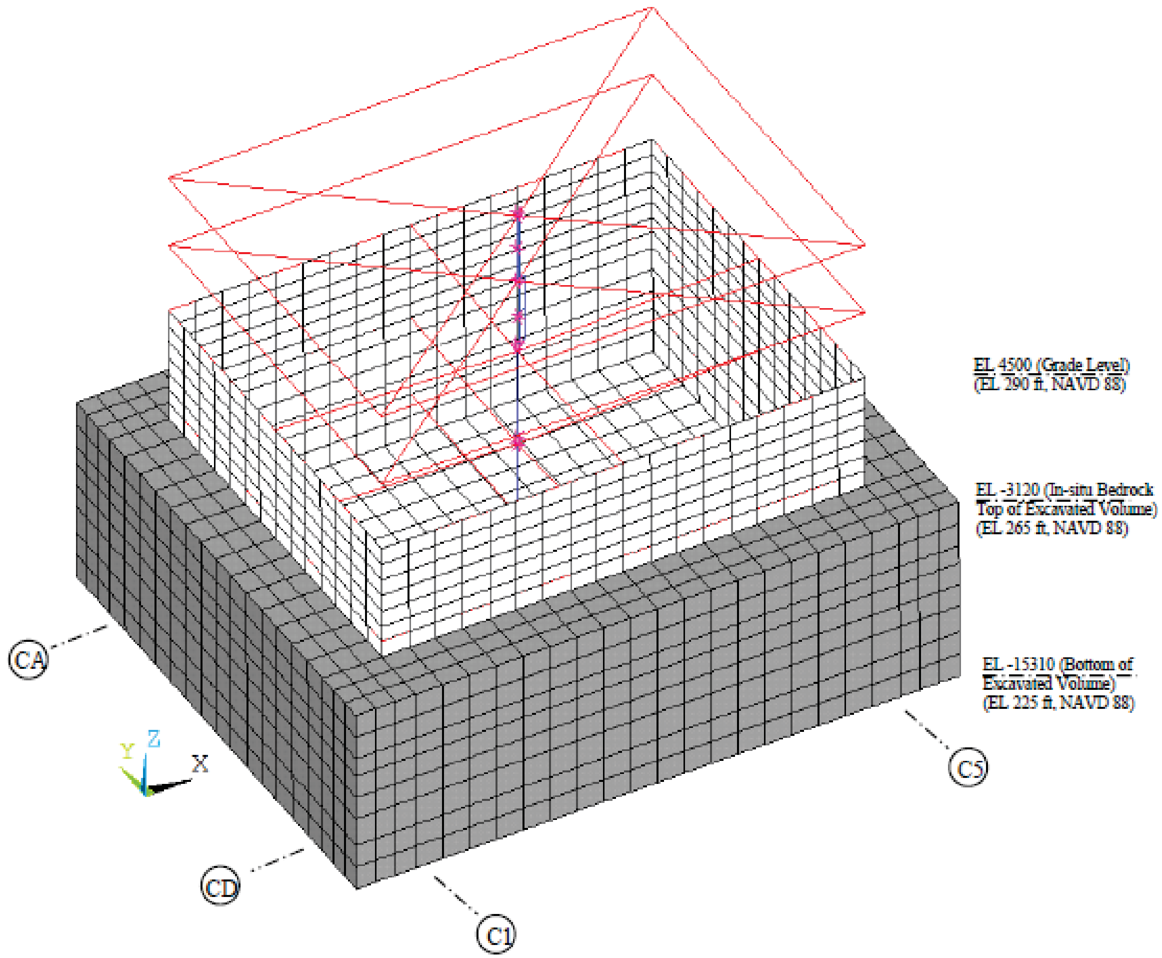


(a) Overview without Concrete Fill and Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams or outriggers indicated in red are installed at the floor levels.

NAPS DEP 3.7-1

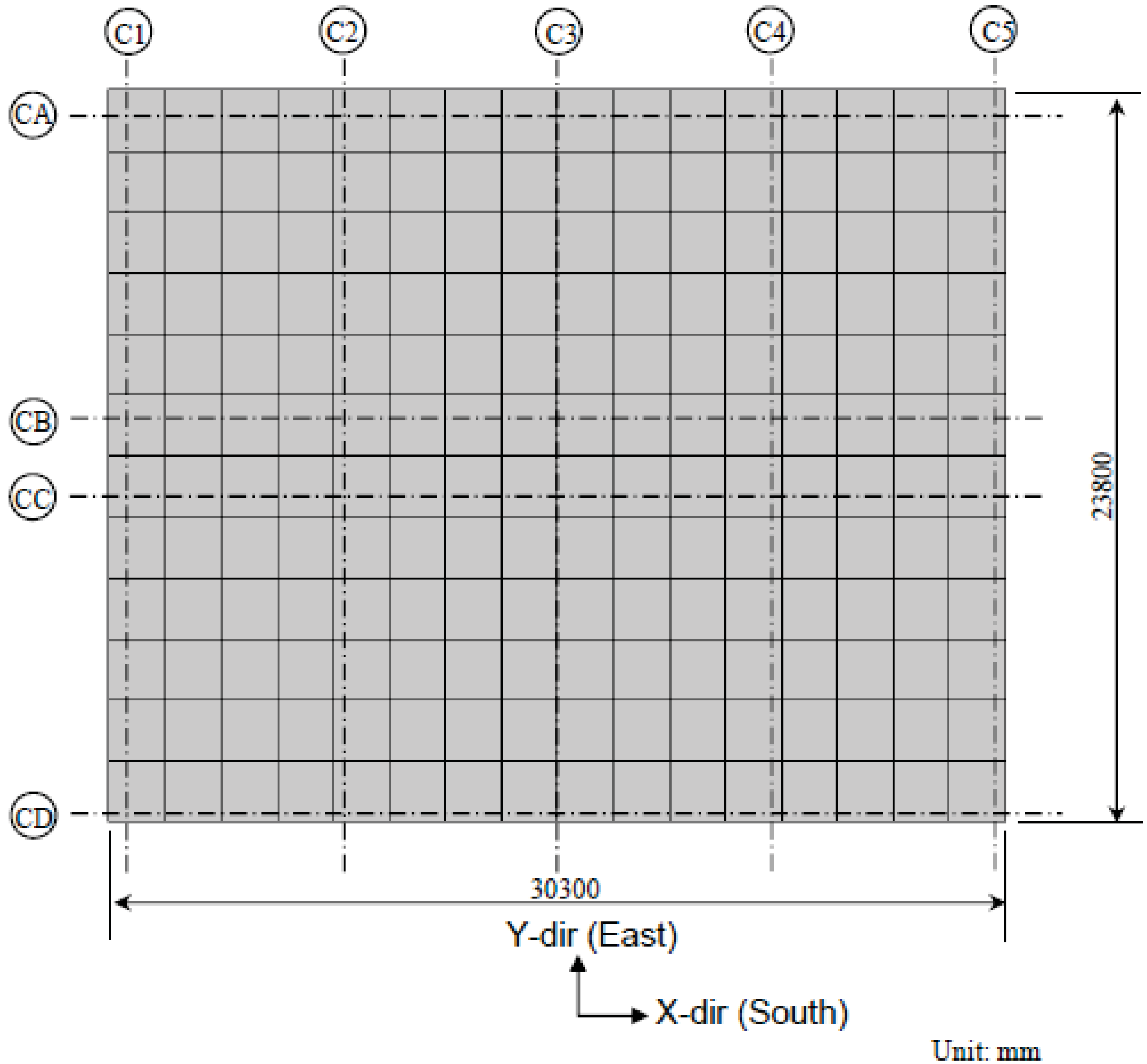
Figure 3A.16.3-213b **Overview of SASSI2010 SSI CB Partially Embedded Model**



(b) Overview with Concrete Fill without Excavated Volume

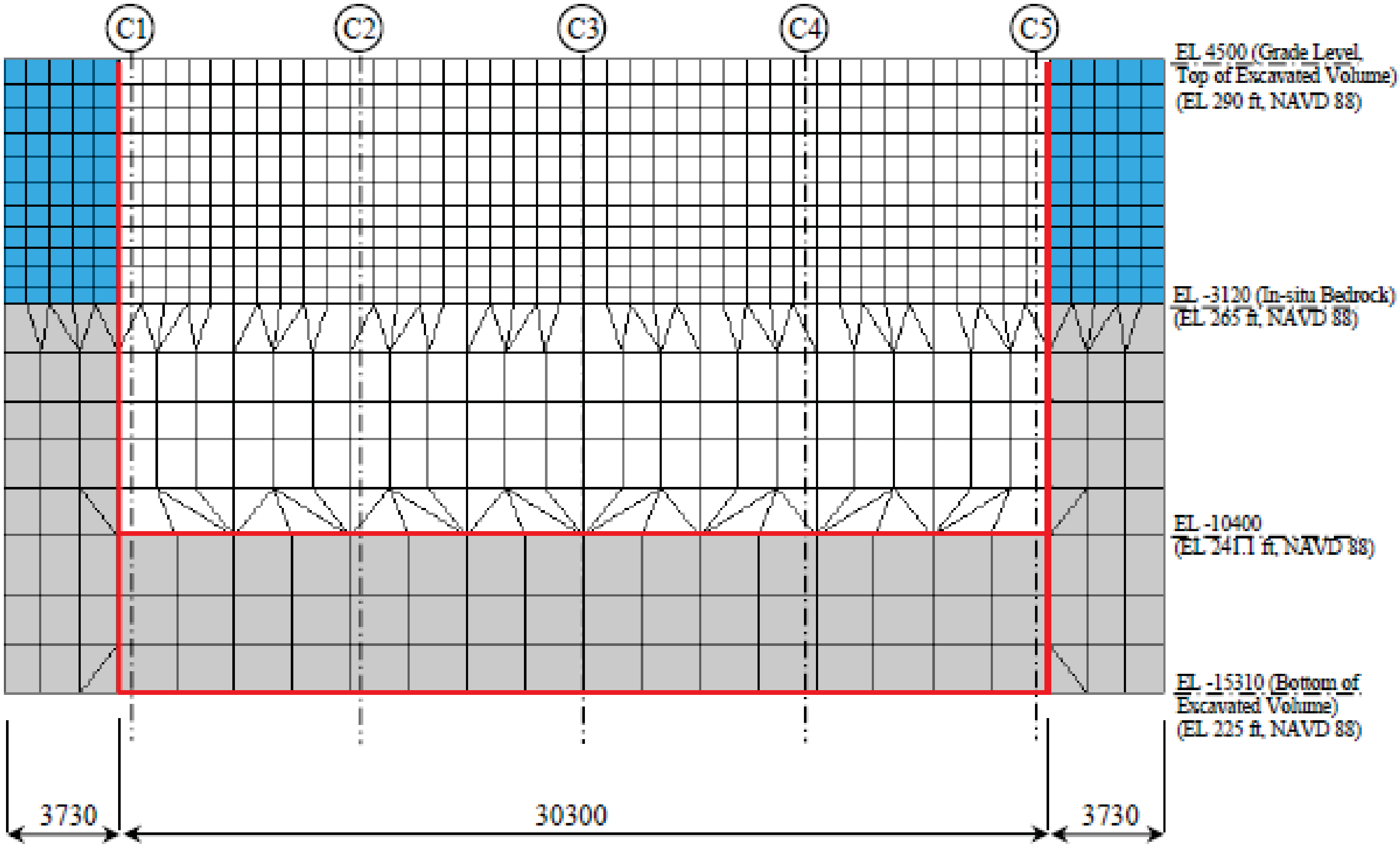
Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams or outriggers indicated in red are installed at the floor levels.

NAPS DEP 3.7-1 Figure 3A.16.3-214 **SASSI2010 Plate Elements for CB Basemat in the Fully Embedded Model**



NAPS DEP 3.7-1

Figure 3A.16.3-215a SASSI2010 Plate Elements for CB Exterior Walls in the Fully Embedded Model

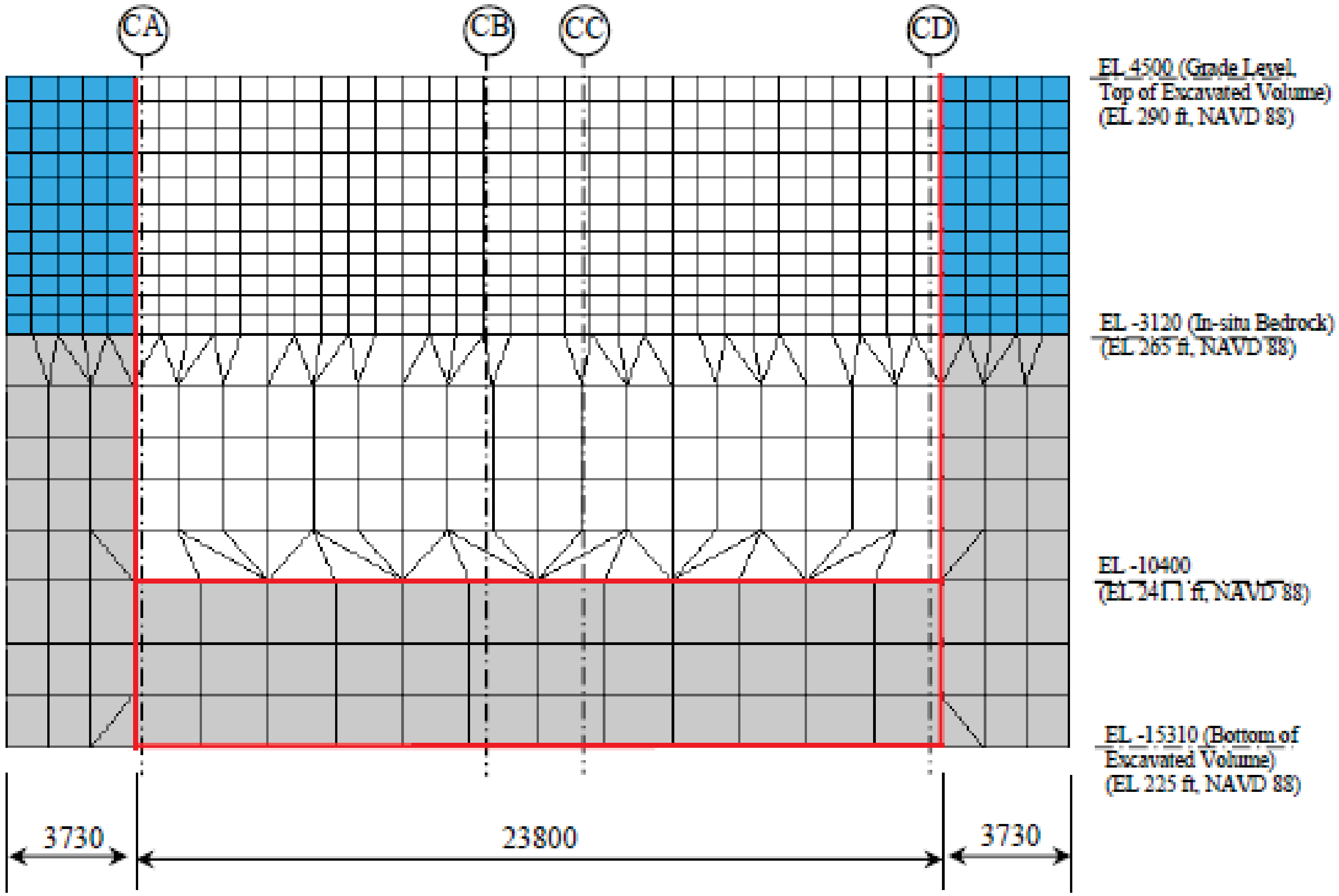


(a) Walls on Column Rows CA and CD (Unit: mm)

Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

NAPS DEP 3.7-1

Figure 3A.16.3-215b SASSI2010 Plate Elements for CB Exterior Walls in the Fully Embedded Model

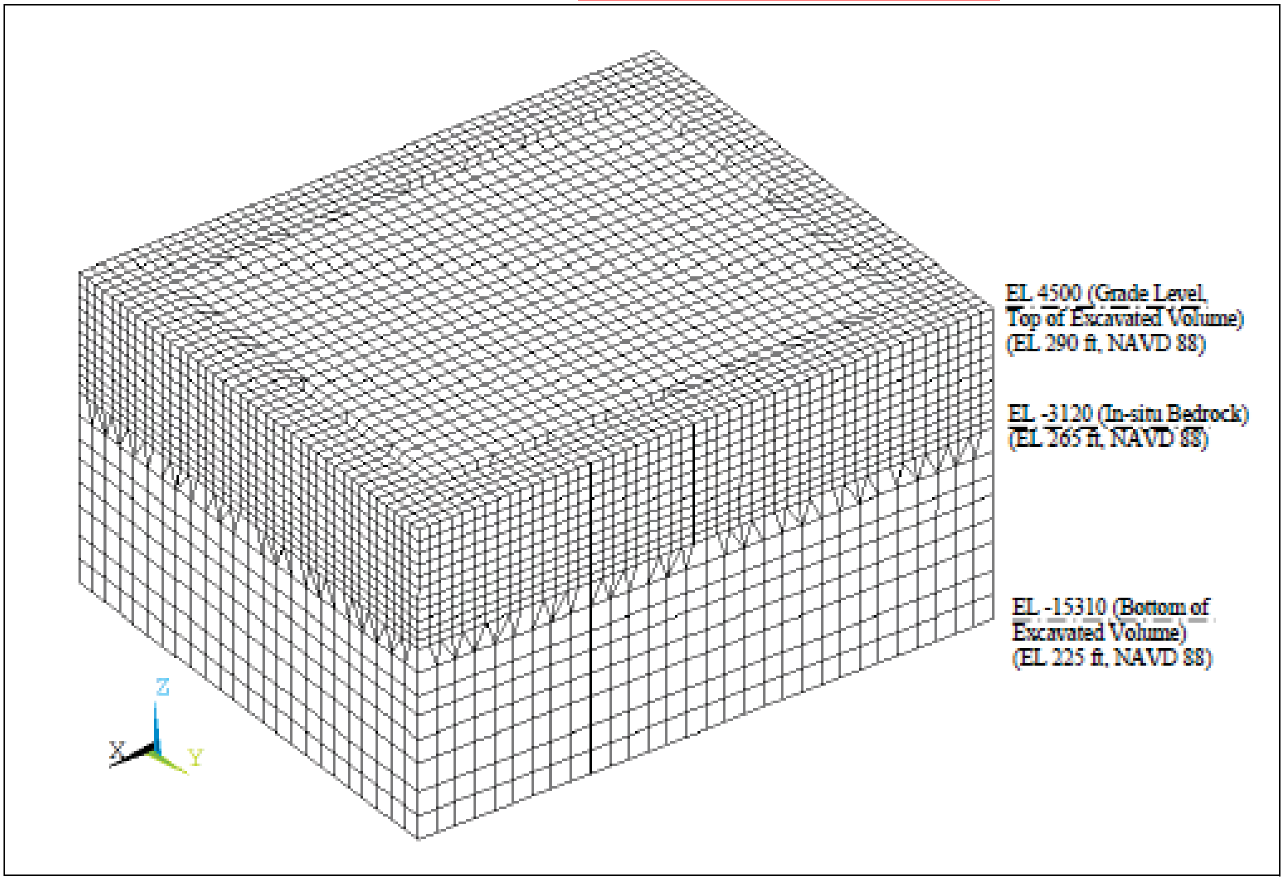


(b) Walls on Column Rows C1 and C5 (Unit: mm)

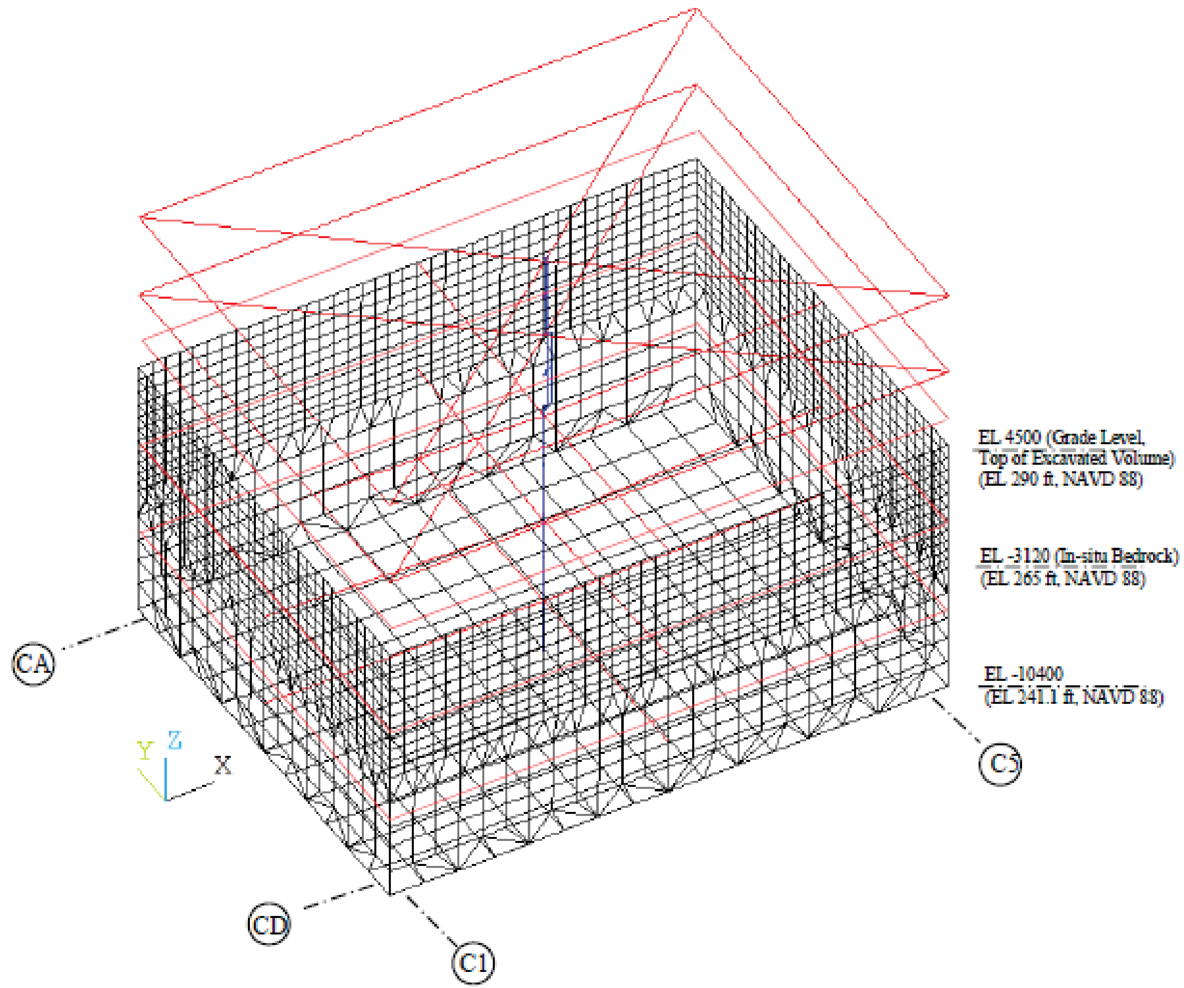
Note: The extent of double nodes used at the structural/soil interfaces is shown in red lines.

NAPS DEP 3.7-1

Figure 3A.16.3-216 **SASSI2010 Excavated Volume Solid Elements for the CB Fully Embedded Model**



NAPS DEP 3.7-1 Figure 3A.16.3-217a **Overview of SASSI2010 SSI CB Fully Embedded Model**

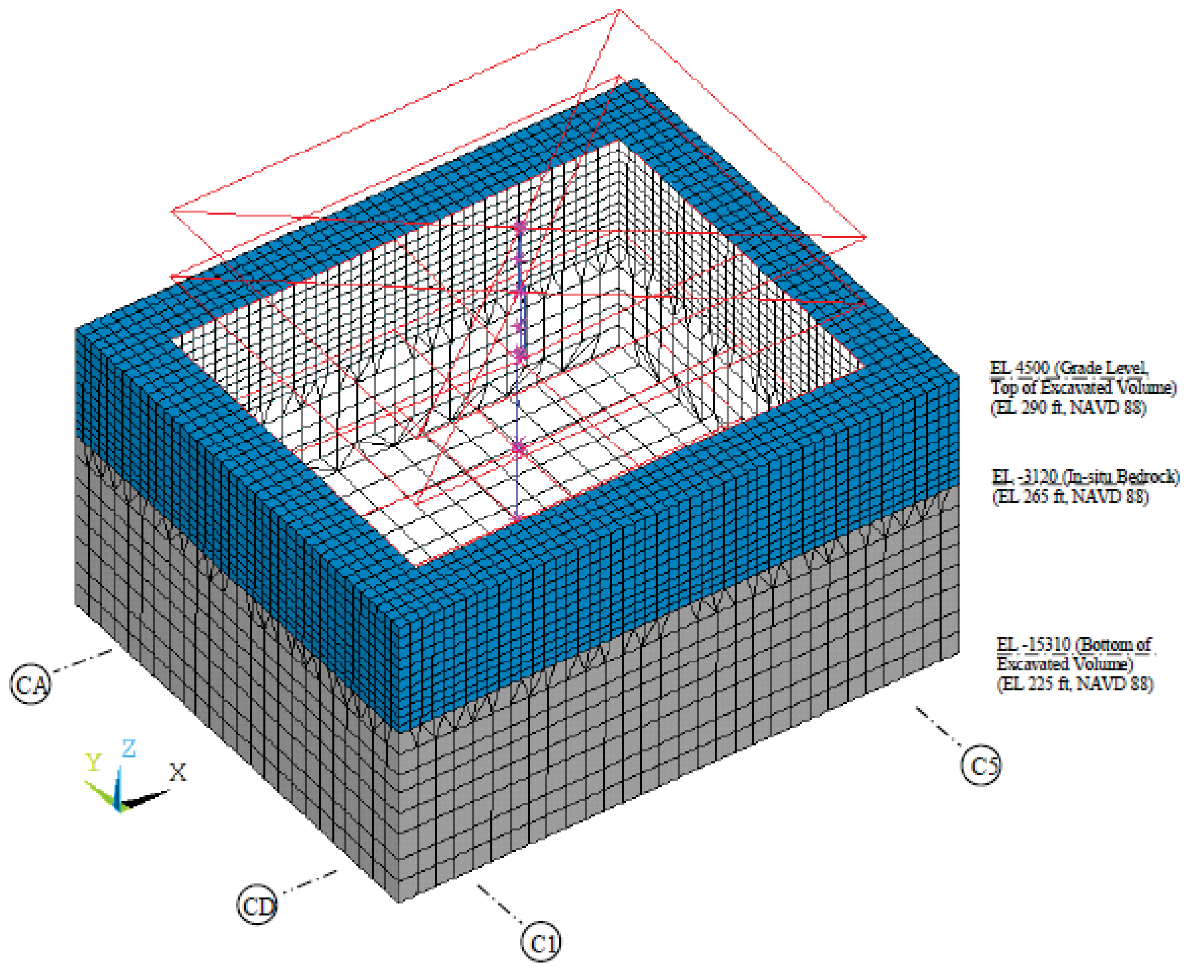


(a) Overview without Concrete Fill and Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams or outriggers indicated in red are installed at the floor levels.

NAPS DEP 3.7-1

Figure 3A.16.3-217b Overview of SASSI2010 SSI CB Fully Embedded Model

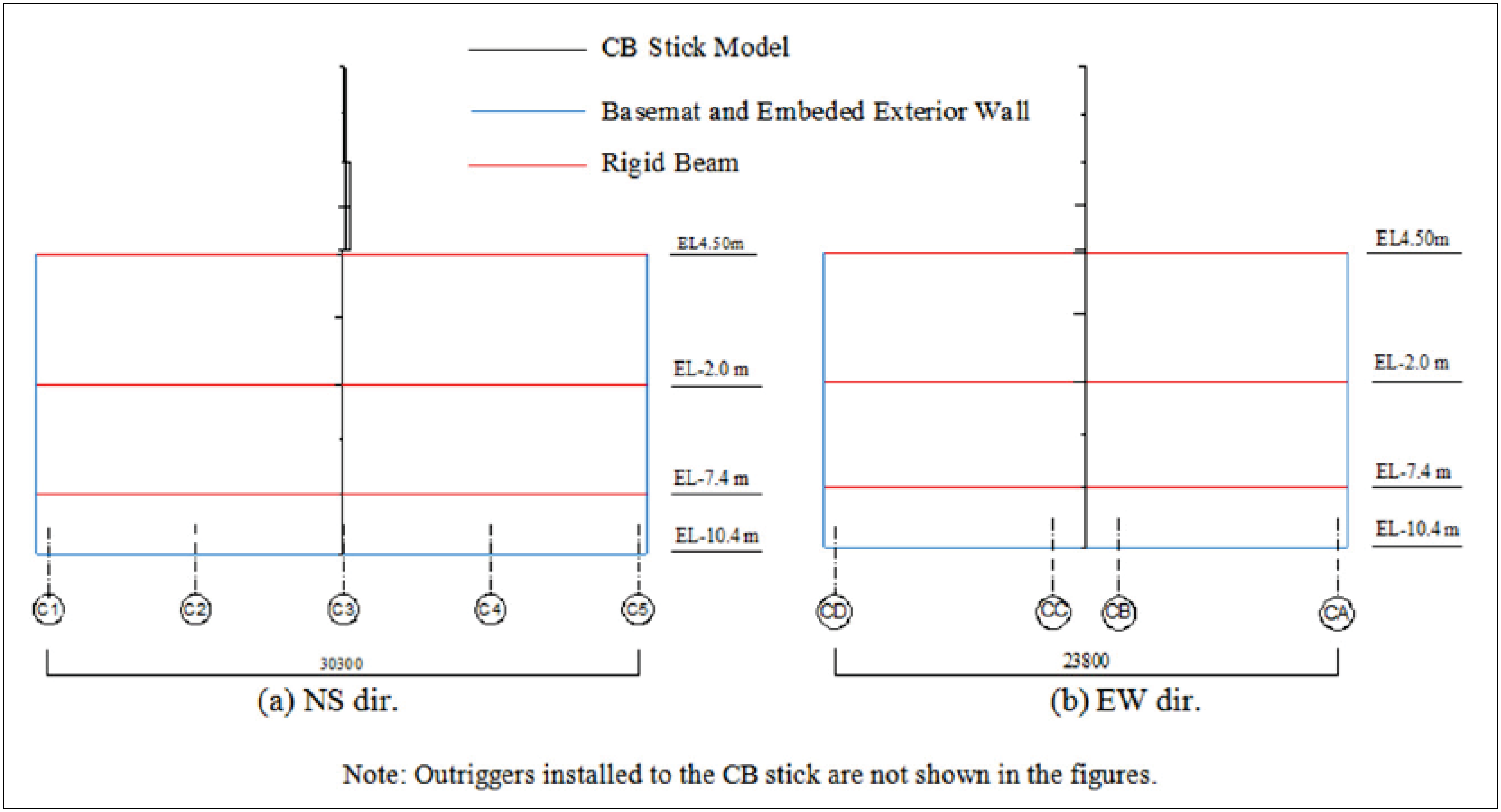


(b) Overview with Concrete Fill without Excavated Volume

Note: 1) Wall and basemat are modeled with shell elements.
2) Rigid beams or outriggers indicated in red are installed at the floor levels.

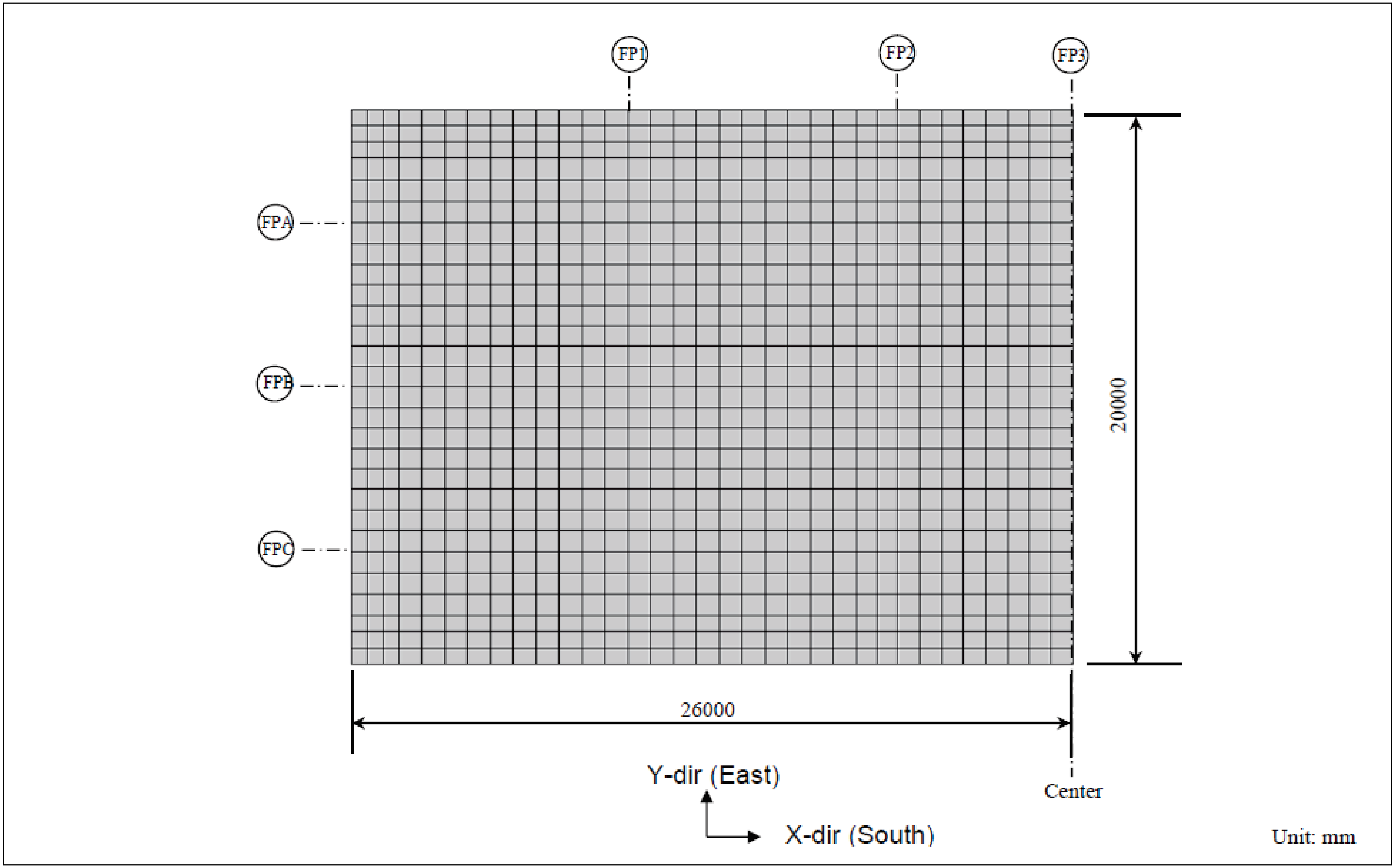
NAPS DEP 3.7-1

Figure 3A.16.3-218 **Connection between CB Stick Model and Foundation**



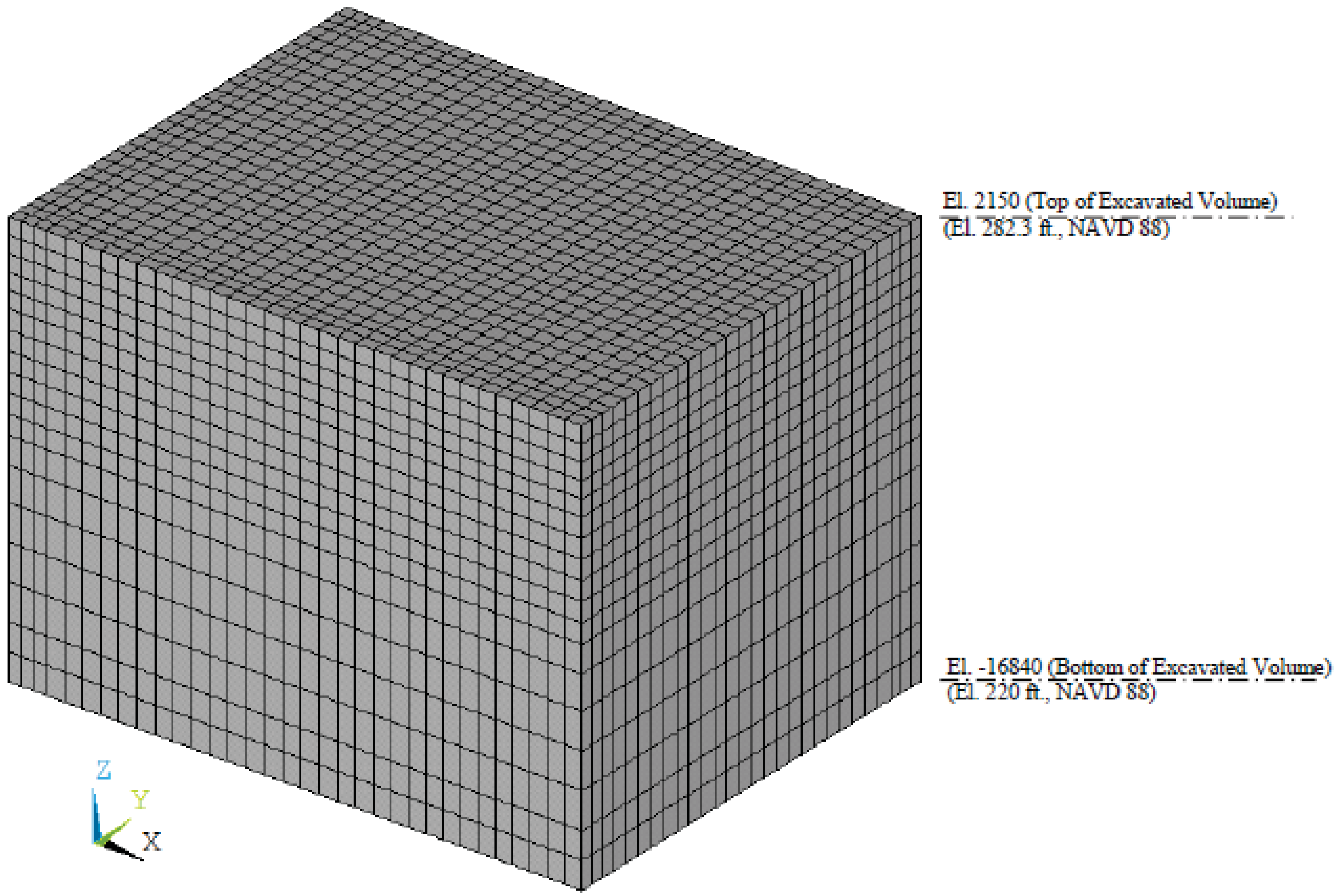
NAPS DEP 3.7-1

Figure 3A.16.3-219 **SASSI2010 Plate Elements for FWSC Basemat**

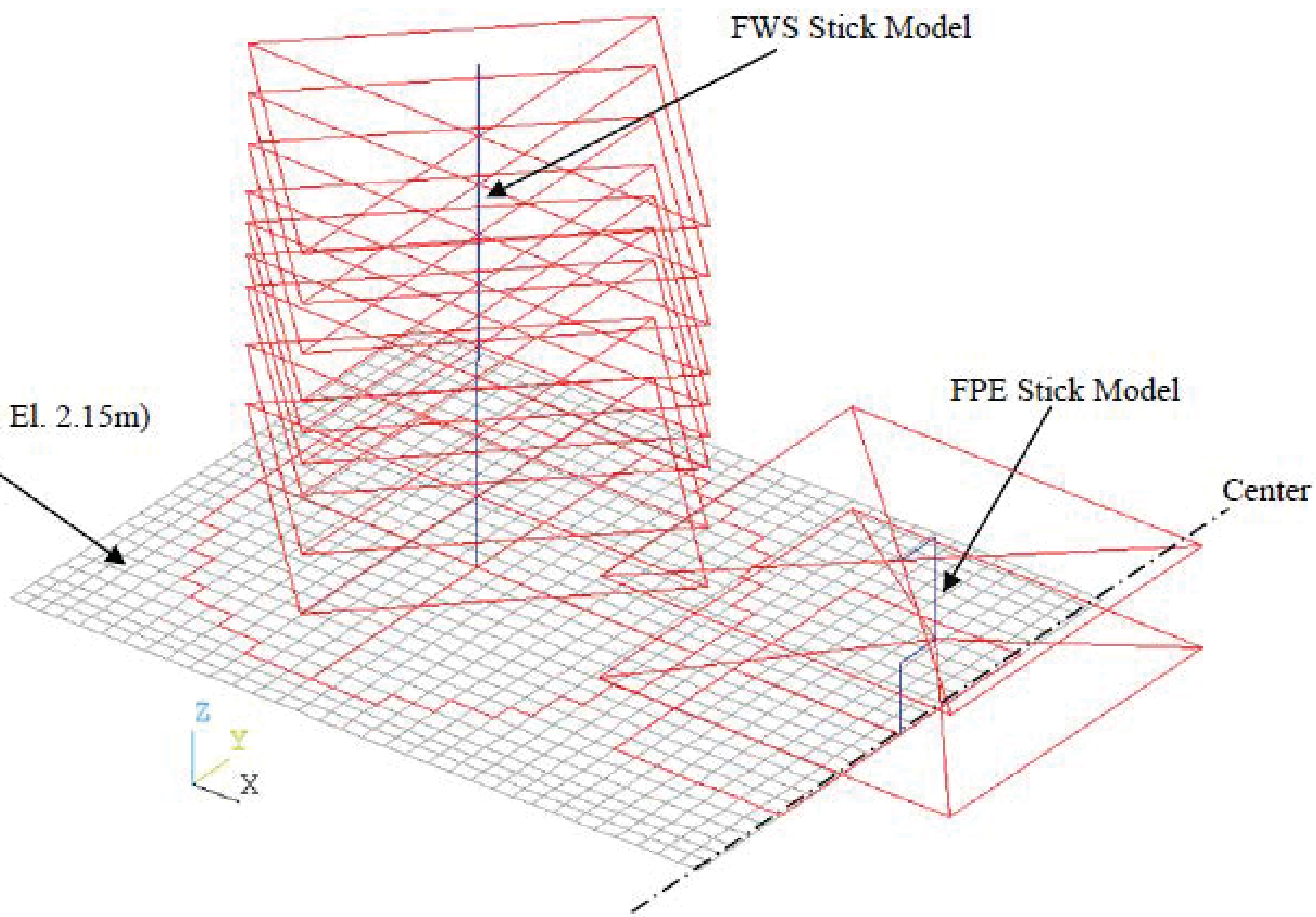


NAPS DEP 3.7-1

Figure 3A.16.3-220 **SASSI2010 Solid Elements for FWSC Concrete Fill and or Excavated Volume**



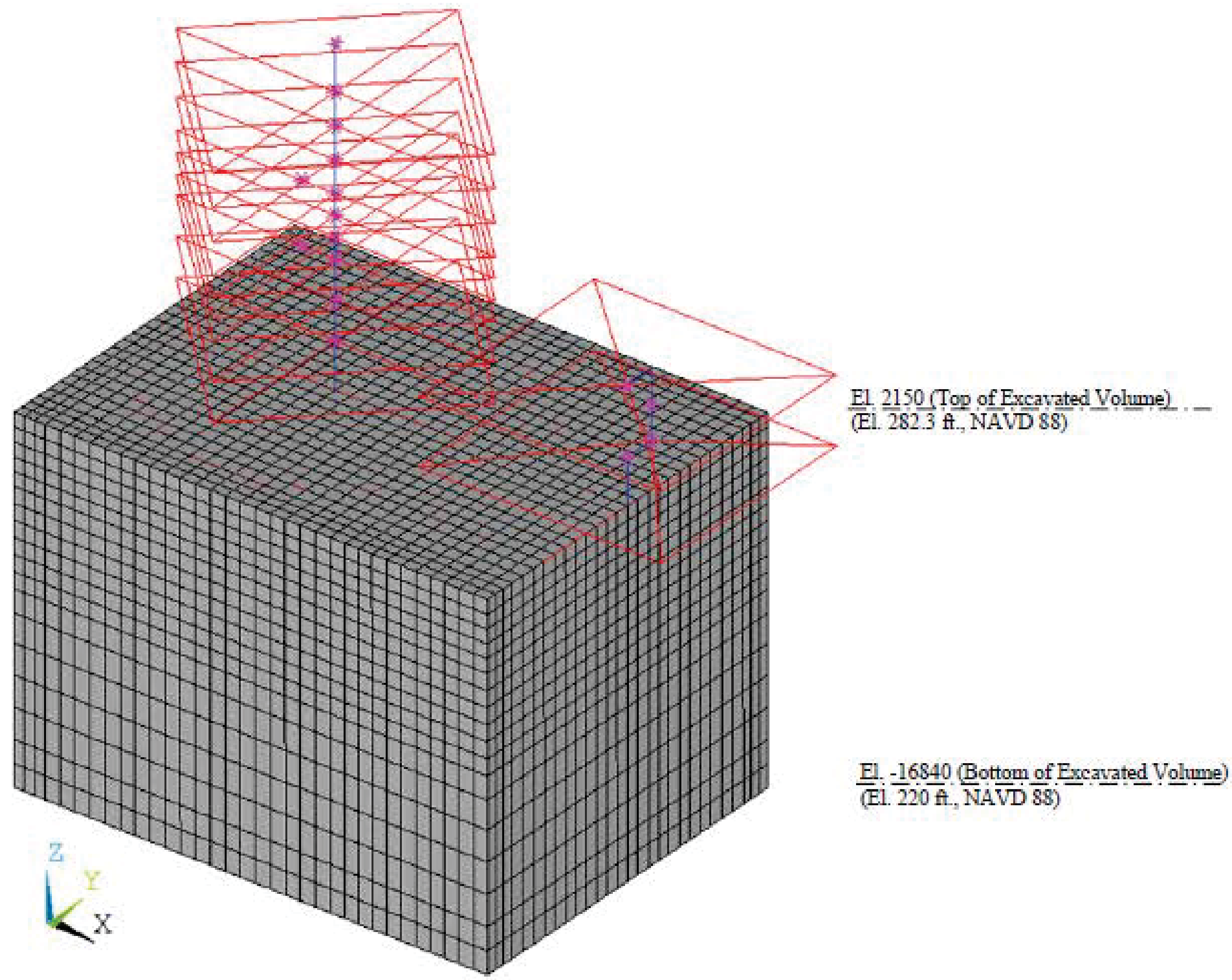
NAPS DEP 3.7-1 Figure 3A.16.3-221a **Overview of SASSI2010 SSI FWSC Half Model**



(a) Overview without Concrete Fill

Note: Basemat is modeled with shell elements.

NAPS DEP 3.7-1 Figure 3A.16.3-221b **Overview of SASSI2010 SSI FWSC Half Model**



(b) Overview with Concrete Fill

Note: Basemat is modeled with shell elements.

NAPS DEP 3.7-1

3A.17 Unit 3 SSI Analysis Results

The following sections present the results of the site-specific SSI analyses. The site-specific SSI analyses results are compared with the standard seismic design envelopes presented in DCD Sections 3A.1 through 3A.9. Comparisons are provided for maximum seismic structural loads and ISRS. The site-specific enveloping seismic responses are presented in Section 3A.18.

DCD Section 3A.8, which provides the standard design SSI analysis results, presents a number of typical SSI results to show the effect of different soil properties on seismic responses at selected locations in terms of acceleration response spectra and seismic forces. Certain effects that are discussed in DCD Section 3A.8, including soil stiffness, single envelope ground motion, layered sites, embedment, and lateral soil pressures, are evaluated in the site-specific SSI analyses using the Unit 3 conditions and, therefore, are not discussed separately in Section 3A.17. This section discusses the Unit 3 SSI analysis results for the RB/FB, CB, and FWSC with the Unit 3 site conditions.

3A.17.1 Effect of Soil Stiffness

Site-specific SSI analyses consider BE, UB, and LB properties to address the effects of variation of dynamic properties of subsurface materials. The partial and full embedment configurations considered in the RB/FB and CB SSI analyses provide responses that bound the effects of subgrade stiffness variations related to the soil separation, backfill horizontal extent, and groundwater level variations.

Results of site-specific SSI analyses presented in Section 3A.17.12 show that the response of RB/FB structures is insensitive to the subgrade stiffness variations.

The results of CB SSI analysis show that the saprolite and structural fill above the top of Zone III rock affect the response of the light and deeply embedded CB, resulting in shifts of the CB peak responses to higher frequencies. The effects of rock subgrade properties variations on the CB response are smaller. The SSI analysis of UB partial column profile provides bounding site-specific seismic demands on CB structure. The CB analysis of the partial column profiles yield bounding results with few exceptions in the ISRS results at lower (< 20 Hz) frequencies that are bounded by responses obtained from analyses of full column profiles.

The FWSC site-specific design basis is developed as envelope of responses obtained from the SSI analyses of FWSC stand-alone model and SSSI analyses of FWSC-CB combined model for LB, BE, and UB profiles representing the stiffness variation of subgrade below the FWSC basemat bottom elevation. These analyses consider the FWSC as a surface mounted structure and neglect the effect of engineered fill and in-situ soil locate above FWSC basemat elevation. The models used for the analyses explicitly model the concrete fill placed below the FWSC foundation. The FWSC stand-alone model neglects the structural fill placed around the concrete fill supporting the FWSC basemat. The FWSC-CB combined model includes the structural fill placed in the gap between the CB and FWSC and around the concrete fill below the FWSC foundation. The envelope of SSI and SSSI analyses results addresses the effects of the structural fill horizontal extent variations on the FWSC seismic response. The variation of subgrade properties affects the response of the FWSC by shifting the peak responses to higher frequencies as the stiffness of the subgrade increases.

The Unit 3 site-specific analyses for the SSSI effects described in Section 3A.17.11 consider variations of subgrade stiffness properties.

3A.17.2 Effect of Single Enveloping Ground Motion

Unit 3 site-specific SSI analyses are performed using the Unit 3 site-specific ground motion, as described for each of the Seismic Category I structures. See Sections 3A.17.12 through 3A.17.14 for details of the site-specific SSI analyses results.

3A.17.3 Effect of Updated Design of Reactor Shield Wall and Vent Wall

As explained in Section 3A.16.3, the dynamic models used for the site-specific SSI analyses of RB/FB are based on the standard design structural model used for the Case RU-4 that reflects the updated design of the reactor shield wall (RSW) and VW.

3A.17.4 Effect of In-fill Concrete Stiffness of Vent Wall and Diaphragm Floor

As described in Section 3A.16.3.1, site-specific SSI analyses of RB/FB use dynamic models with upper bound stiffness properties representing full (100 percent) stiffness contribution of the in-fill concrete to the stiffness of the concrete-filled steel structures to provide conservative seismic responses for the Unit 3 rock site with high frequency design

ground motion. Consistent with the approach used for the standard design to address the effects of variations of the in-fill concrete stiffness on the stiffness of the VW and D/F, site-specific sensitivity analyses are performed on two RB/FB dynamic models with reduced stiffness properties that consider a 50 percent and 0 percent stiffness contribution of the in-fill concrete to the concrete-filled steel structures. See Section 3A.17.9 for the results of the evaluation of the effects of structural stiffness variations on the seismic response of the RB/FB at Unit 3 site.

3A.17.5 Effect of Loss-of-Coolant Accident (LOCA) Flooding

The responses obtained from the site-specific SSI analyses using the models representing the plant normal operation conditions envelope the effects of LOCA flooding inside the containment. The additional water mass due to LOCA flooding increases the dynamic mass of the RB/FB, shifting the structural frequencies and the peak responses of the building away from the amplified region of the Unit 3 ground motion at high frequencies.

3A.17.6 Effect of Layered Sites

Site-specific SSI analyses use Unit 3 layered subgrade properties.

3A.17.7 Effect of Embedment

The RB/FB and CB SSI analyses consider partial and full embedment configurations. The results of the SSI analyses indicate that the responses of the RB/FB at the Unit 3 site are generally insensitive to the embedment. The differences between the responses obtained from the RB/FB SSI analyses of the partially and fully embedded models are mainly due to the differences in the energy content of the input motions used for the SSI analyses of the partial and full column profiles at structural frequencies. The results of SSI analysis show that the saprolite and structural fill above the top of the Zone III rock affect the response of the light and deeply embedded CB resulting in shifts of the CB peak responses to higher frequencies. The dissipation of energy in the engineered fill material also reduces the CB response. Section 3A.17.12 discusses how these two different embedment configurations affect the RB/FB and CB seismic response. The lower SSI damping of the partial rock column subgrade profiles and the higher energy of the partial column ground motion at frequencies close to the CB structure natural frequencies resulted in higher responses of the CB.

The site-specific SSI analyses consider the FWSC as a surface mounted structure and neglect the effects of the in-situ saprolite and structural fill placed around the FWSC.

3A.17.8 Effect of Lateral Soil Pressures

Section 3A.17.12.4 presents the results of the site-specific SSI analyses of the RB/FB for the maximum seismic lateral pressures on the RB/FB below-grade exterior walls. Comparison of the results from analysis Cases 1 through 6 in Table 3A.15-201 show that the variation of the soil properties has a small effect on the calculated dynamic pressures on the exterior walls. Section 3A.17.13.5 presents the results of the site-specific SSI analyses of the CB for the maximum seismic lateral pressures on the CB below-grade exterior walls. Comparisons of the results from analysis Cases 7 through 12 in Table 3A.15-202 show that the variation of the soil properties has a small effect on the calculated dynamic pressures on the exterior walls.

3A.17.9 Effect of Concrete Cracking

Site-specific sensitivity evaluations are performed of the effects of concrete cracking on the response of the reinforced concrete members and the out-of-plane vibrations of flexible slabs and walls. These evaluations are based on the results of site-specific sensitivity analyses of models representing dynamic properties of reinforced concrete structures under fully cracked conditions when all of the concrete structural members are considered fully cracked. SSE damping values are assigned to the concrete and steel structural members in the models used for these sensitivity SSI analyses in conjunction with reduced (cracked concrete) stiffness properties to represent the higher dissipation of energy in the structures when subjected to high stresses corresponding to the fully cracked concrete condition.

The shear and bending stiffness properties of the reinforced concrete members in the models used for the concrete cracking evaluations are reduced by 50 percent in accordance with ASCE 43-05. Since the 50 percent reduced flexural stiffness of the walls and slabs also lowers their natural frequencies of out-of-plane vibrations by $\sqrt{2}$, additional SDOF oscillators are added to the lumped mass stick models to adequately capture all modes of out-of-plane vibration with frequencies ranging from 35 Hz ($\frac{50}{\sqrt{2}}$ Hz) to 50 Hz under fully cracked conditions.

3A.17.9.1 **Effect of Structural Stiffness Variations on RB/FB**

Specifically for the RB/FB, a sensitivity study is performed to evaluate effects of structural stiffness variations on the seismic response of the RB/FB at Unit 3 site including the effects of:

- Reduced stiffness of the reinforced concrete members due to concrete cracking, and
- Contribution of in-fill concrete stiffness on the dynamic properties of the concrete-filled steel structures discussed in Section 3A.17.4.

The study is based on the results of a set of twelve sensitivity SSI analyses (Cases S1 to S12 in Table 3A.15-201) of the following two structural models with reduced stiffness and SSE damping:

CR00 Model representing:

- Fully cracked reinforced concrete structures with 50 percent reduced shear and bending stiffness
- No (0 percent) in-fill concrete contribution to the stiffness of the concrete-filled VW and D/F steel structures

CR50 Model representing:

- Fully cracked reinforced concrete structures with 50 percent reduced shear and bending stiffness
- 50 percent in-fill concrete contribution to the stiffness of the concrete-filled VW and D/F steel structures

These evaluations of the effects of structural stiffness variations on the seismic response of RB/FB show that the site-specific design basis SSI analyses of the **UC100 Model** with full (uncracked concrete) stiffness and OBE damping (analyses Cases 1 to 6 in Table 3A.15-201) provide site-specific seismic demands on the RB/FB reinforced concrete structures and site-specific design ISRS that envelope concrete cracking effects with a few exceedances that are small and have a local effect.

Based on the results of the sensitivity SSI analyses on the RB/FB models with reduced stiffness properties (analyses Cases S1 to S12 in Table 3A.15-201), the enveloping site-specific seismic load demands and site-specific design ISRS in Section 3A.18.2 are adjusted to bound effects of structural stiffness variations. Amplified input seismic loads are

the site-specific evaluation of the RB/FB structures that bound the small local exceedances of:

- Seismic loads on the RSW, VW, and pedestal structures (an example is shown in Figure 3A.17.9.1-201)
- Out-of-plane vertical seismic loads on the RB/FB slabs and walls presented in Tables 3A.17.9.1-201 and 3A.17.9.1-202

To address effects of structural stiffness variations, the site-specific design and qualification of RB/FB equipment and components will use enhanced ISRS that envelope all significant (>10 percent) peak exceedances of site-specific design ISRS observed in the results of the sensitivity analysis cases (Cases S1 through S12 in Table 3A.15-201) for frequencies below 50 Hz. The vertical ISRS used for site-specific design and qualification of equipment and components supported by flexible slabs are developed as follows:

1. Site-specific design ISRS in Section 3A.18.2 for out-of-plane slab response are grouped and enveloped based on the SDOF oscillator slab region grouping.
2. The ±15 percent broadened design ISRS developed in Step 1 are compared to the envelope of the ISRS representing the out-of-plane responses of the slab under fully cracked conditions obtained from results of sensitivity analysis Cases S1 through S12 in Table 3A.15-201.
3. If comparisons in Step 2 show that the cracked slab ISRS exceed the design ISRS at frequencies below 50 Hz by more than 10 percent, the design ISRS are adjusted to bound the exceedances.

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Table 3A.17.9.1-201 **Exceedances of Out-of-Plane Loads on RB/FB Flexible Slabs Due to Structural Stiffness Variations**

Elev. (m)	Location	Equivalent Average Vertical Acceleration (g) PE CR50	PE CR00	FE CR50	FE CR00	NA3 Enveloping	Standard Design
-6.4	RCCV-Pedestal	0.46	0.45	0.59	0.60	0.60	0.63
	RB-RCCV	0.59	0.59	**0.66**	0.65	0.57	0.71
	FB	0.55	0.55	0.57	**0.58**	0.50	-
17.5	MS Tunnel	0.79	0.77	1.15	1.13	1.74	1.06
	RCCV	0.64	0.63	0.84	0.78	0.94	0.78
	D/F	0.87	1.64	1.23	**2.38**	1.53	1.84

Note: The shaded values are exceedances from the enveloping out-of-plane loads presented in Table 3A.18.1.1-203. Maximum values presented in bold.

NAPS DEP 3.7-1

Table 3A.17.9.1-202 **Exceedances of Out-of-Plane Loads on RB/FB Flexible Walls Due to Structural Stiffness Variations**

Elev. (m)	Column Line Location	Direction	Equivalent Horizontal Acceleration (g) PE CR50	PE CR00	FE CR50	FE CR00	NA3 Enveloping	Standard Design
30.5	R1 and R7s	NS	0.50	0.51	0.57	0.58	-	-
	RA and RGs	EW	0.54	0.54	0.51	0.51	-	-

Note: The shaded values are exceedances from the enveloping out-of-plane loads presented in Table 3A.18.1.1-204.

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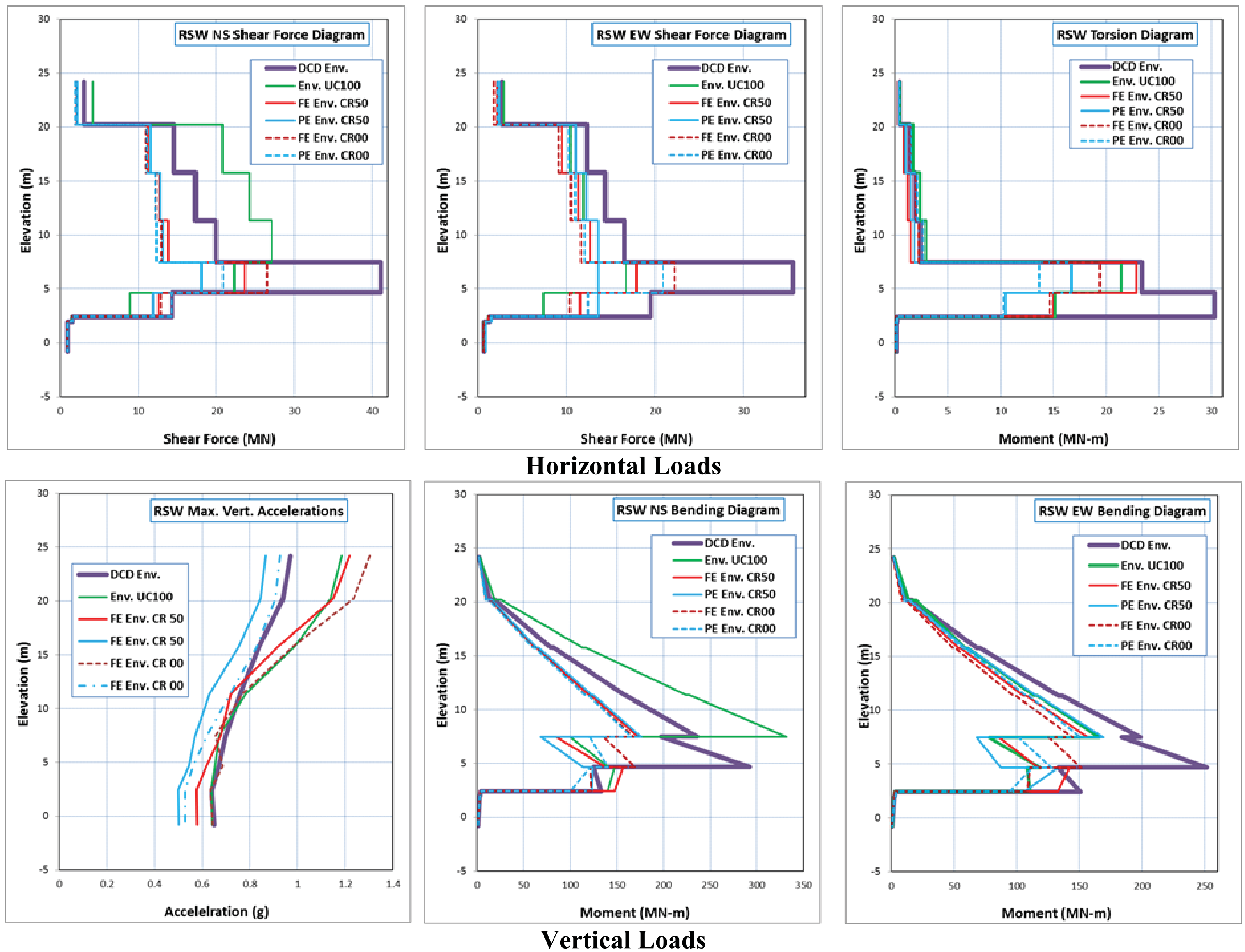
Figure 3A.17.9.1-201 **Exceedances of RSW Seismic Loads Due to Structural Stiffness Variations**



3A.17.9.2 Effect of Structural Stiffness Variations on CB

Site-specific sensitivity studies are performed to evaluate the effects of structural stiffness variations on the seismic response of the CB based on comparing CB responses with:

- UB stiffness when all members of the CB are uncracked (analysis Cases 1 through 12 in Table 3A.15-202)
- 50 percent reduced shear and bending stiffness of the reinforced concrete members due to concrete cracking (analysis Cases S1 through S6 in Table 3A.15-202)

These evaluations consider the effects of concrete cracking on the response of the CB reinforced concrete members and the out-of-plane vibrations of the flexible slabs. The effects of the concrete cracking on the site-specific seismic demands on the CB structure are evaluated by comparing the enveloping maximum member forces and accelerations results from the analyses of the UC_{SSE} models with full stiffness properties and SSE damping (analysis Cases 7 through 12 in Table 3A.15-202) with the corresponding results obtained from the sensitivity analyses of the CR_{SSE} models with reduced (cracked concrete) stiffness properties (analyses Cases S1 through S6 in Table 3A.15-202). The effect of the concrete cracking on the CB site-specific ISRS are evaluated by comparing the 5 percent damped ISRS results from the analyses of the UC_{OBE} models with full stiffness properties and OBE damping (analysis Cases 1 through 6 in Table 3A.15-202) with the ISRS obtained from the sensitivity analysis cases S1 to S6 that are performed on the CR_{SSE} models.

Results show that the site-specific CB SSI analyses of the models with full (uncracked concrete) stiffness and SSE damping provide site-specific seismic demands on the CB structure that envelope the effects of concrete cracking. Only the local out-of-plane loads on some of the CB slabs exceed the loads obtained from the analyses of the CB model with full stiffness and SSE damping, as shown on Table 3A.17.9.2-201.

The site-specific SSI analyses of the CB model with full (uncracked concrete) stiffness properties and OBE damping values provide site-specific design ISRS that envelope the concrete cracking effects on the design and qualification of CB equipment and components. Exceptions are the small sharp peak exceedances observed in some of the SDOF oscillator ISRS. To address these exceedances, the vertical

ISRS used for site-specific design and qualification of equipment and components supported by CB flexible slabs will be developed as follows:

1. Site-specific design ISRS in Section 3A.18.2 for out-of-plane slab responses are grouped and enveloped based on the SDOF oscillator slab region grouping.

2. The ±15 percent broadened design ISRS developed in Step 1 is compared to the envelope of the ISRS representing the out-of-plane responses of the slab under fully cracked conditions, which are obtained from the results of sensitivity analysis Cases S1 through S6 in Table 3A.15-202.

If comparisons in Step 2 show that the cracked slab ISRS exceed the design ISRS at frequencies below 50 Hz by more than 10 percent, the design ISRS are adjusted to bound the exceedances.

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Table 3A.17.9.2-201 **Vertical Out-of-Plane Loads on Flexible Slabs – CB**

		Slab Equivalent Out-of-Plane Acceleration Load (g)							
		Reduced Stiffness CR_{SSE} Model							
		Partial Column			**Full Column**				
EL (m)	**Location**	**BE**	**UB**	**LB**	**BE**	**UB**	**LB**	**NA3 Envelope (UC_{SSE})**	**Max. Exceedance**
13.80	Roof	1.37	*1.53*	1.19	1.02	1.17	0.87	1.52	0%
9.06*)	CA-CD	1.07	*1.19*	0.94	0.85	0.97	0.73	1.21	-
4.65	CA-CD	0.91	*1.03*	0.80	0.72	0.82	0.60	0.91	13%
-2.00	CA-CD	0.64	*0.69*	0.60	0.54	0.62	0.47	0.66	5%

Note: *) An enveloping load is used for all of the slabs regions at EL 9.06 m
The shaded values are exceedances from the UC_{OBE} model enveloping results.
Values in italic are the maximum values obtained from the analyses of the CR_{SSE} model

3A.17.9.3 Effect of Structural Stiffness Variations on FWSC

Site-specific sensitivity evaluations of the effects of concrete cracking on the SSI response of the FWSC are performed based on the comparisons of the results obtained from:

- Models UC_{OBE} and UC_{SSE} with UB structural stiffness properties representing the condition when all members of the FWSC reinforced concrete structures are assigned full (uncracked concrete) stiffness properties (analysis Cases 1 through 9 in Table 3A.15-203)
- Model CR_{SSE} with 50 percent reduced shear and bending stiffness properties representing the condition when all members of the FWSC reinforced concrete structures are fully cracked (analyses Cases S1 to S4 in Table 3A.15-203)

These evaluations consider the effects of concrete cracking on the response of the FWSC reinforced concrete members and the out-of-plane vibrations of the FWS roof. The effects of the concrete cracking on the site-specific seismic demands on the FWSC structures are evaluated by comparing the results from the analyses of the CR_{SSE} model with reduced (cracked concrete) stiffness properties with the enveloping maximum member forces and accelerations results from the site-specific SSI analyses of the FWSC UC_{SSE} model with full stiffness properties and SSE damping (analysis Cases 7 to 9 in Table 3A.15-203). The effects of concrete cracking on the FWSC site-specific ISRS are evaluated by comparing the 5 percent ISRS obtained from the sensitivity analysis cases performed on the CR_{SSE} models with the 5 percent damped broadened and valley-filled ISRS obtained as an envelope of results from the SSI analyses of the FWSC UC_{OBE} models with full stiffness properties and OBE damping (analysis Cases 1 to 6 in Table 3A.15-203).

Results of these evaluations show that the site-specific SSI analyses of the models with full (uncracked concrete) stiffness and SSE damping provide site-specific seismic demands on the FWS structures that envelope the effects of concrete cracking. The evaluations also show that cracking of the concrete amplifies the horizontal load demands on the FPE structure and the local vertical out-of-plane load on the FPE roof. The site-specific evaluations of the FWSC structures address the effects

of concrete cracking by using amplified input seismic loads that bound the exceedances of:

- Horizontal hydrodynamic load from the water contained in the FWSC tanks (Table 3A.17.9.3-201)
- Horizontal loads on the FPE structure (Figure 3A.17.9.3-201)
- Out-of-plane vertical load on the FPE roof (Table 3A.17.9.3-202)

The site-specific SSI analyses of the FWSC model with full (uncracked concrete) stiffness properties and OBE damping values provide site-specific design ISRS that, in general, envelope the concrete cracking effects. The site-specific design basis ISRS in Section 3A.18.2 of the lumped mass and SDOF roof oscillator at the FPE top are adjusted to bound the effects of concrete cracking based on the ISRS results of the sensitivity analyses.

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Table 3A.17.9.3-201 Maximum Accelerations of SDOF Oscillators for UB – FWSC

SDOF Oscillator				Acceleration (g)					
				UB Profile Analysis with Surface Input Motion		UB Profile Analysis with Deep Input Motion			
Elev. (m)	Node No.	Description	Direction	CR_{SSE} ~~Model~~	UC_{OBE} Model	CR_{SSE} ~~Model~~	UC_{SSE} Model	UC_{SSE} ~~Model~~ SSI Envelope	Standard Design
8.81	30	FWS Water Impulsive Mode	NS (X)	0.51	0.72	0.76	0.87	0.87	1.10
			EW (Y)	0.59	0.64	1.10 (shaded)	1.01	1.01	1.40

Note: Shaded cell identifies an exceedance due to concrete cracking effects.

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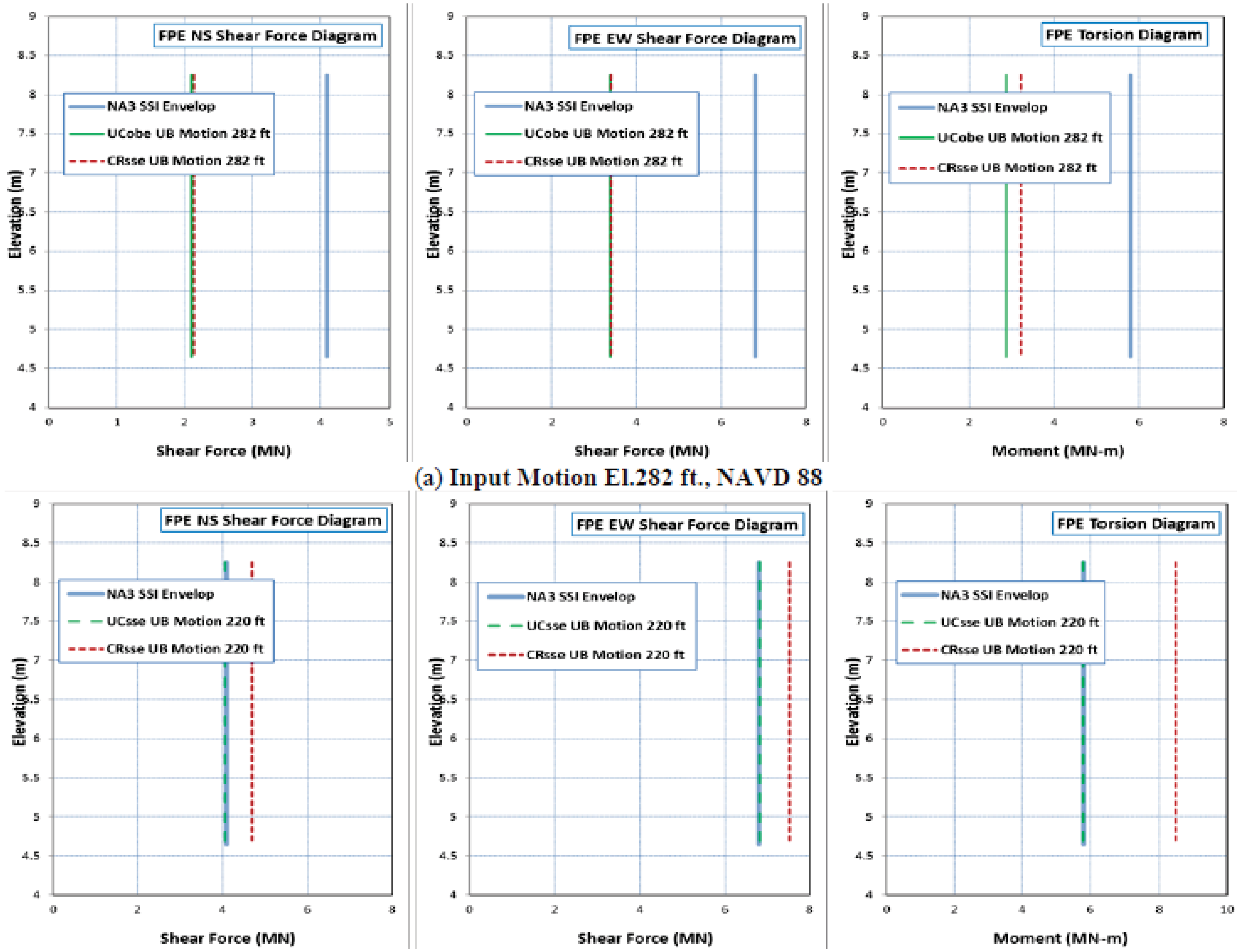
Table 3A.17.9.3-202 Vertical Out-of-Plane Loads on FPE Roof UB

Slab		Flexible Mode (SDOF Oscillator)			Rigid Mode (LMSM)		Eq. Ave. Acc. (g)			
			Acceleration (g)		Acceleration (g)		CR_{SSE}			
Elev. (m)	Location	Mass Node	Surf. Motion (Case S3)	Deep Motion (Case S4)	Surf. Motion (Case S3)	Deep Motion (Case S4)	Surf. Motion (Case S3)	Deep Motion (Case S4)	UC_{SSE} ~~SSI~~ Envel.	Stand. Design
8.25	FPE Roof	13	0.74	1.88	0.49	0.71	0.57	1.09 (shaded)	0.72	1.12

Note: Shaded cell identifies an exceedance due to concrete cracking effects.

NAPS DEP 3.7-1

Figure 3A.17.9.3-201 **Comparison of Horizontal Seismic Load Demands on FPE from UB Profile Analyses**



(a) Input Motion El.282 ft., NAVD 88

(b) Input Motion El.220 ft., NAVD 88

3A.17.10 Effect of Wall Out-of-Plane Vibration

As explained in DCD Section 3A.8.10, to obtain design loads and ISRS for flexible walls, site-specific seismic analyses of the RB/FB are performed on LMSM with SDOF oscillators representing the out-of-plane responses of flexible walls shown in the RB/FB complex seismic analysis model in DCD Figure 3A.7-4. Section 3A.17.12 describes the calculated results of the RB/FB out-of-plane oscillators and Section 3A.18.1.1 describes the site-specific enveloping out-of-plane loads on the RB/FB flexible walls.The out-of-plane response of CB and FWSC walls under both uncracked and cracked conditions are characterized with modes of vibrations for which frequencies are higher than 50 Hz.

3A.17.11 Effect of Structure-Structure Interaction

Unit 3 site-specific evaluations of the effects of SSSI between the FWSC and the adjacent CB, which are of similar size and weight, follow a methodology consistent with the one used in the standard design to determine:

- SSSI effects of the FWSC on the CB seismic response based on results of site-specific SSSI analyses of the CB-FWSC combined model presented in Figure 3A.17.11-202
- SSSI effects of the CB on the FWSC seismic response based on results of site-specific SSSI analyses of the FWSC-CB combined model presented in Figure 3A.17.11-203.

The site-specific evaluations of the SSSI effects of the large and heavy RB/FB on the seismic response of the small and light CB are also based on the results of site-specific SSSI analyses using the CB-RB/FB combined model presented in Figure 3A.17.11-201. This explicit approach that captures the effects of dynamic coupling between the RB/FB and CB at the Unit 3 site is different from the approximate approach used for the standard design SSSI evaluation in DCD Section 3A.8.11, which considers only the effect of the RB/FB on the seismic ground motion at the CB location. The standard design approach to evaluate the SSSI effects of the RB/FB on the CB cannot be directly implemented for the Unit 3 site-specific evaluations because it cannot explicitly capture the conditions between the two buildings and address the effect of subgrade property variations across the Unit 3 site.

Table 3A.15-204 lists the cases used in the Unit 3 analyses for the SSSI effect of the RB/FB on the CB. Table 3A.15-205 and 3A.15-206 list the

cases used in analyses for the SSSI effects of the FWSC on the CB and CB on the FWSC, respectively.

The site-specific SSSI analyses are performed on combined models using the MSM and the SASSI2010 and ACS SASSI computer programs.

The CB-FWSC and FWSC-CB combined SSSI models representing the dynamic properties of the CB and FWSC structures also include near-field subgrade elements providing an explicit representation of the subgrade conditions existing between the FWSC and the CB. The near-field solid elements model the concrete fill and structural fill backfilled into the gap between the buildings. The concrete fill is backfilled up to the top of the Zone III rock elevation, and the structural fill is backfilled above the Zone III rock elevation up to the finished ground level grade.

The site-specific SSSI effects of the FWSC on the CB seismic response are evaluated using the results of the SSSI analyses of the CB-FWSC combined model for the full column subgrade profiles representing strain-compatible dynamic soil/rock properties at the CB location. The SSSI analyses are performed on full column profiles to accurately capture the effects of concrete fill placed below the FWSC foundation on the CB seismic response. To account for the effects of the potential variability in the properties of the soil and rock, the CB-FWSC SSSI analyses are performed for the two bounding subgrade stiffness conditions using the UB and LB CB full column profiles and corresponding in-layer input motions defined by the CB SSI design spectra applied at the bottom of the CB foundation. The subgrade dynamic properties and input motions used for the CB-FWSC SSSI analyses are identical to those used for the SSI analyses of the CB stand-alone model. The use of identical inputs enables the SSSI effect of the FWSC on the CB seismic response to be directly evaluated by comparing the results obtained from the SSSI analyses of the CB-FWSC combined model with the results of the SSI analyses of the CB stand-alone model.

The site-specific SSSI effects of the FWSC on the CB site-specific seismic design basis are evaluated by comparing the site-specific CB-FWSC SSSI analyses results with the corresponding CB site-specific seismic design basis structural loads and ISRS that are developed as the envelope of the results of the SSI analyses of the CB stand-alone

models. Comparisons are also made with the corresponding seismic loads and ISRS used for the standard design of the CB.

The site-specific SSSI effects of the CB on the FWSC seismic response are evaluated using the results of the SSSI analyses of the FWSC-CB combined model for the subgrade profiles representing strain-compatible dynamic soil/rock properties at the FWSC location. The subgrade dynamic properties and the input motions used for the site-specific FWSC-CB SSSI analyses are identical to those used for the site-specific FWSC stand-alone model SSI analyses. The only difference between the FWSC-CB combined model and the FWSC stand-alone model is that the structural fill placed in the gap between the two buildings and around the concrete fill below the FWSC foundation is explicitly included in the FWSC-CB combined model and is neglected in the FWSC stand-alone model. Inclusion of the structural fill in the combined model is intended to address the effects of the structural fill on the FWSC seismic response, in particular the structural fill placed between the two buildings. Consideration of the site conditions at the FWSC location enables the SSSI effect of the CB on the FWSC seismic response to be directly evaluated by comparing the results obtained from the SSSI analyses of the FWSC-CB combined model with the results of the SSI analyses of the FWSC stand-alone model.

The SSSI effects of the CB on the FWSC site-specific seismic design basis are evaluated by comparing the site-specific FWSC-CB SSSI analyses results with the corresponding FWSC enveloping seismic load demands and site-specific ISRS developed as the envelope of the results of the SSI analyses of the FWSC stand-alone model. Comparisons are also made with the corresponding structural seismic loads and ISRS used for the standard design of the FWSC.

The following SSSI analyses cases are performed on the FWSC-CB combined model with OBE damping values to obtain the SSSI responses needed for the development of the FWSC site-specific design ISRS:

- LB, BE, and UB full column profiles using the surface outcrop input motion compatible with the FWSC FIRS that governs the FWSC ISRS at lower frequencies

- LB, BE, and UB full column profiles using the corresponding in-column input motion compatible with the SSI design spectra at the bottom of the concrete fill placed below the FWSC foundation that governs the FWSC ISRS at high frequencies

The following SSSI analyses cases are performed on the FWSC-CB combined model with SSE damping values to obtain the SSSI responses for the development of the site-specific seismic load demands on the FWSC:

- LB, BE, and UB full column profiles using the corresponding in-column input motion compatible to the SSI design spectra at the bottom of the concrete fill placed below the FWSC foundation that governs the FWSC maximum SSI site-specific seismic load demand

The SSSI analyses of the FWSC-CB combined model are performed for the same set of inputs as the ones used for the SSI analyses of the FWSC stand-alone model.

Results of the evaluations show that, in general, the SSSI between the CB and the FWSC have small effects on the site-specific seismic responses of these two structures.

To provide explicit representation of the subgrade conditions that exist between the RB/FB and the CB, the CB-RB/FB combined model includes the Access Tunnel, which is structurally isolated from the RB/FB and CB, and the near-field elements representing the concrete fill and structural fill below the Access Tunnel and surrounding the CB. The concrete fill is backfilled up to the top of the Zone III rock elevation, and the structural fill is backfilled above the Zone III rock elevation up to the finished ground level grade.

The site-specific SSSI effects of the RB/FB on the CB seismic response are evaluated using the results of the SSSI analyses of the CB-RB/FB combined model for the full column and partial column subgrade profiles representing strain-compatible dynamic soil/rock properties at the CB location. To account for the effects of the potential variability in the properties of the soil and rock, two different embedment conditions and two sets of bounding subgrade strain-compatible dynamic properties are

considered. The CB-RB/FB SSSI analyses are performed using the following:

- The LB partial column profile, which is representative of the lower bound subgrade stiffness conditions at the CB location, with the in-column input control motion applied at the CB basemat bottom elevation
- The UB full column profile, which is representative of the upper bound subgrade stiffness conditions at the CB location, with the in-column input control motion applied at the CB basemat bottom elevation
- The UB partial column profile, which is the analysis case that governs the overall seismic response of the CB stand-alone model with the in-column input control motion applied at the CB basemat bottom elevation

The subgrade dynamic properties and input motions used for the CB-RB/FB SSSI analysis are identical to those used for the SSI analysis of the CB stand-alone model. Using identical inputs enables the SSSI effects of the RB/FB on the CB seismic response to be directly evaluated by comparing the results obtained from the SSSI analyses of the CB-RB/FB combined model with the results of the SSI analyses of the CB stand-alone model. The site-specific SSSI evaluations show that the SSSI between the CB and the RB/FB have small effects on the site-specific seismic response of CB at the Unit 3 site.

The site-specific design of the CB envelops the SSSI effects of the FWSC on the CB seismic response. The CB site-specific design is based on the envelope of the results obtained from the SSI analyses of the CB stand-alone models representing two bounding embedment conditions:

- Partially embedded in Zone III rock that neglects the effects of the structural fill and in-situ saprolite on the seismic response of the CB
- Fully embedded that includes the effects of the structural fill and in-situ saprolite on the seismic response of the CB

The consideration of these two bounding embedment conditions provides a site-specific design basis that envelopes the amplification of the CB seismic response due to SSSI with the nearby FWSC. The CB site-specific design based on the envelope of the results obtained from the SSI analyses of the CB stand-alone models, along with the standard design basis, also bound the possible amplifications of the CB seismic response due to SSSI effects from the nearby RB/FB.

The site-specific seismic response analyses of the FWSC consider that the FWSC is essentially a surface-founded structure that is supported by a block of concrete fill embedded in the in-situ saprolite and Zone III rock. The SSI analyses of the FWSC stand-alone model consider LB, BE, and UB subgrade properties and two input control motions specified at the bottom of the FWSC foundation at Elevation 282 ft (NAVD88) and at the bottom of the concrete fill at Elevation 220 ft (NAVD88). The results of the evaluations of the SSSI effects between CB and FWSC show that the responses obtained from these SSI analyses do not envelope all possible SSSI-induced amplifications of the FWSC response. The results obtained from the FWSC-CB SSSI analyses are used to develop the FWSC site-specific seismic design basis to envelope the small amplifications of the FWSC response that are due to the SSSI with the CB.

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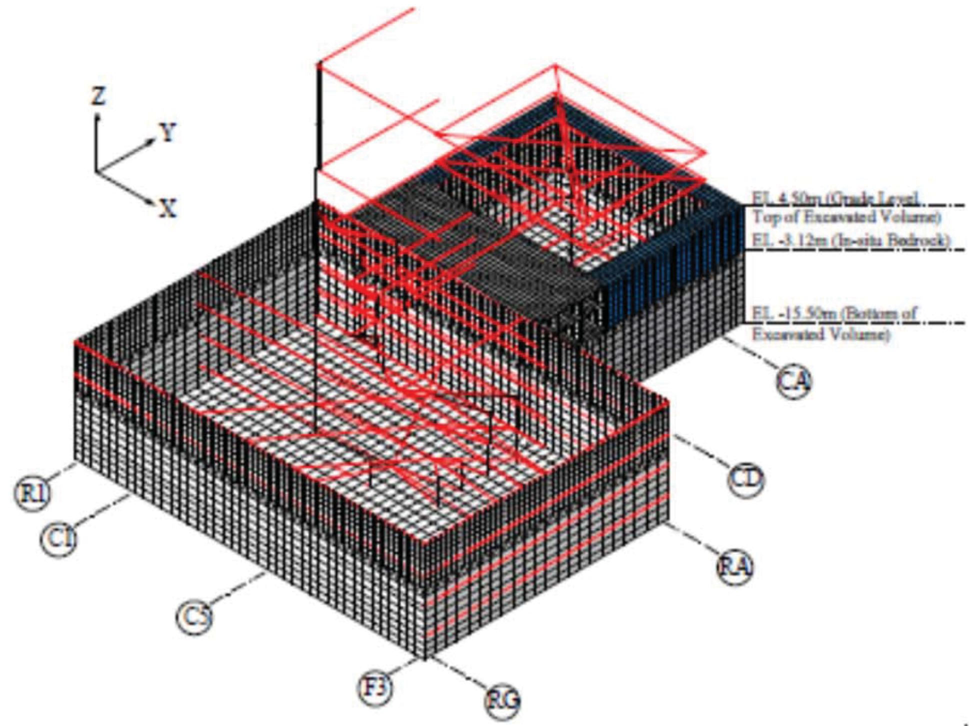
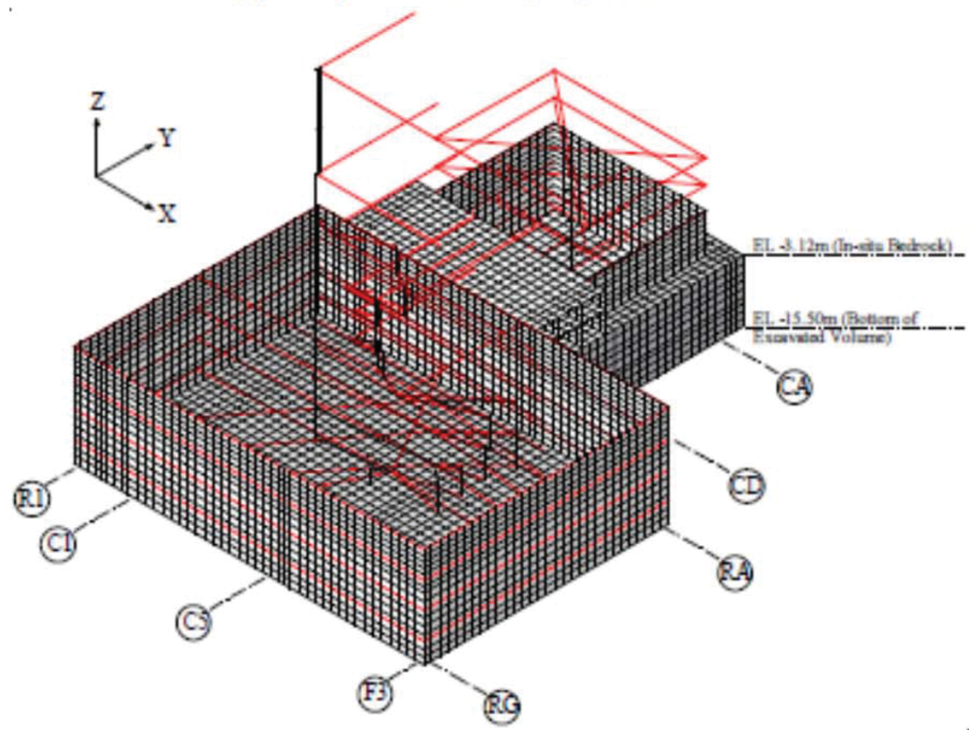
Figure 3A.17.11-201 **CB-RB/FB Combined Model**

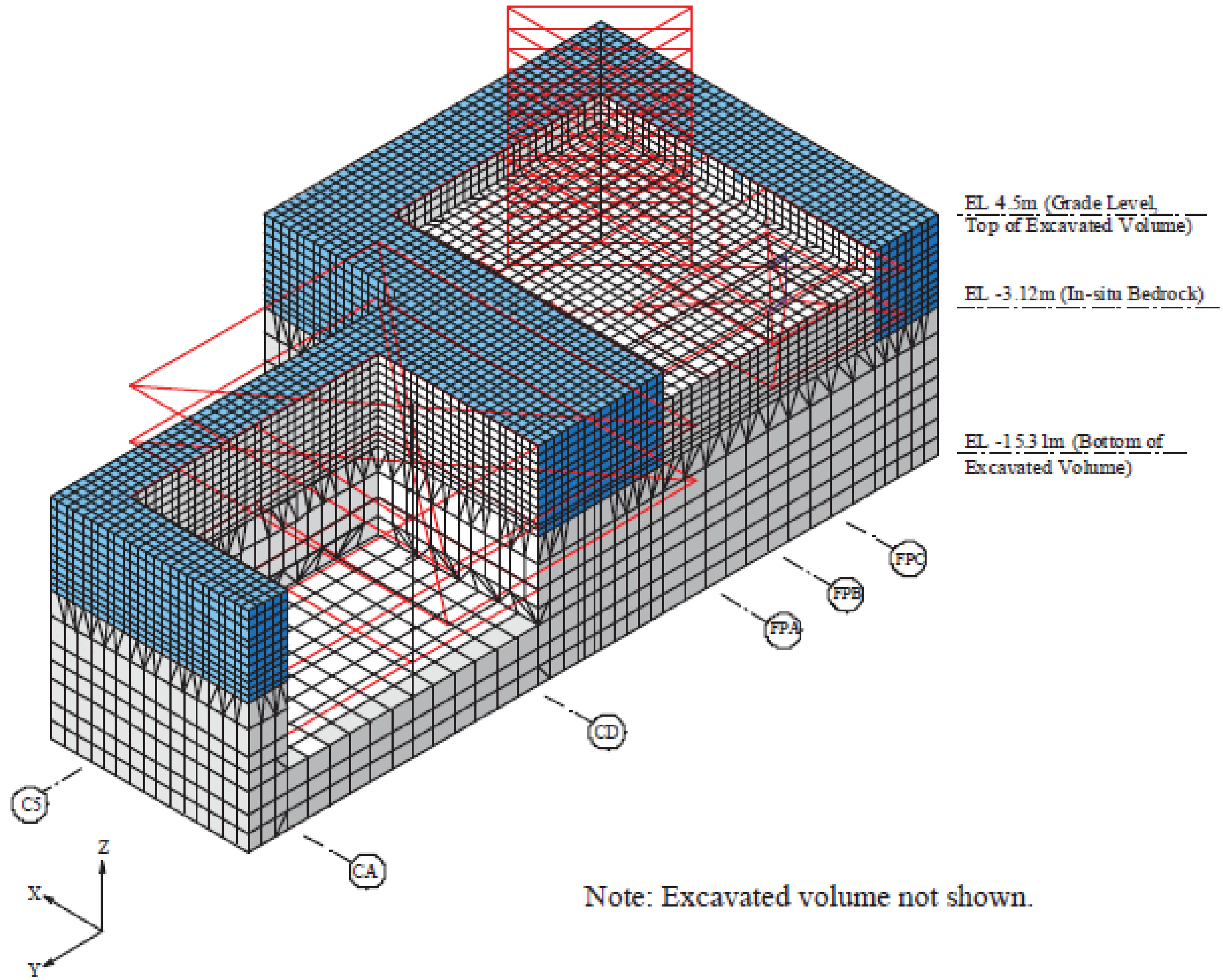


(a) Fully Embedded (FE) Model



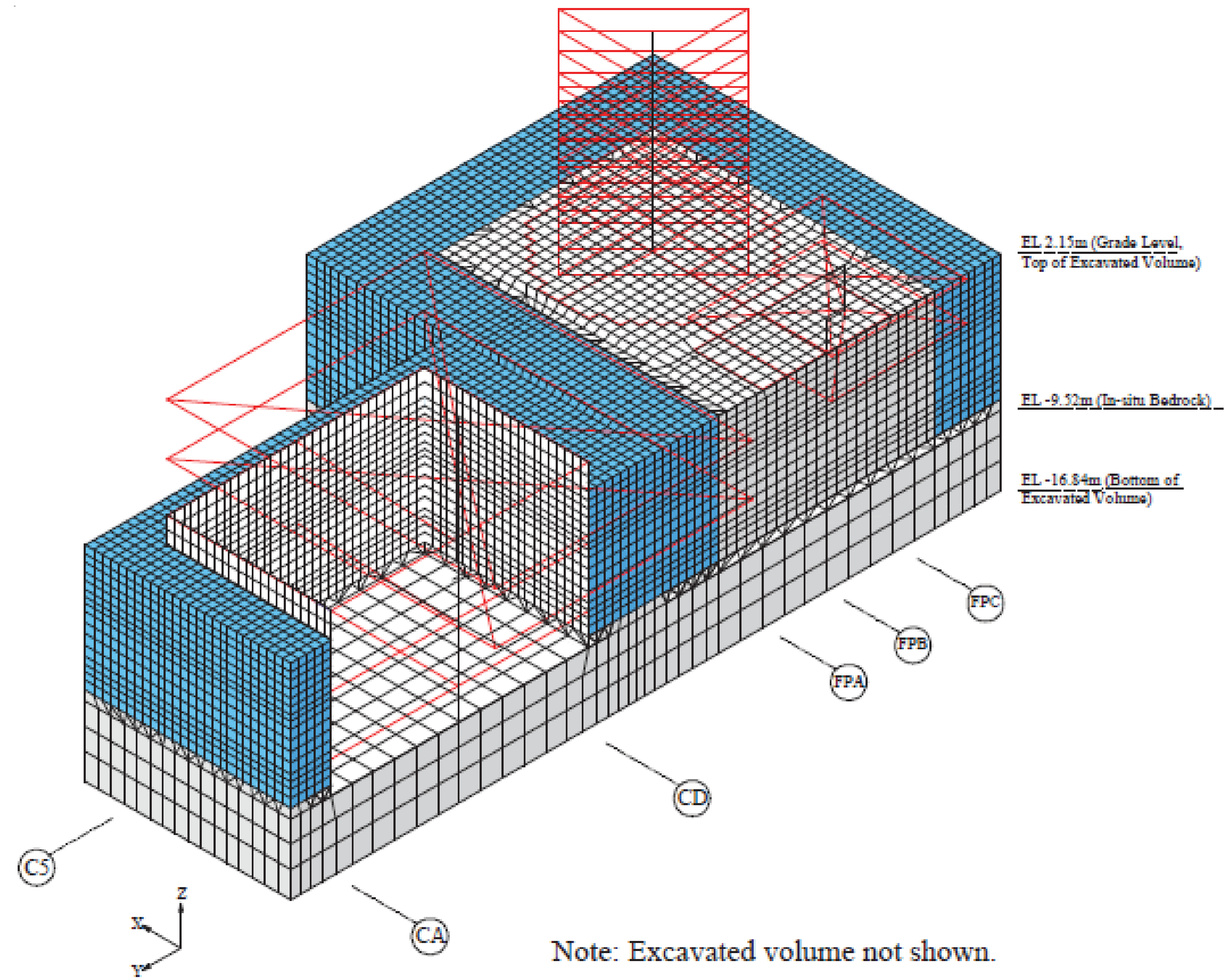
(b) Partially Embedded (PE) Model

NAPS DEP 3.7-1 Figure 3A.17.11-202 **CB-FWSC Combined Model**



NAPS DEP 3.7-1

Figure 3A.17.11-203 **FWSC-CB Combined Model**



3A.17.12 Unit 3 SSI Analysis Results for RB/FB

This section presents the results of the site-specific SSI analyses of the RB/FB models with upper bound stiffness properties and OBE damping values (Cases 1 through 6 in Table 3A.15-201).

3A.17.12.1 RB/FB SSI Response

The results of the site-specific SSI analysis of the RB/FB indicate that the co-directional responses (the responses in the direction of the applied earthquake) govern the seismic response of the RB/FB at the Unit 3 site. The most pronounced cross-directional responses of the RB/FB basemat are observed in the results obtained from the analyses of the LB profile, thus indicating that the softer subgrade amplifies the torsional and rocking response of the RB/FB basemat. The rocking of the RB/FB basemat in the NS(x) direction is the most significant cross directional response.

Comparisons of SSI analyses results for different embedment configurations and subgrade material properties show that the peak responses the RB/FB at Unit 3 site are at frequencies close to the natural frequencies of structures. Variations of subgrade properties affect only the amplitudes of peak responses, with the analyses of the UB subgrade profiles yielding the largest peak response as a result of the lower damping of subgrade materials. The peak horizontal responses of the RB/FB, RCCV, and VW are at lower frequencies where the full column input motion spectra exceed the partial column spectra.

Comparisons of results obtained from the analyses of the six subgrade profiles indicate that the Unit 3 rock subgrade has a very small effect on the seismic response of the RB/FB. The differences in responses obtained from the analyses of different subgrade profiles are due to the differences in the subgrade material damping and the different energy content of the input motion at natural frequencies of the RB/FB structures. The analyses of the UB full column profile mostly yields bounding horizontal responses of the RB/FB, RCCV, and VW.

3A.17.12.2 Maximum Accelerations and Member Forces

Figures 3A.17.12.2-201a through 3A.17.12.2-202e present comparisons of the results for maximum absolute accelerations at the RB/FB mass locations from the site-specific SSI analyses of the RB/FB model with UB stiffness and OBE damping (analyses Cases 1 through 6 in Table 3A.15-201). Table 3A.17.12.2-201 compares the maximum vertical

acceleration results for the RB/FB slab SDOF oscillators obtained from the analyses of the six subgrade profiles. Table 3A.17.12.2-202 compares the maximum accelerations of wall SDOF oscillators obtained from the analyses.

Figures 3A.17.12.2-203a through 3A.17.12.2-204e present comparisons of the maximum shear forces and torsion results obtained from the analyses of the RB/FB PE and RB/FB FE models. Comparisons of the maximum member force and acceleration results show that the analyses of the UB subgrade profiles typically govern the RB/FB maximum responses.

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Table 3A.17.12.2-201 **Maximum Accelerations of Slab SDOF Oscillators – RB/FB**

SDOF Oscillator		Vertical Acceleration (g)							
		Full Column			Partial Column			NA3 Enve-lope	Stan-dard Design
Elev. (m)	Node No.	LB	BE	UB	LB	BE	UB		
52.40	9101	*0.33*	0.33	0.32	0.30	0.30	0.30	0.33	1.20
	9102	*1.33*	1.29	1.28	0.80	0.79	0.77	1.33	1.82
	9103	3.90	5.58	*6.27*	2.91	4.11	4.79	6.27	3.14
	9104	2.02	2.38	*2.62*	1.89	2.01	2.26	2.62	2.45
	9105	1.60	2.27	*2.42*	1.62	2.10	2.17	2.42	2.32
	9106	2.44	3.09	3.52	2.11	3.06	*3.74*	3.74	2.99
	9107	1.42	2.31	*3.22*	1.59	2.08	3.08	3.22	2.80
	9108	1.12	1.69	*2.50*	1.24	1.49	2.09	2.50	2.61
34.00	9091	0.94	1.42	*1.61*	0.79	1.03	1.29	1.61	1.29
	9092	0.88	1.11	*1.61*	0.85	1.07	1.34	1.61	1.08
27.00	9081	1.15	1.36	*1.60*	1.00	1.10	1.26	1.60	1.16
	9082	0.89	1.17	*1.52*	0.76	0.93	1.26	1.52	0.99
	9083	0.78	1.06	*1.30*	0.74	0.84	1.06	1.30	1.09
	9084	0.73	1.25	*1.67*	0.90	0.99	1.42	1.67	1.32
	9085	0.86	1.14	*1.46*	0.72	0.89	1.19	1.46	0.97
22.50	9071	*1.15*	1.15	1.15	0.64	0.65	0.67	1.15	1.60
	9072	*1.79*	1.70	1.64	1.12	1.07	1.03	1.79	1.31
	9073	2.37	3.56	*4.47*	1.84	2.57	3.13	4.47	2.03
	9074	0.93	1.24	*1.53*	0.87	1.07	1.25	1.53	1.31
	9075	1.04	1.29	*1.51*	0.98	1.08	1.32	1.51	1.16
17.50	9061	1.97	3.00	*3.65*	1.61	2.30	2.90	3.65	1.79
	9062	1.15	1.57	2.40	1.37	1.68	*2.62*	2.62	1.49
	9063	0.77	0.96	*1.13*	0.64	0.82	0.98	1.13	0.82
	9064	1.14	1.38	*1.53*	0.93	1.07	1.17	1.53	1.84
	9065	0.59	1.00	*1.28*	0.80	0.93	1.24	1.28	1.42

NAPS DEP 3.7-1

Table 3A.17.12.2-201 **Maximum Accelerations of Slab SDOF Oscillators – RB/FB** *(continued)*

SDOF Oscillator		Vertical Acceleration (g)							
		Full Column			Partial Column			NA3 Enve-lope	Stan-dard Design
Elev. (m)	Node No.	LB	BE	UB	LB	BE	UB		
13.57	9051	0.74	0.95	*1.11*	0.64	0.82	0.96	1.11	0.81
	9052	0.59	0.94	*1.25*	0.75	0.89	1.22	1.25	1.46
9.06	9041	0.83	0.84	*0.95*	0.69	0.76	0.82	0.95	0.88
	9042	0.61	0.94	*1.26*	0.70	0.85	1.19	1.26	1.42
4.65	9031	0.84	1.42	*1.62*	0.98	1.41	1.59	1.62	1.17
	9032	0.79	0.86	*0.89*	0.69	0.74	0.84	0.89	0.97
	9033	0.78	0.93	*1.12*	0.78	0.85	0.98	1.12	1.02
	9034	0.60	1.20	*1.73*	1.10	1.16	1.81	1.81	1.51
	9035	0.57	0.87	*1.07*	0.69	0.77	1.05	1.07	1.38
-1.00	9021	0.92	0.86	*0.97*	0.88	0.84	0.92	0.97	1.12
	9022	1.26	1.60	1.90	0.99	1.64	*2.07*	2.07	1.45
	9023	0.80	0.92	*0.98*	0.75	0.79	0.89	0.98	1.01
	9024	0.72	0.98	*1.12*	0.67	0.94	1.03	1.12	0.89
	9025	0.55	1.00	1.14	0.75	0.85	*1.21*	1.21	1.34
	9026	0.54	1.06	1.38	1.06	1.22	*1.63*	1.63	1.57
	9027	0.55	0.65	0.67	0.54	0.57	*0.68*	0.68	0.88
-6.40	9011	0.77	0.76	*0.84*	0.73	0.76	0.84	0.84	0.92
	9012	0.68	1.03	*1.17*	0.76	1.05	1.13	1.17	0.92
	9013	0.71	0.88	1.35	0.68	0.93	1.52	1.52	1.35

Note: The shaded values in the table show exceedances from the standard design.
The values shown in Italics are the governing case.

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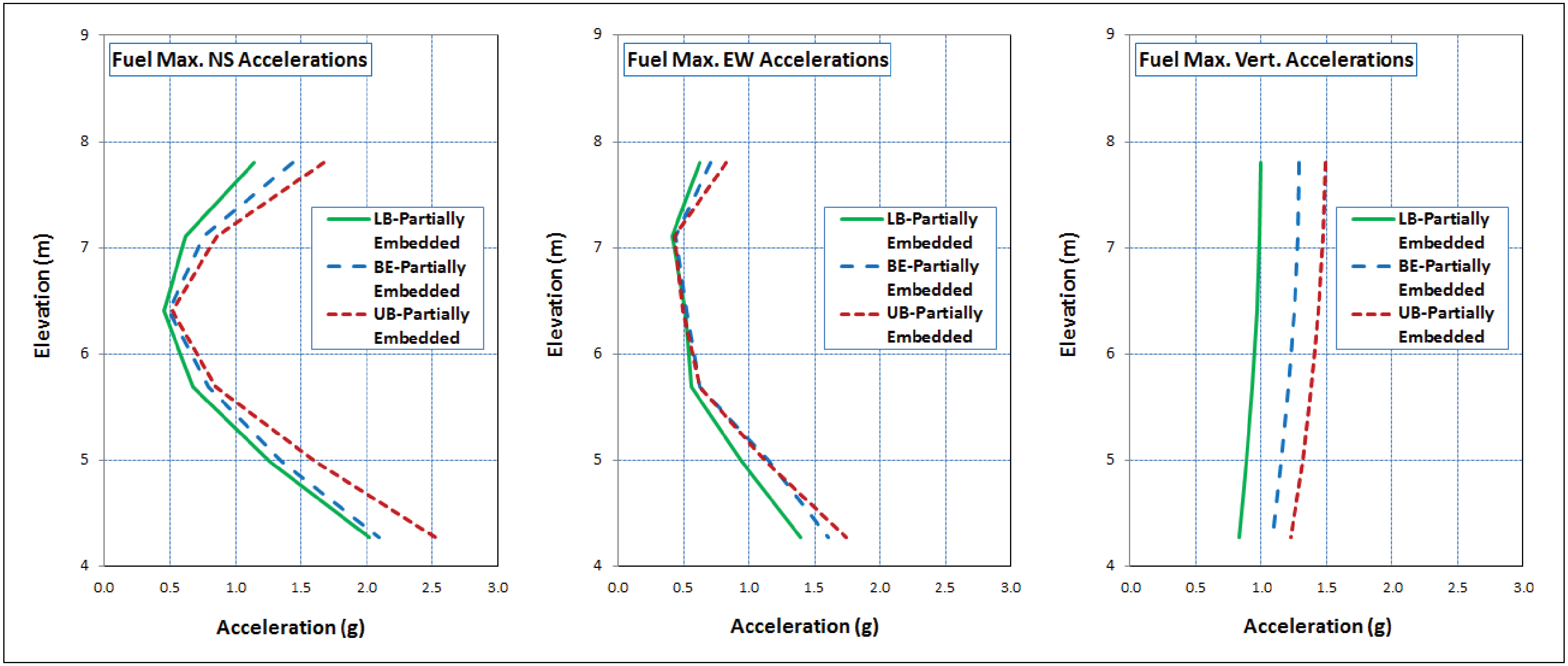
Table 3A.17.12.2-202 **Maximum Accelerations of Wall SDOF Oscillators – RB/FB**

SDOF Oscillator			Horizontal Acceleration (g)							
			Full Column			Partial Column			NA3 Envelope	Standard Design
Elev. (m)	Node No.	Dir.	LB	BE	UB	LB	BE	UB		
42.00	99981	NS (X)	1.73	2.32	*2.66*	1.45	1.93	2.20	2.66	1.54
	99982		1.07	1.46	*1.54*	0.93	1.22	1.53	1.54	1.00
13.57	99971		1.19	1.66	*2.11*	1.21	1.51	1.81	2.11	1.38
	99972		1.57	1.46	2.00	1.84	1.90	*2.29*	2.29	1.37
	99973		0.73	1.19	1.35	0.91	1.26	*1.88*	1.88	1.15
	99974		0.64	0.74	1.04	0.59	0.81	*1.10*	1.10	1.00
42.00	99983	EW (Y)	1.27	1.45	*1.86*	1.09	1.26	1.59	1.86	1.71
	99984		0.98	0.89	*1.02*	0.75	0.99	1.00	1.02	1.56
	99985		0.88	0.89	*1.00*	0.68	0.75	0.85	1.00	1.25
13.57	99975		1.10	1.50	1.66	1.38	1.74	*2.16*	2.16	1.28
	99976		0.43	0.64	0.78	0.53	0.69	*0.92*	0.92	1.00

Note: The shaded values in the table show exceedances from the standard design.
The values shown in Italics are the governing case.

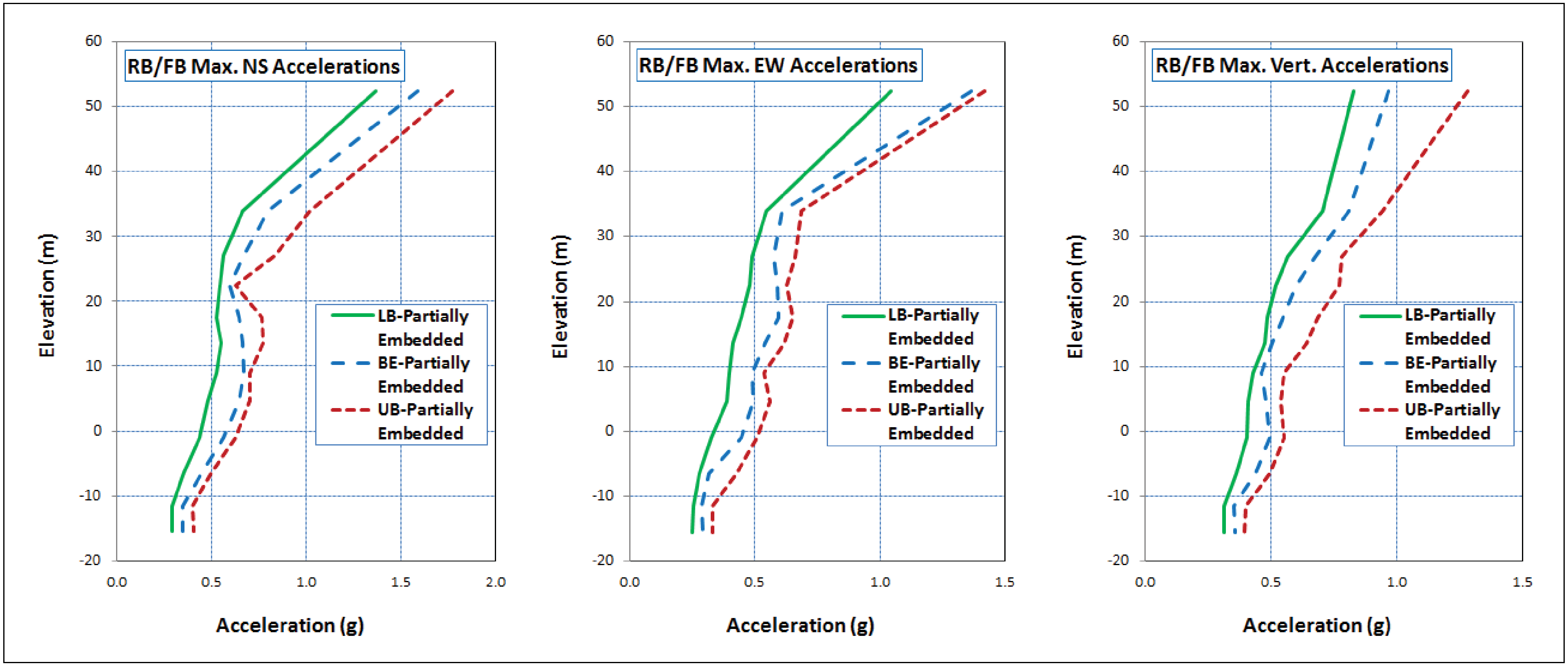
NAPS DEP 3.7-1

Figure 3A.17.12.2-201a Maximum Acceleration Results from Analyses of Fuel PE Model



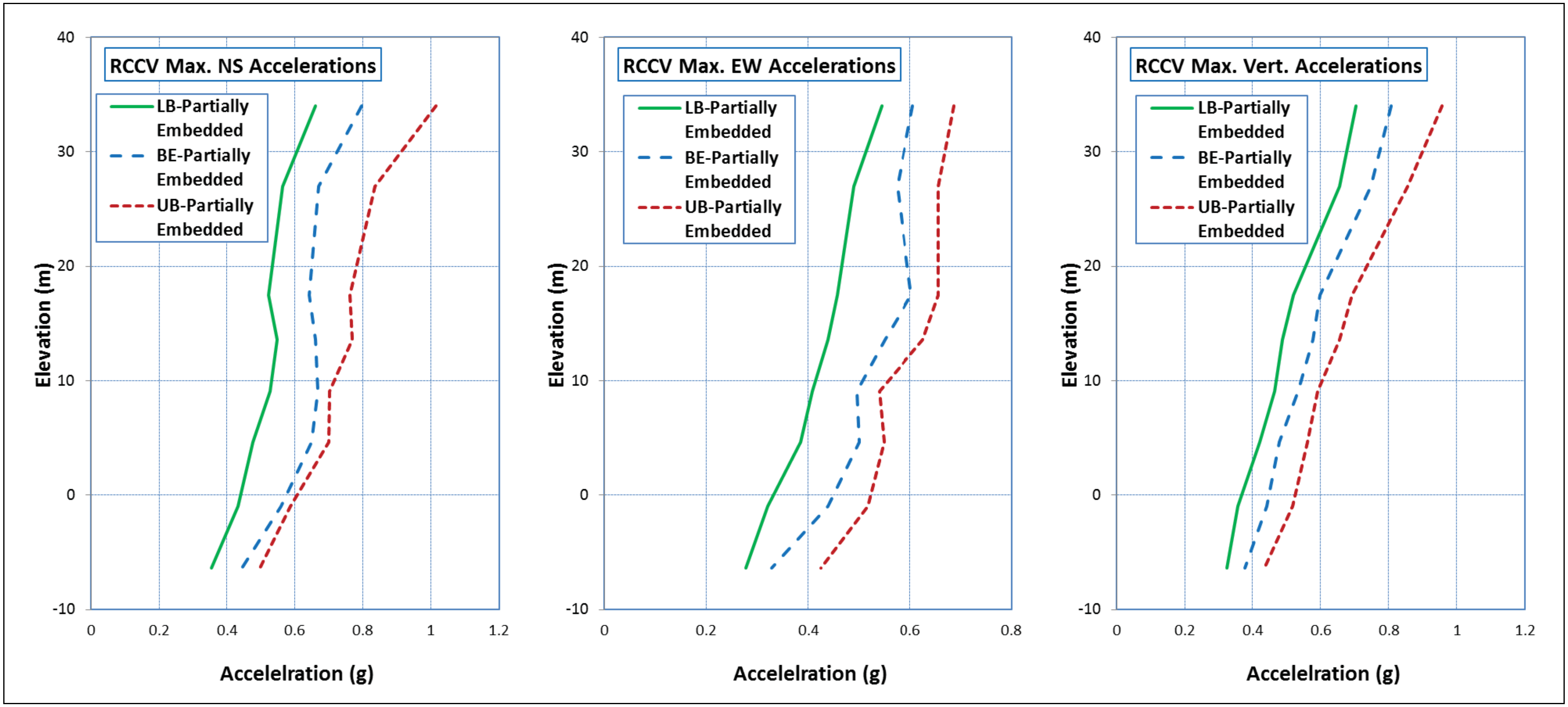
NAPS DEP 3.7-1

Figure 3A.17.12.2-201b **Maximum Acceleration Results from Analyses of RB/FB PE Model**



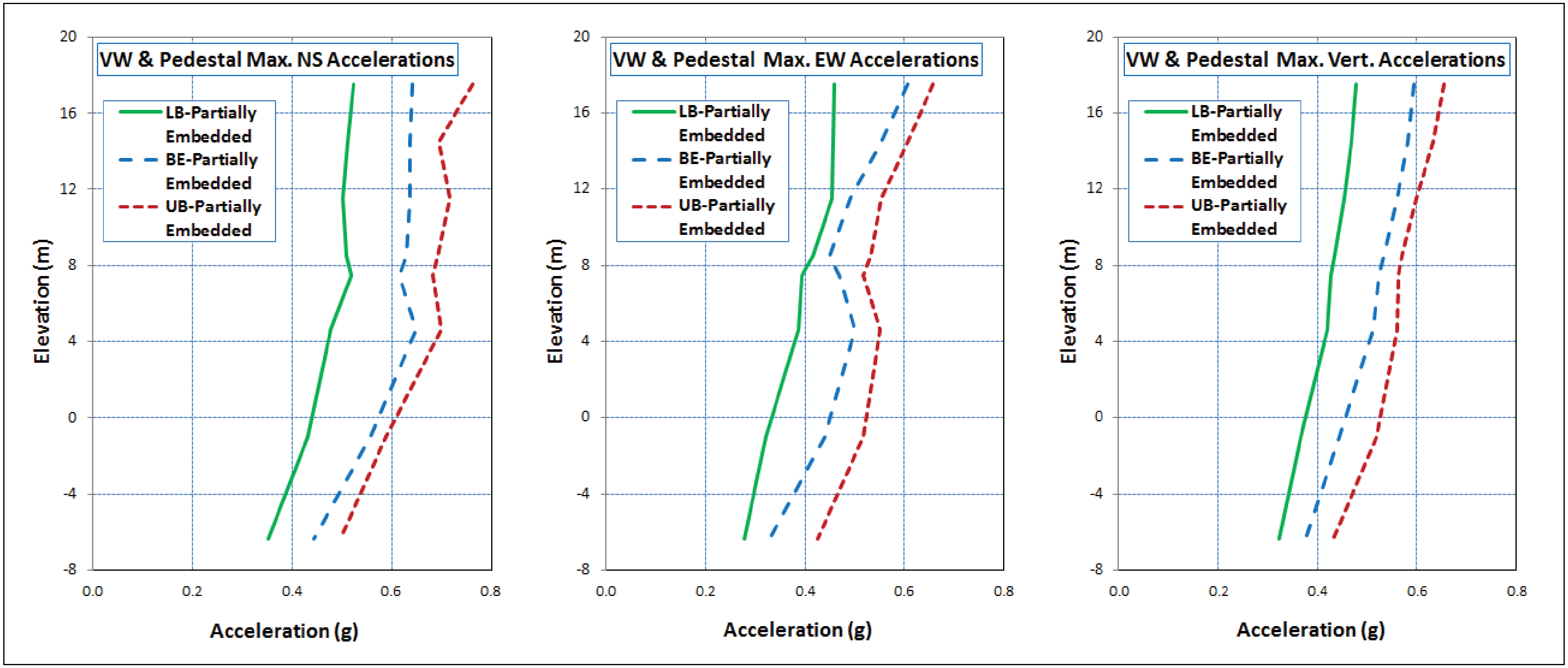
NAPS DEP 3.7-1

Figure 3A.17.12.2-201c **Maximum Acceleration Results from Analyses of RCCV PE Model**



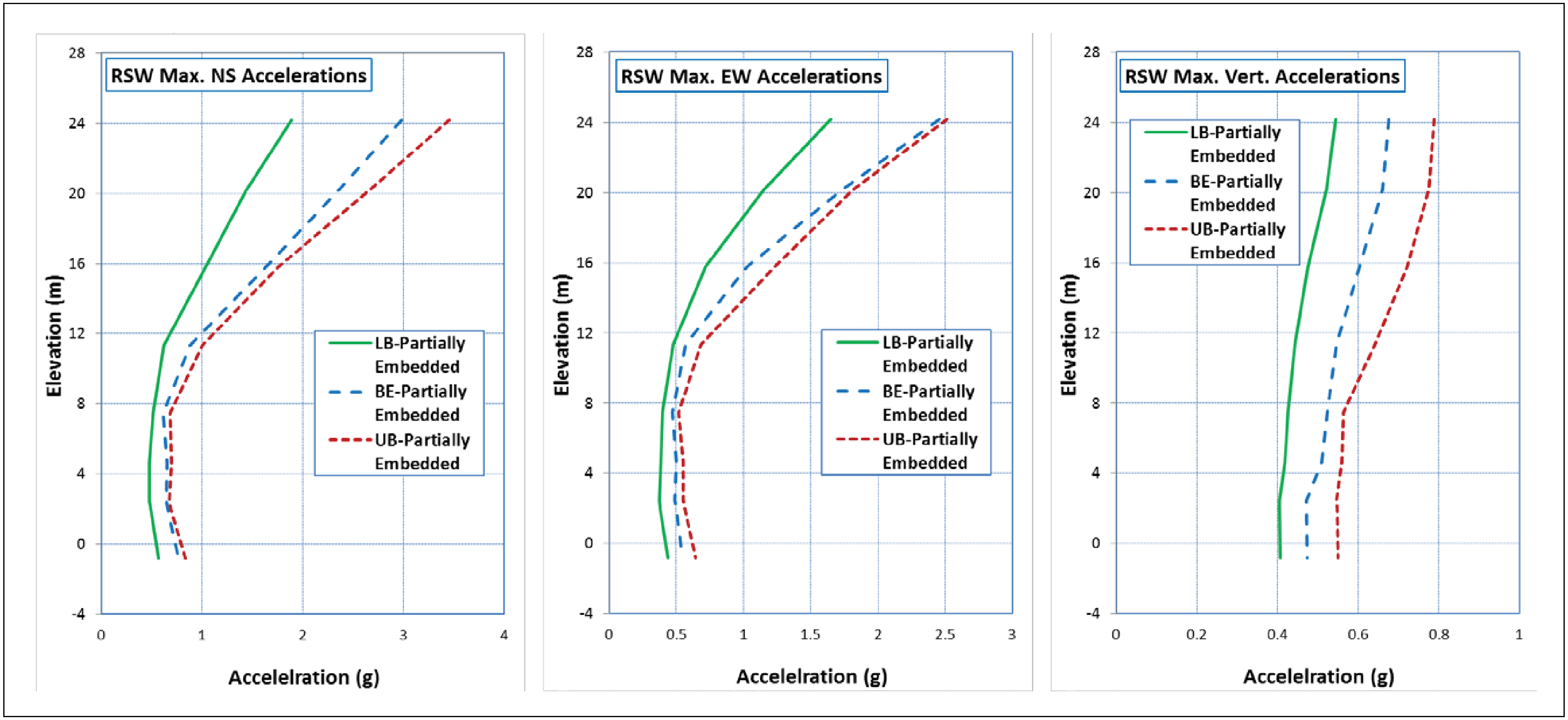
NAPS DEP 3.7-1

Figure 3A.17.12.2-201d **Maximum Acceleration Results from Analyses of Vent Wall/Pedestal PE Model**



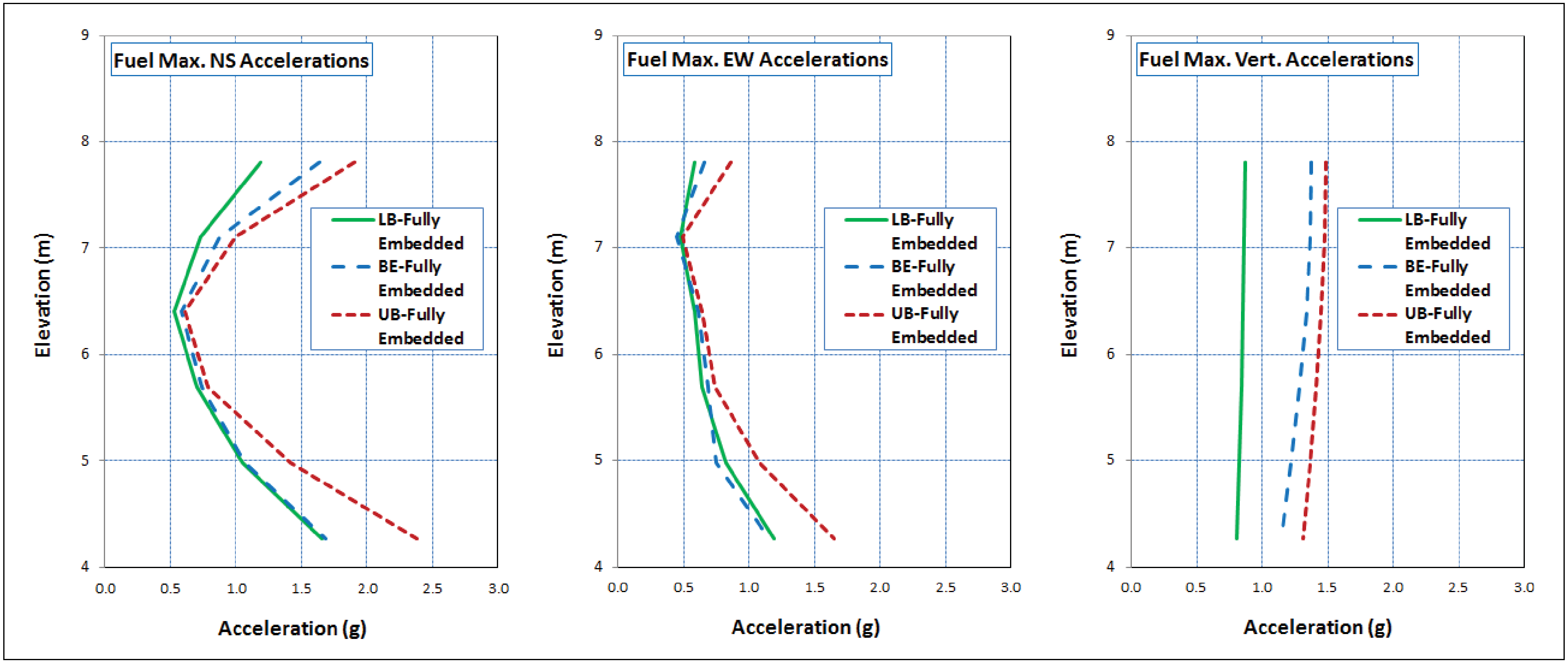
NAPS DEP 3.7-1

Figure 3A.17.12.2-201e Maximum Acceleration Results from Analyses of Reactor Shield Wall PE Model



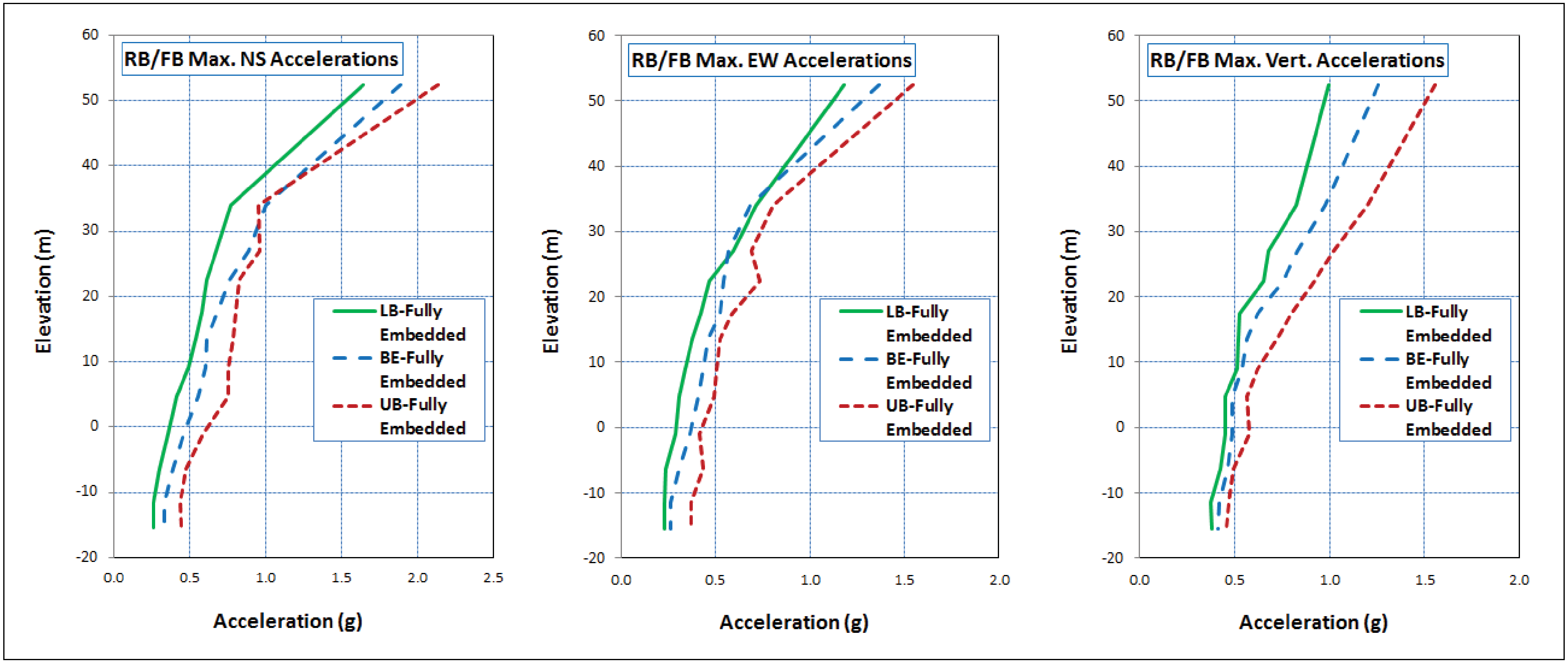
NAPS DEP 3.7-1

Figure 3A.17.12.2-202a **Maximum Acceleration Results from Analyses of Fuel FE Model**



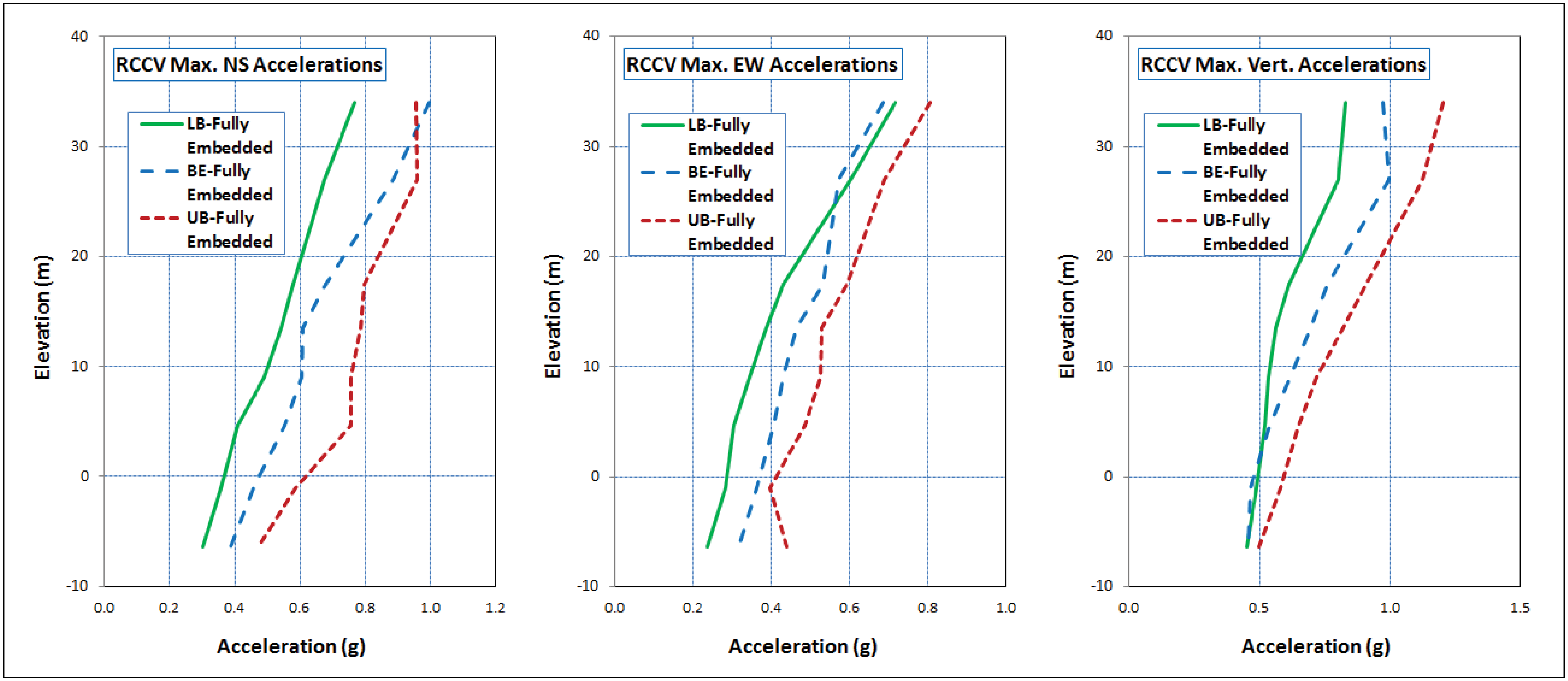
NAPS DEP 3.7-1

Figure 3A.17.12.2-202b Maximum Acceleration Results from Analyses of RB/FB FE Model



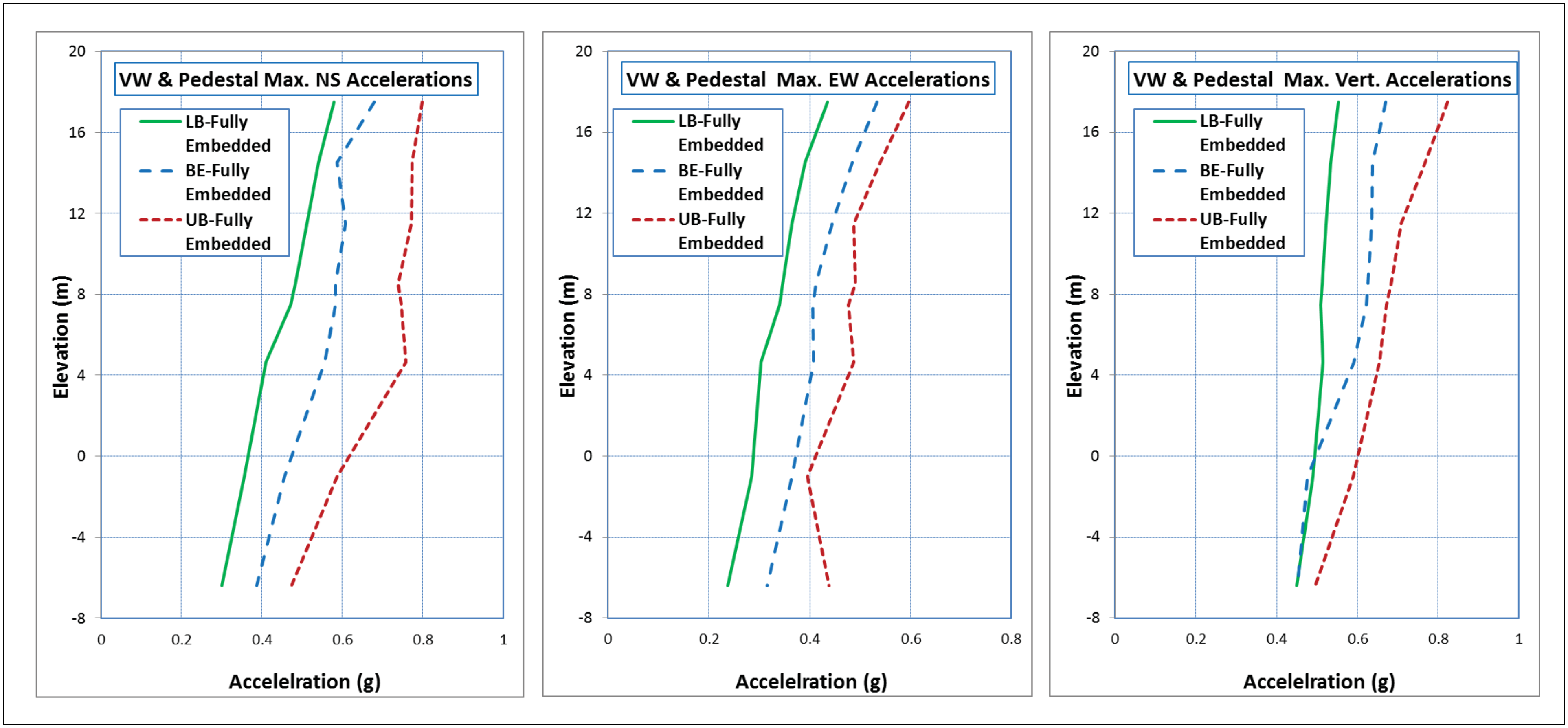
NAPS DEP 3.7-1

Figure 3A.17.12.2-202c Maximum Acceleration Results from Analyses of RCCV FE Model



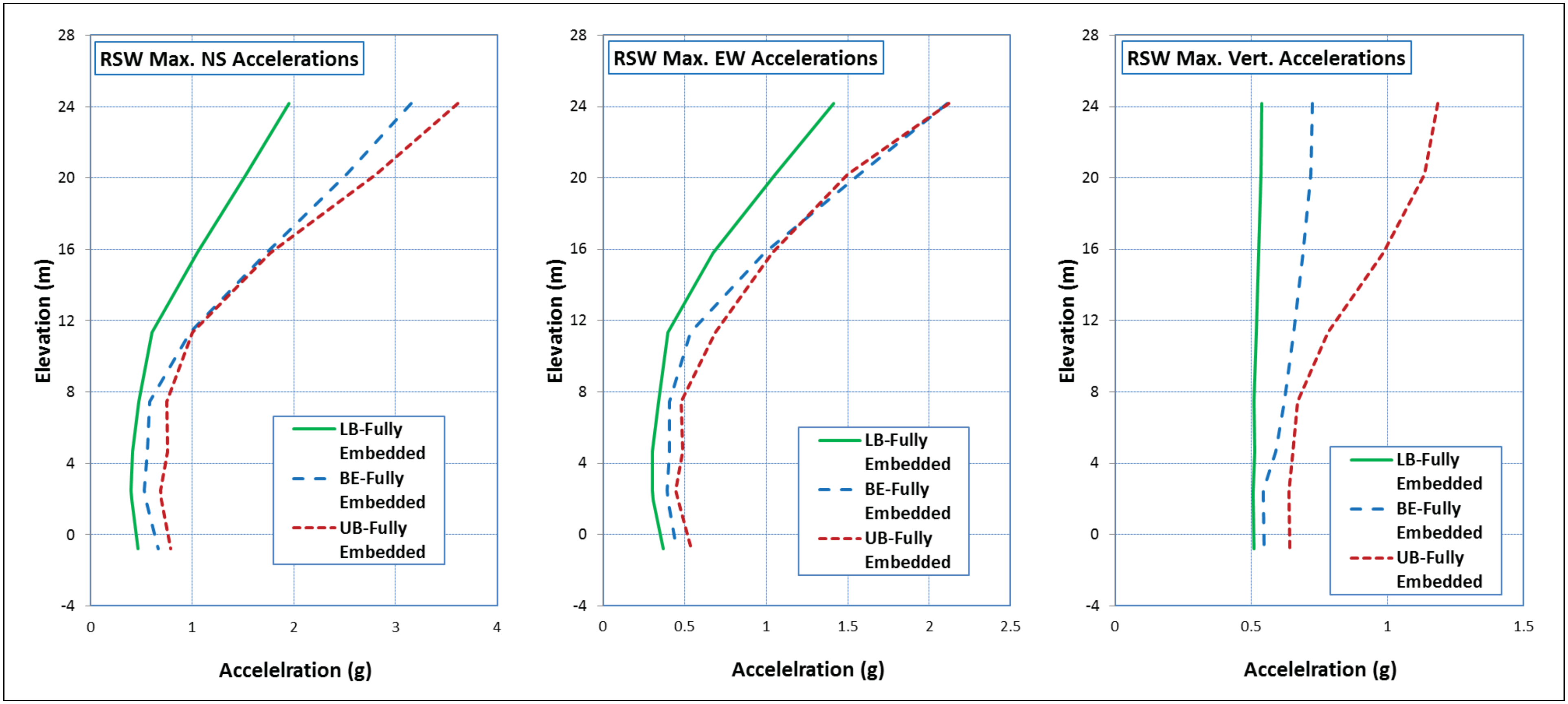
NAPS DEP 3.7-1

Figure 3A.17.12.2-202d **Maximum Acceleration Results from Analyses of Vent Wall/Pedestal FE Model**



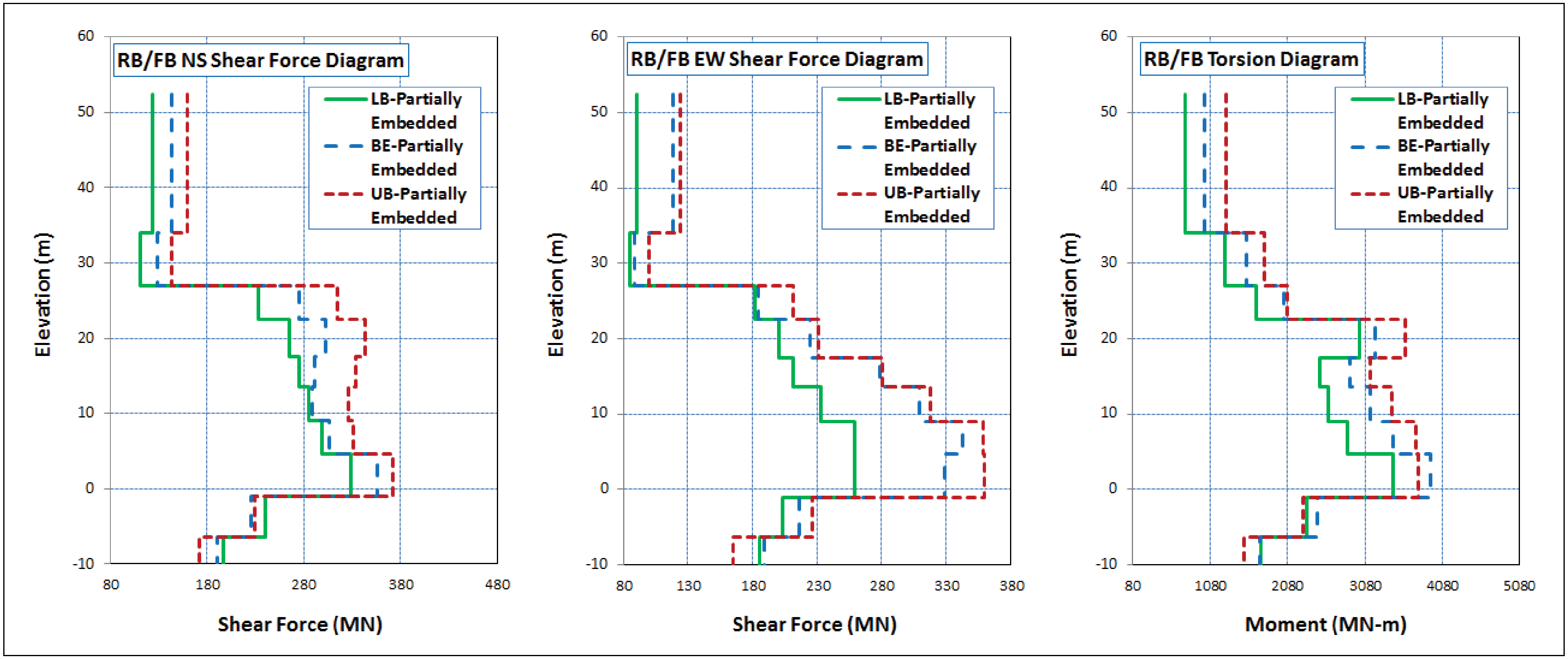
NAPS DEP 3.7-1

Figure 3A.17.12.2-202e **Maximum Acceleration Results from Analyses of Reactor Shield Wall FE Model**



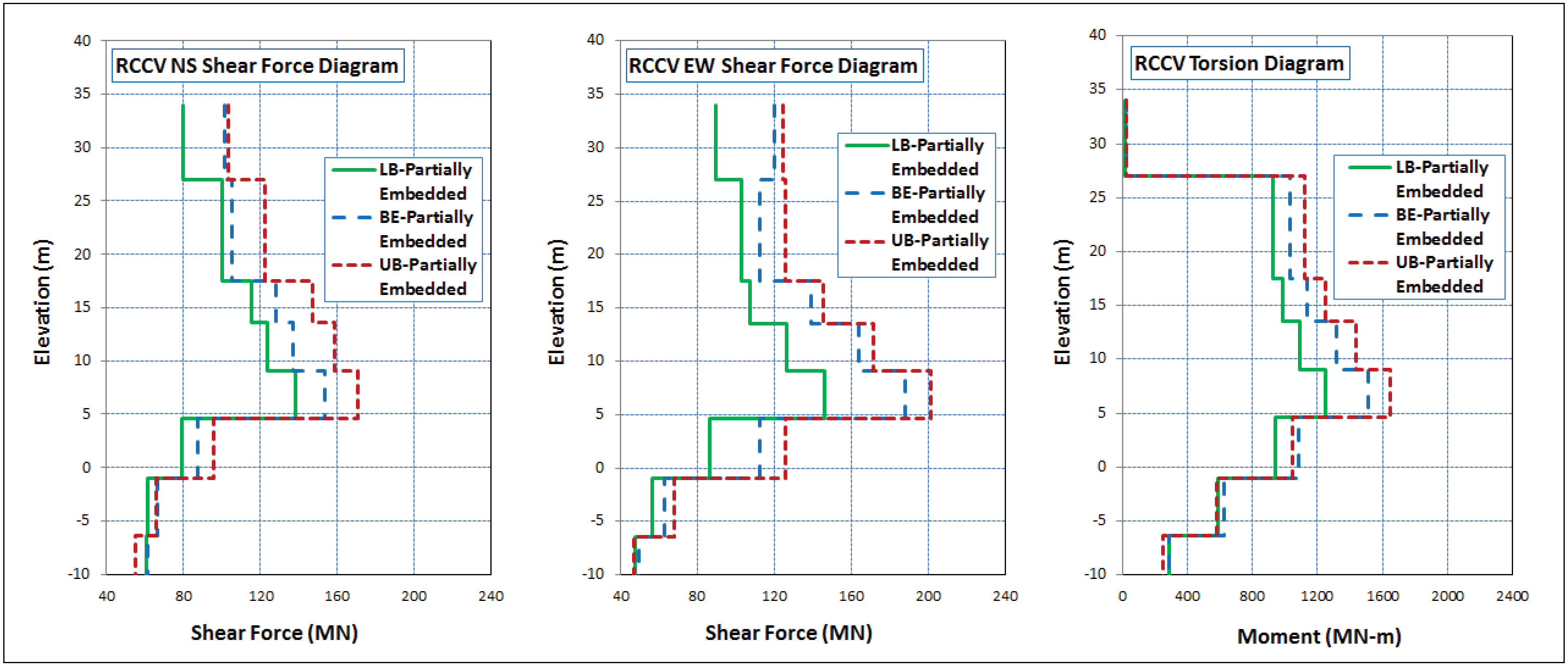
NAPS DEP 3.7-1

Figure 3A.17.12.2-203a Maximum Shear Forces and Torsion Results from Analyses of RB/FB PE Model



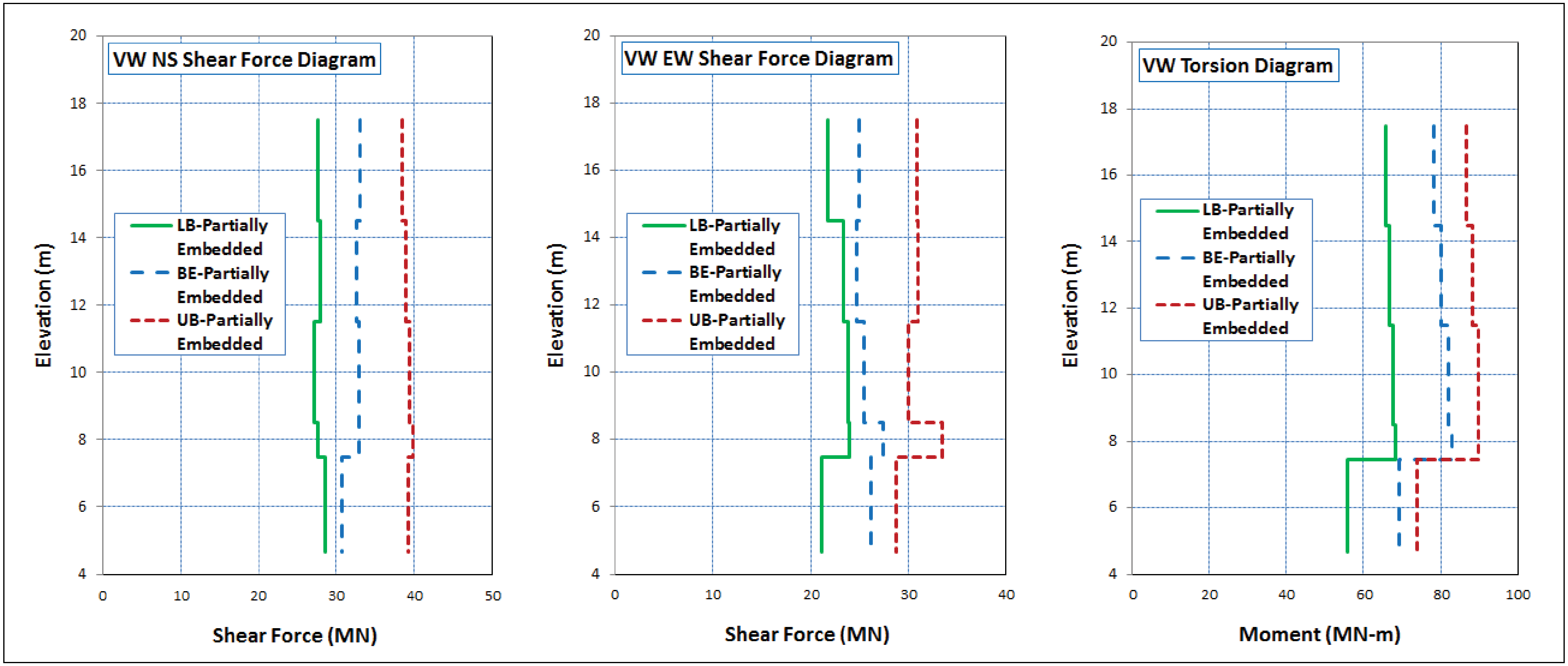
NAPS DEP 3.7-1

Figure 3A.17.12.2-203b **Maximum Shear Forces and Torsion Results from Analyses of RCCV PE Model**



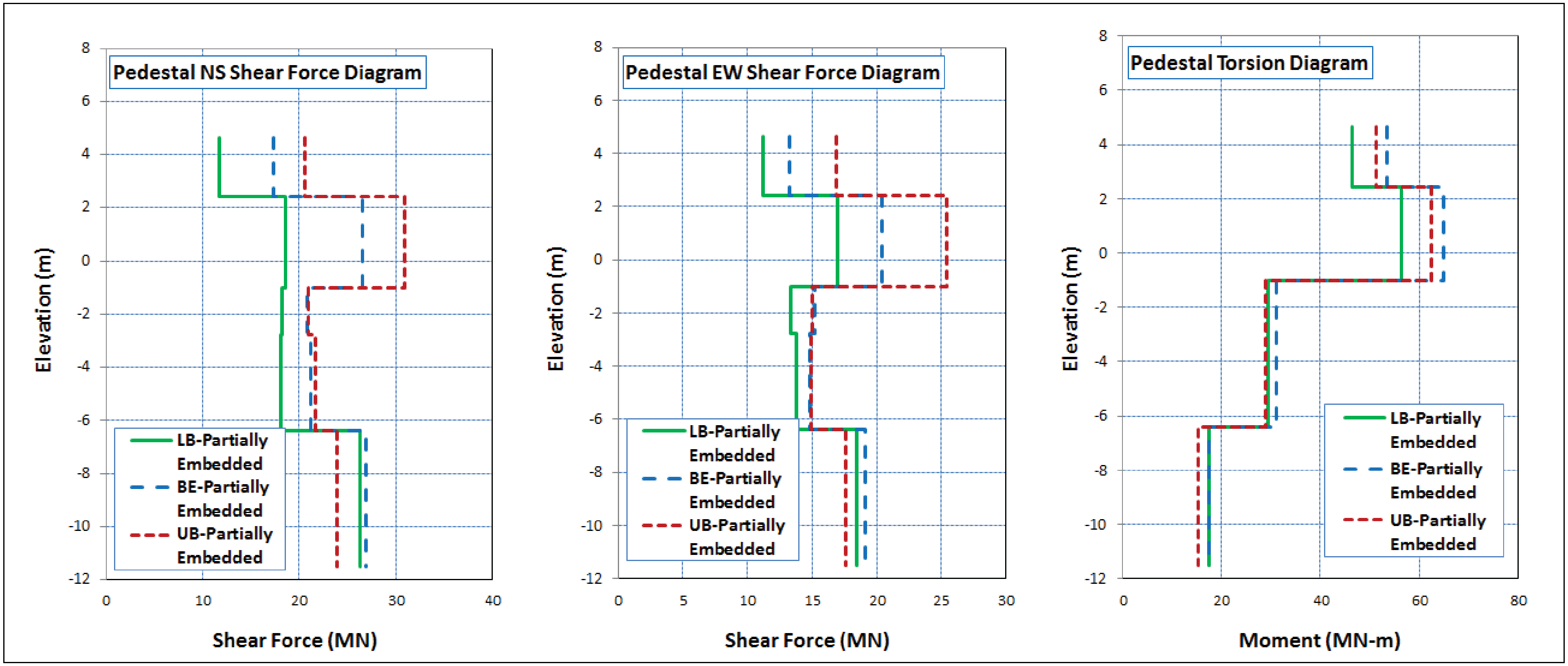
NAPS DEP 3.7-1

Figure 3A.17.12.2-203c Maximum Shear Forces and Torsion Results from Analyses of Vent Wall PE Model



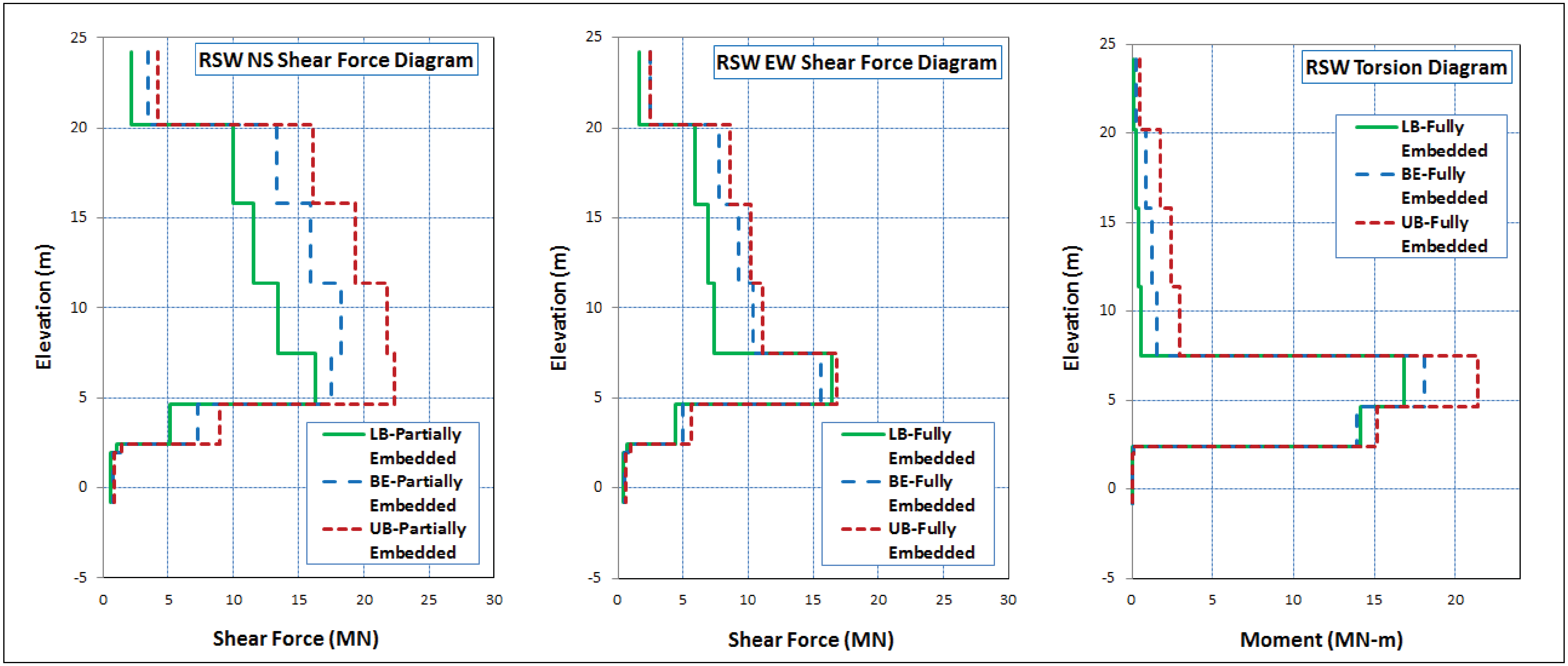
NAPS DEP 3.7-1

Figure 3A.17.12.2-203d **Maximum Shear Forces and Torsion Results from Analyses of Pedestal PE Model**



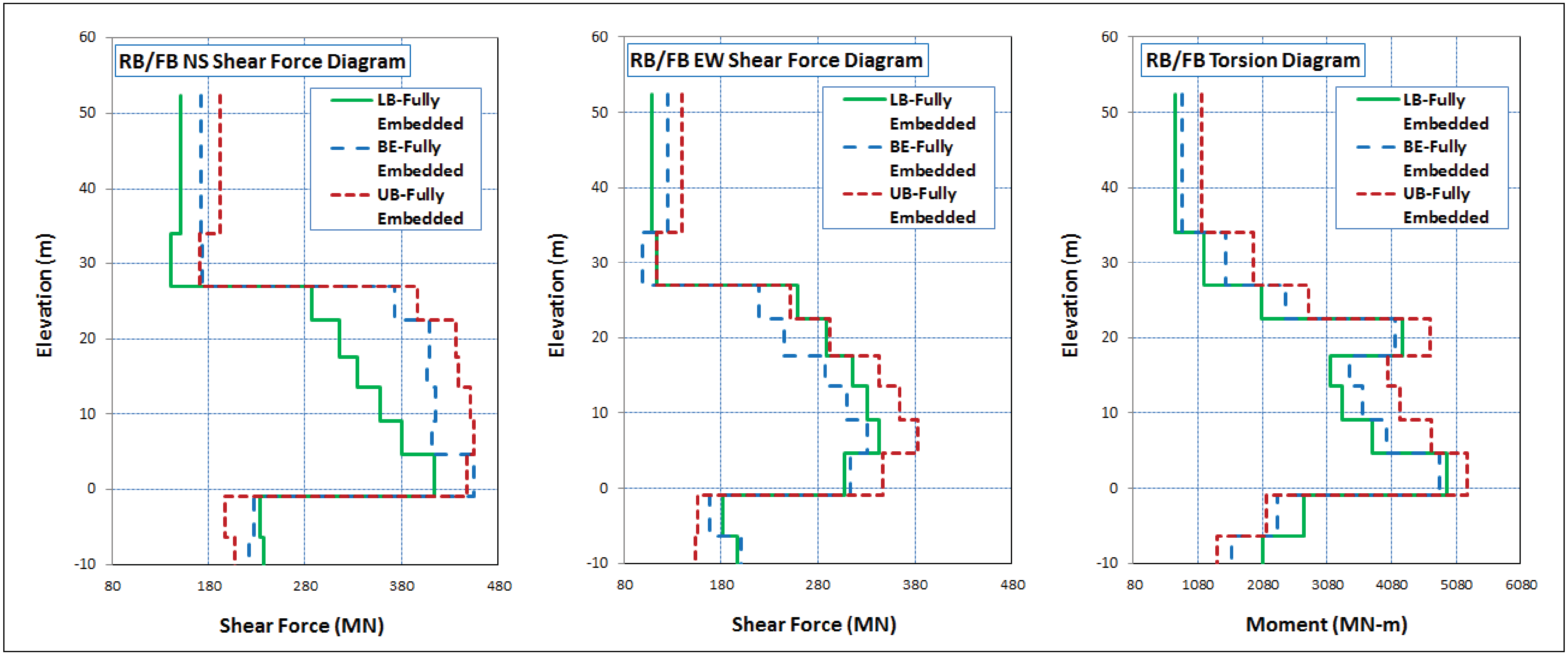
NAPS DEP 3.7-1

Figure 3A.17.12.2-203e **Maximum Shear Forces and Torsion Results from Analyses of Reactor Shield Wall PE Model**



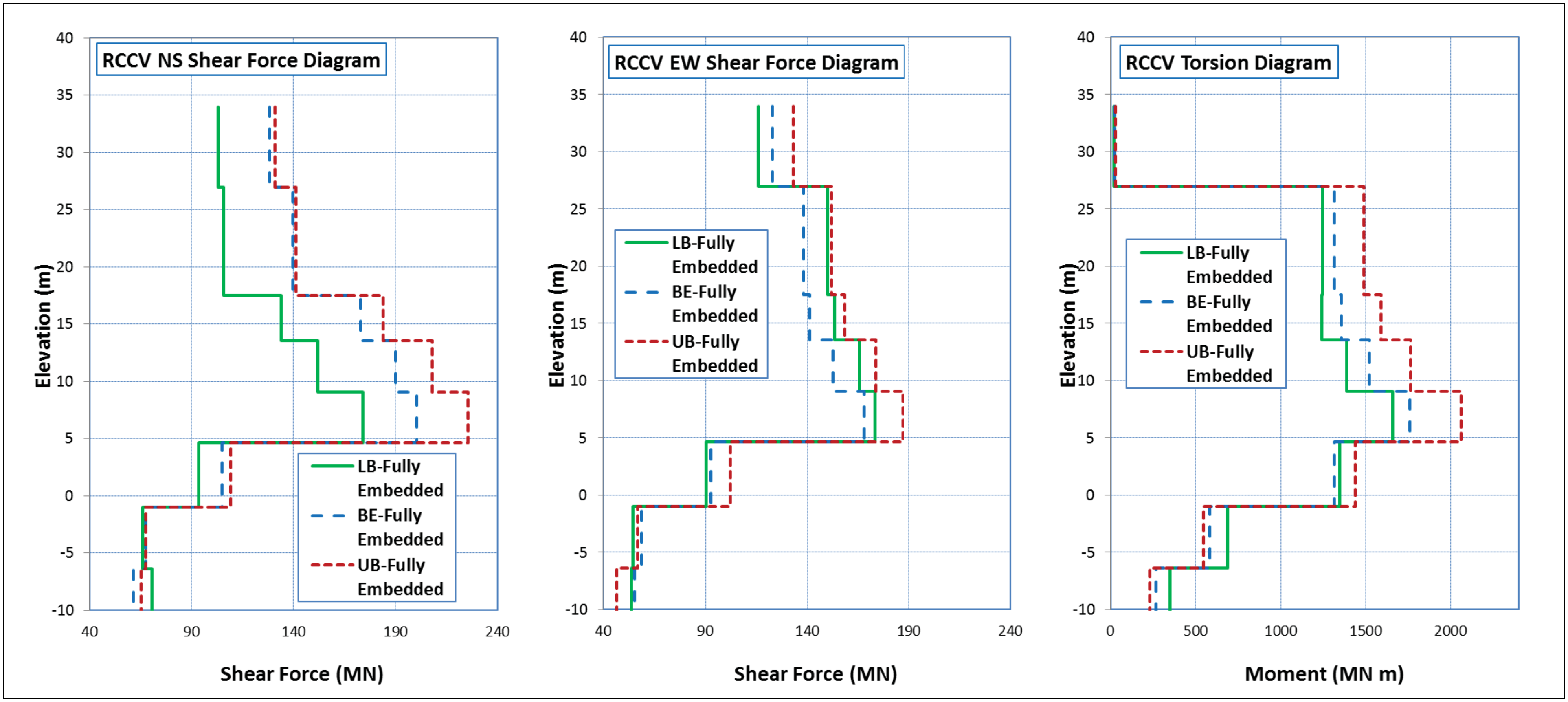
NAPS DEP 3.7-1

Figure 3A.17.12.2-204a Maximum Shear Forces and Torsion Results from Analyses of RB/FB FE Model



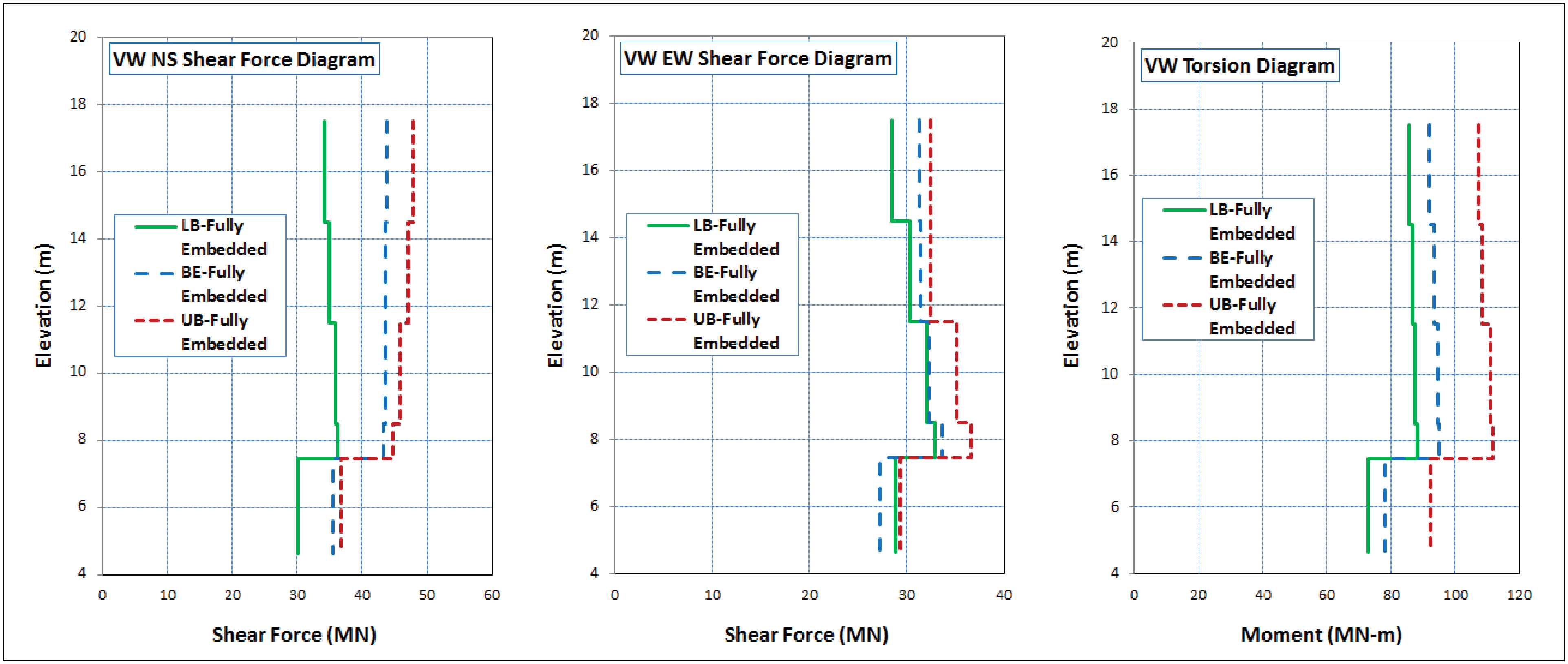
NAPS DEP 3.7-1

Figure 3A.17.12.2-204b Maximum Shear Forces and Torsion Results from Analyses of RCCV FE Model



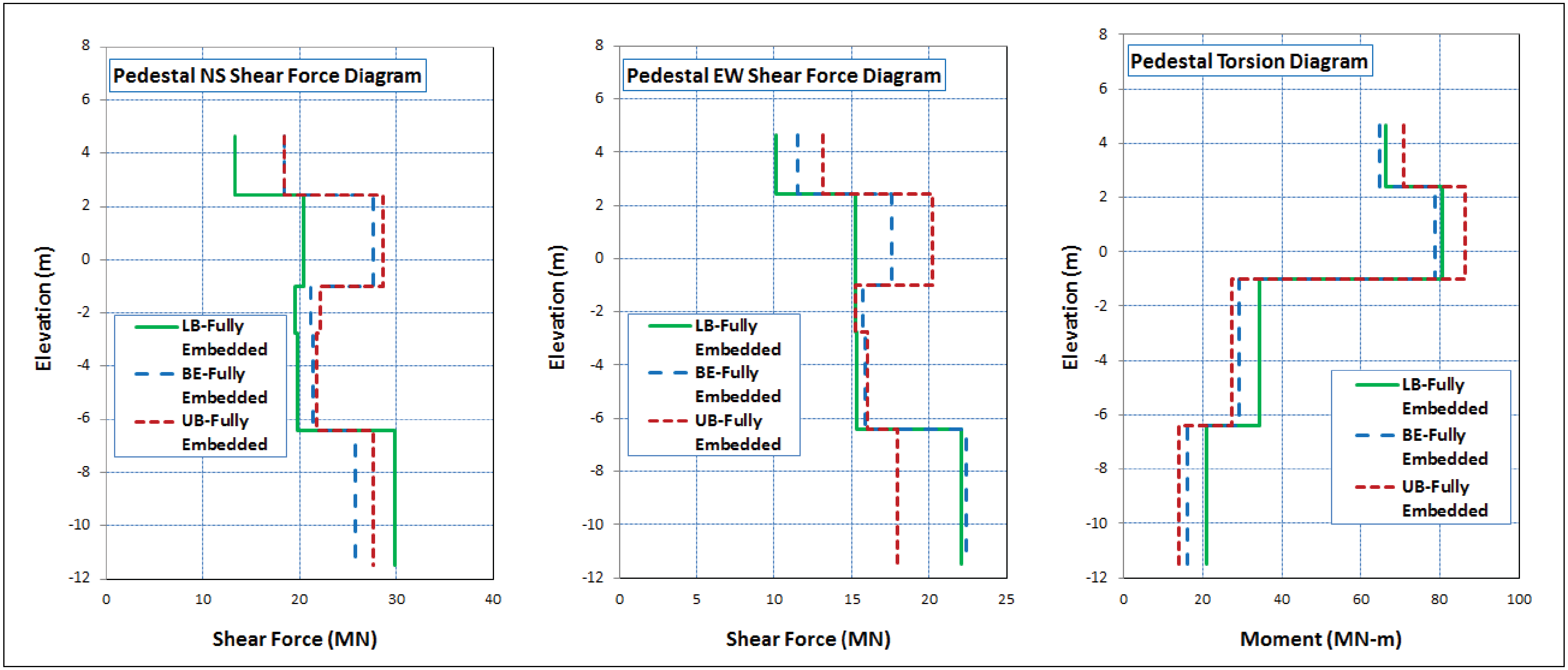
NAPS DEP 3.7-1

Figure 3A.17.12.2-204c **Maximum Shear Forces and Torsion Results from Analyses of Vent Wall FE Model**



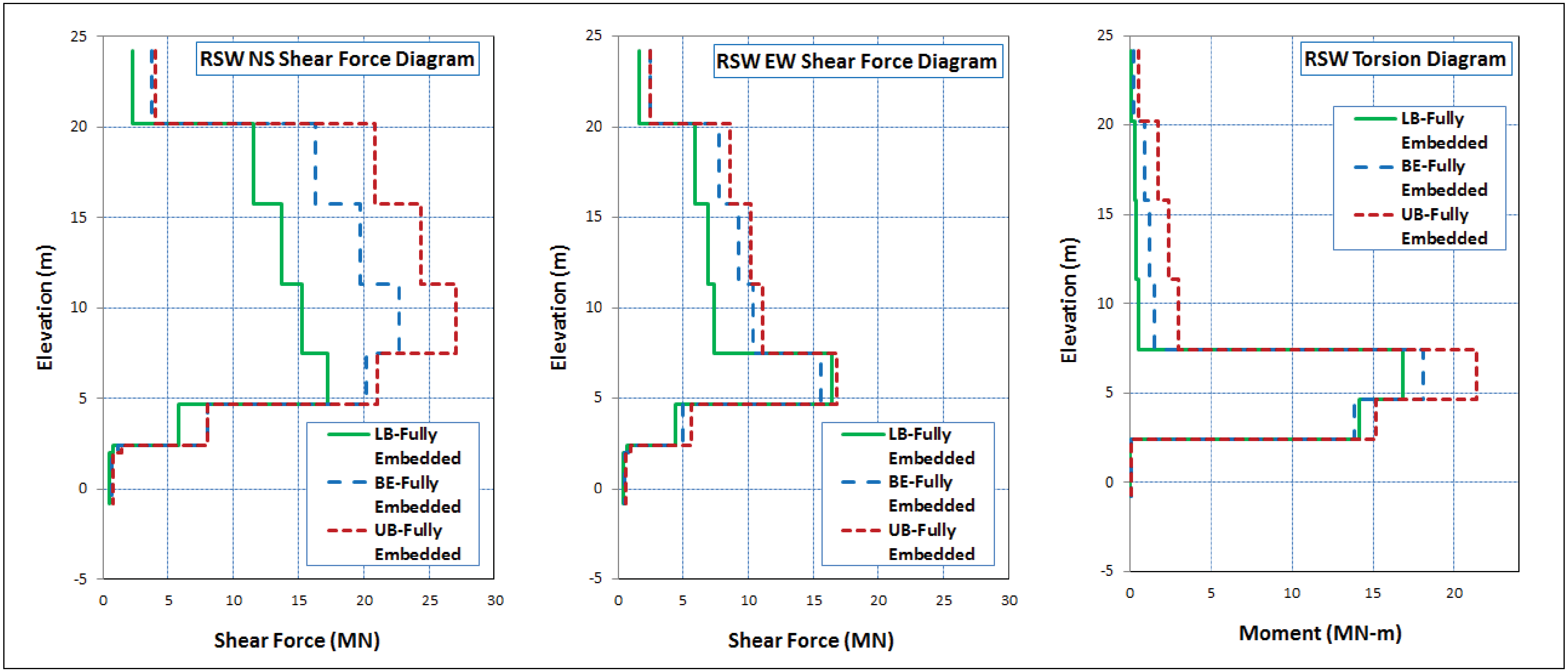
NAPS DEP 3.7-1

Figure 3A.17.12.2-204d Maximum Shear Forces and Torsion Results from Analyses of Pedestal FE Model



NAPS DEP 3.7-1

Figure 3A.17.12.2-204e **Maximum Shear Forces and Torsion Results from Analyses of Reactor Shield Wall FE Model**



3A.17.12.3 **Acceleration Response Spectra**

Comparisons of the 5 percent damped ARS results are presented for selected locations within the RB/FB and selected slab and wall oscillators at different elevations. Floor ISRS are obtained for particular floor elevations as the envelope of responses at the four outrigger locations. The ISRS for the out-of-plane response of flexible slabs and walls are obtained from the ARS calculated for SDOF oscillator mass nodes. The ISRS obtained from the analyses of the three orthogonal components of the earthquake motion are combined using the SRSS method.

Figures 3A.17.12.3-201a through 3A.17.12.3-201f, 3A.17.12.3-202a through 3A.17.12.3-202f, and 3A.17.12.3-203a through 3A.17.12.3-203f compare the 5 percent damped ISRS for the response in NS(x), EW(y) and vertical (z) directions at key locations within the RB/FB, obtained from the SSI analyses of the RB/FB model, with UB stiffness properties and OBE damping values (Cases 1 to 6 in Table 3A.15-201) with the corresponding 5 percent damped standard design enveloping ISRS in DCD Section 3A.9.

The comparison of the ISRS results obtained from the different site-specific SSI analysis cases shows that the responses obtained from the analyses of the UB subgrade profiles govern the ISRS with the exception of narrow frequency intervals (< 25 Hz) where the analyses of the BE and LB profiles can be bounding. The results from the analyses of the full column profiles bound the horizontal ISRS for frequencies between 3 Hz and 8 Hz and, in general, bound the vertical ISRS for frequencies between 3 Hz and 15 Hz, reflecting the higher energy content of the full column input motion at lower frequencies.

The comparisons in Figures 3A.17.12.3-201a through 3A.17.12.3-203f indicate that the site-specific ISRS exceed the corresponding standard design ISRS, mainly above 10 Hz where the site-specific FIRS exceed the CSDRS. The peak exceedances occur in the site-specific horizontal ISRS close to the natural frequencies of the containment internal structures (10 Hz to 30 Hz). The site-specific vertical ISRS exceed the standard design ISRS at frequencies above 10 Hz.

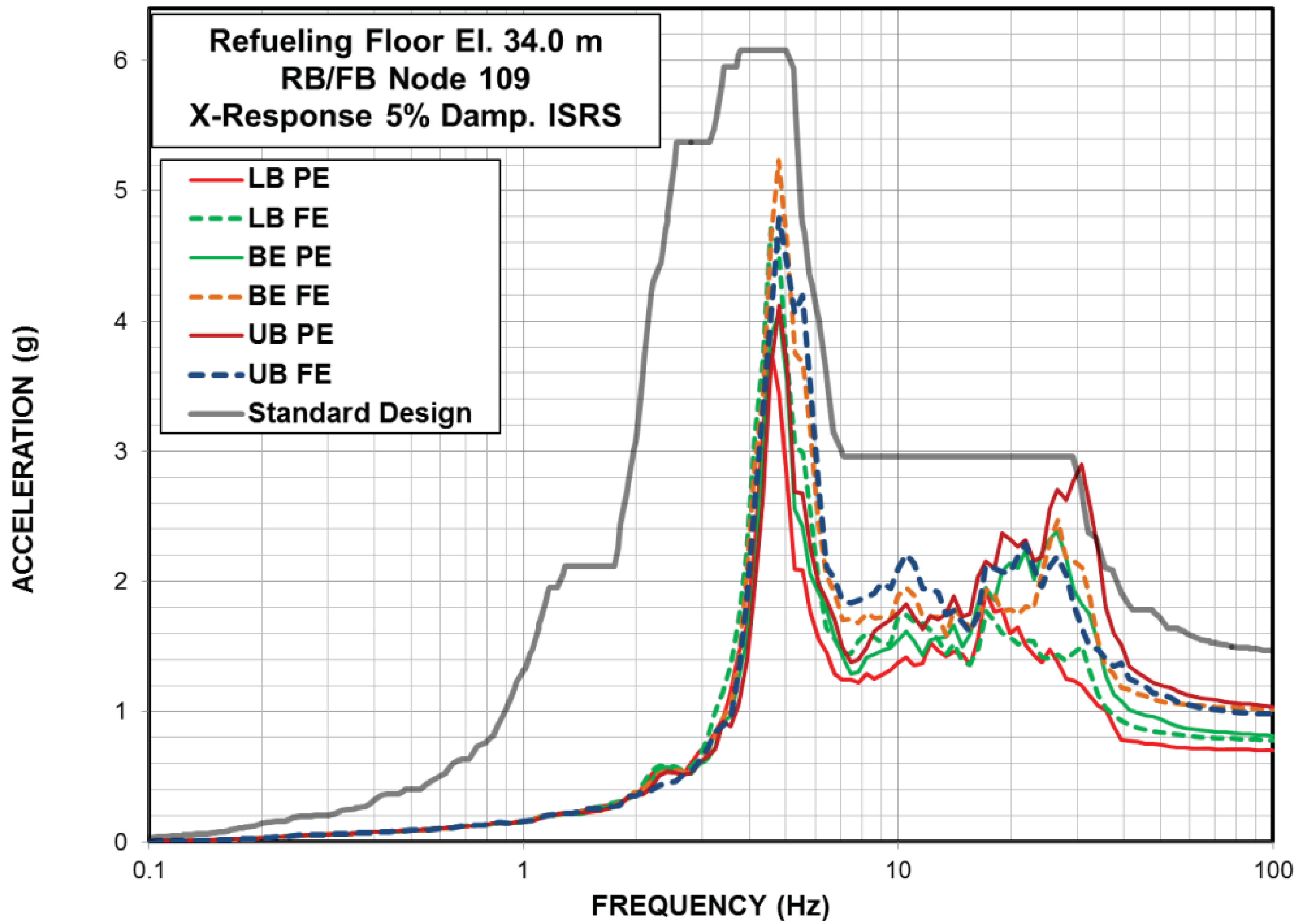
Additionally, a review of site-specific SSI analyses results indicates that there are exceedances in SDOF oscillator ISRS representing the out-of-plane response of RB/FB flexible slabs and walls that occur mainly at frequencies corresponding to the frequencies of the oscillators.

Figures 3A.17.12.3-204a through 3A.17.12.3-204c show selected examples of slab and wall oscillator ISRS.

The site-specific ISRS exceed the standard design enveloping ISRS due to the lower OBE structural damping values used for the site-specific SSI analyses versus the SSE damping values used for standard design SSI analyses. The fact that the Unit 3 site-specific design motion has higher energy content than the standard design CSDRS at frequencies close to the SSI frequencies of the RB/FB structure also explains the exceedances in ISRS.

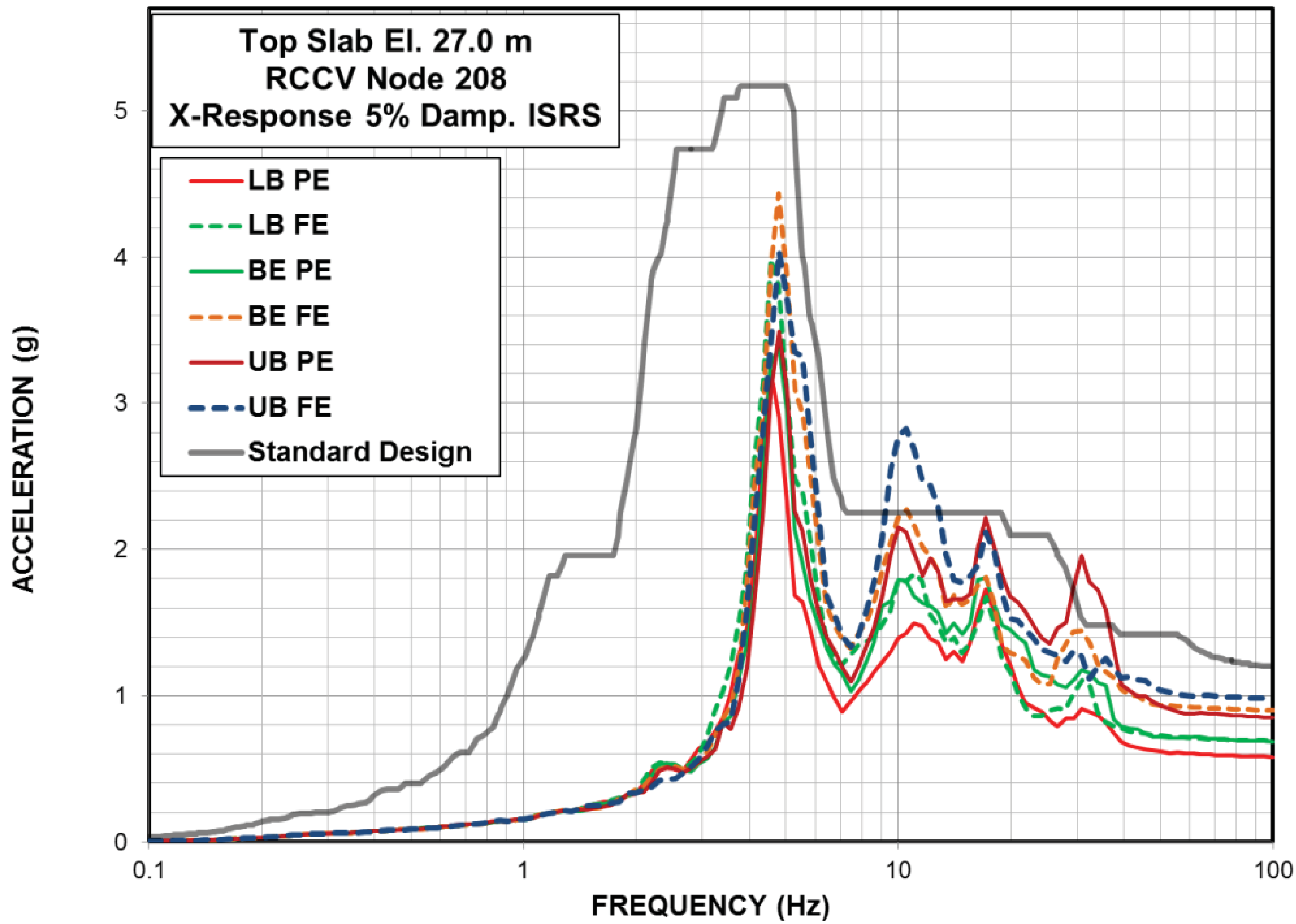
NAPS DEP 3.7-1

Figure 3A.17.12.3-201a **Comparison of ISRS – RB/FB Refueling Floor in X-Direction**



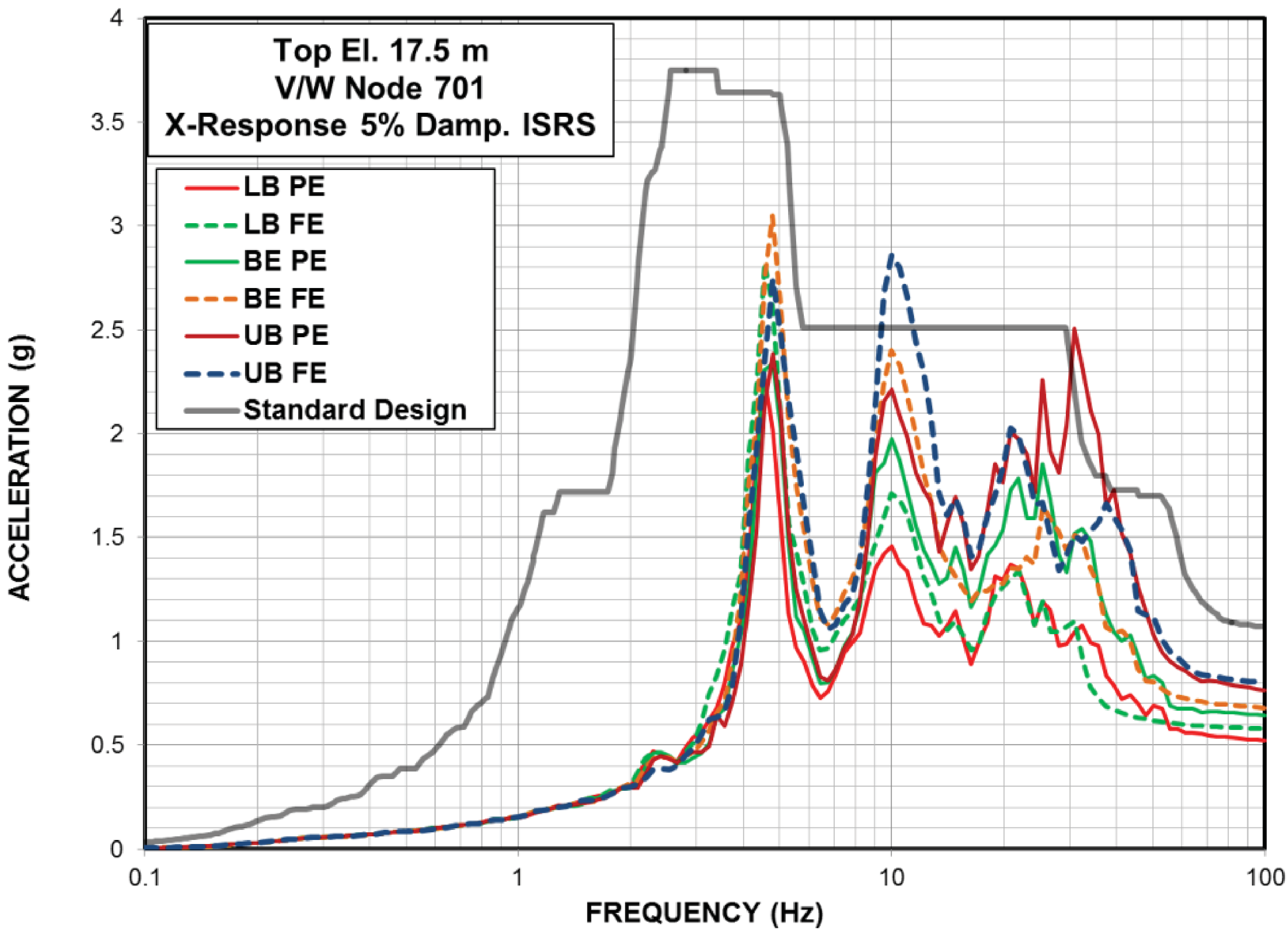
NAPS DEP 3.7-1

Figure 3A.17.12.3-201b **Comparison of ISRS - RCCV Top Slab in X-Direction**



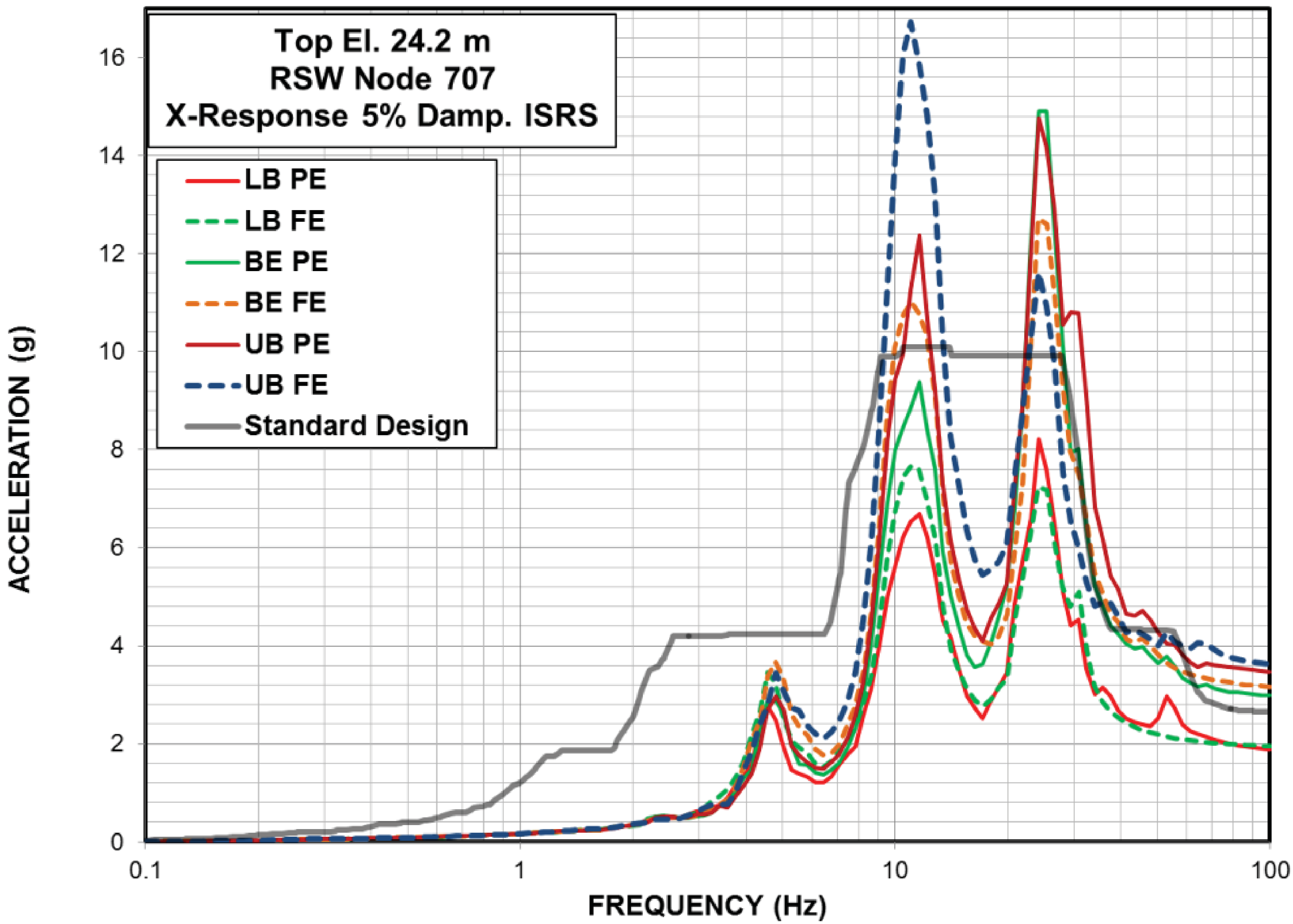
NAPS DEP 3.7-1

Figure 3A.17.12.3-201c **Comparison of ISRS - Vent Wall Top in X-Direction**



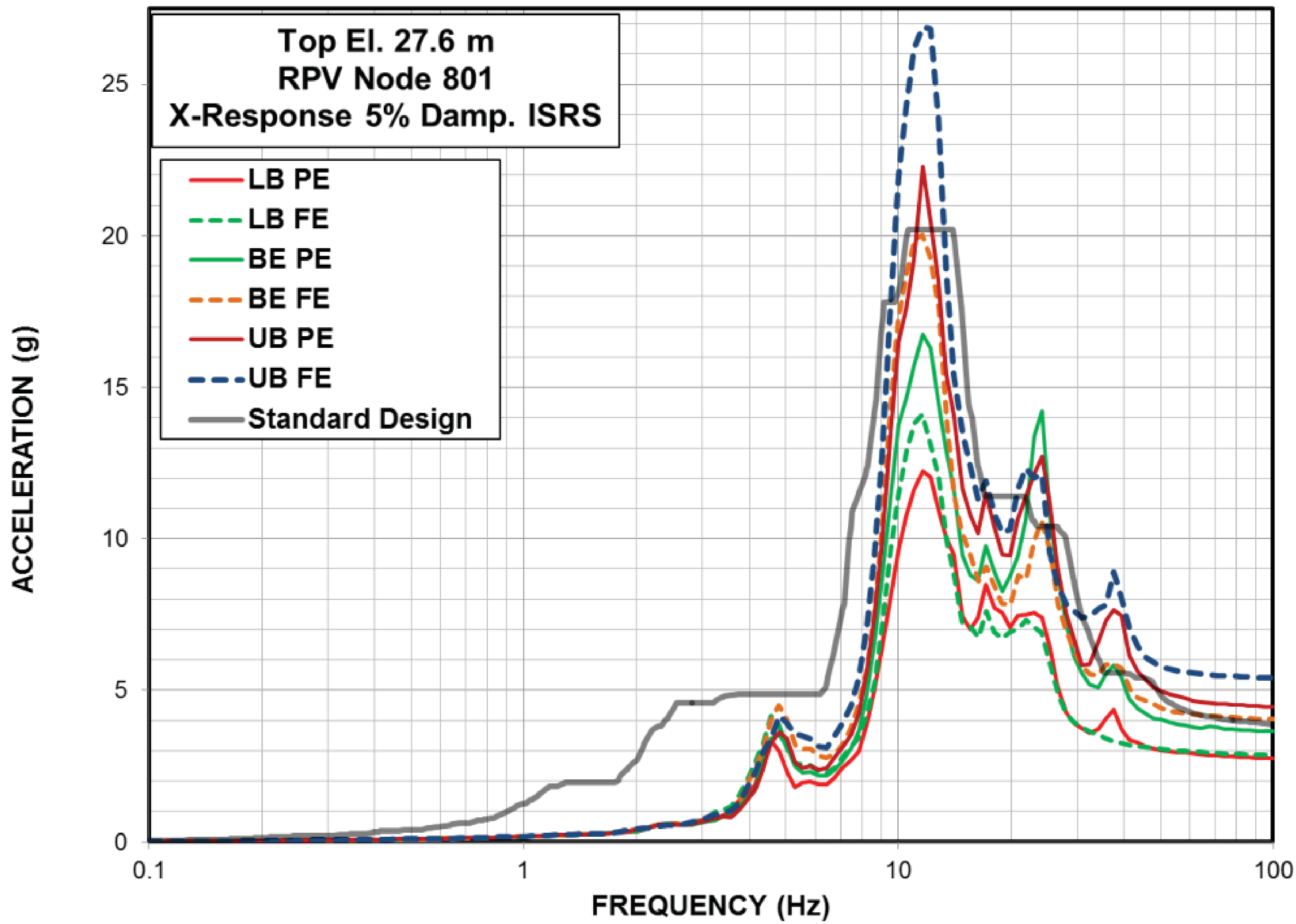
NAPS DEP 3.7-1

Figure 3A.17.12.3-201d **Comparison of ISRS - RSW Top in X-Direction**



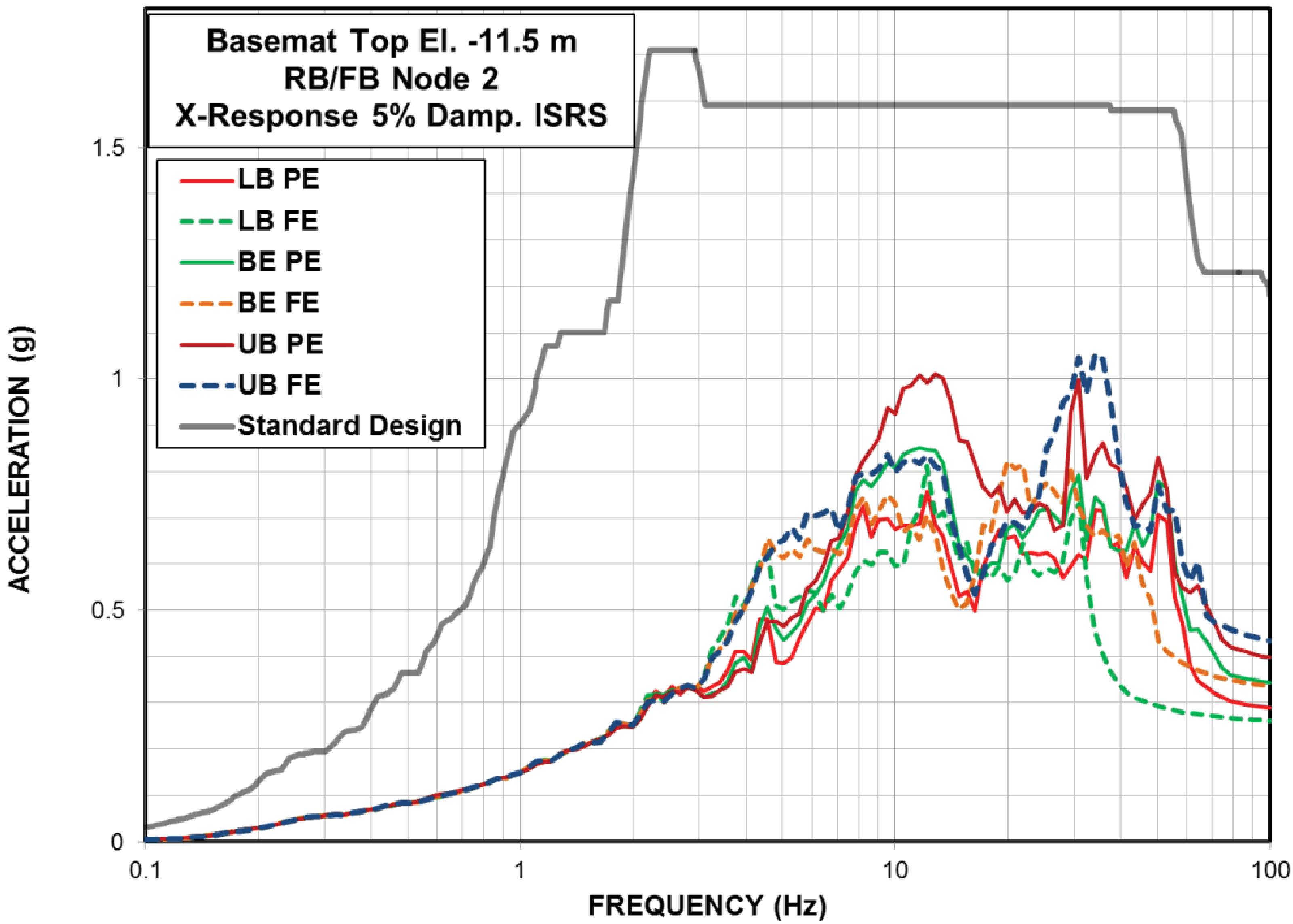
NAPS DEP 3.7-1

Figure 3A.17.12.3-201e **Comparison of ISRS - RPV Top in X-Direction**



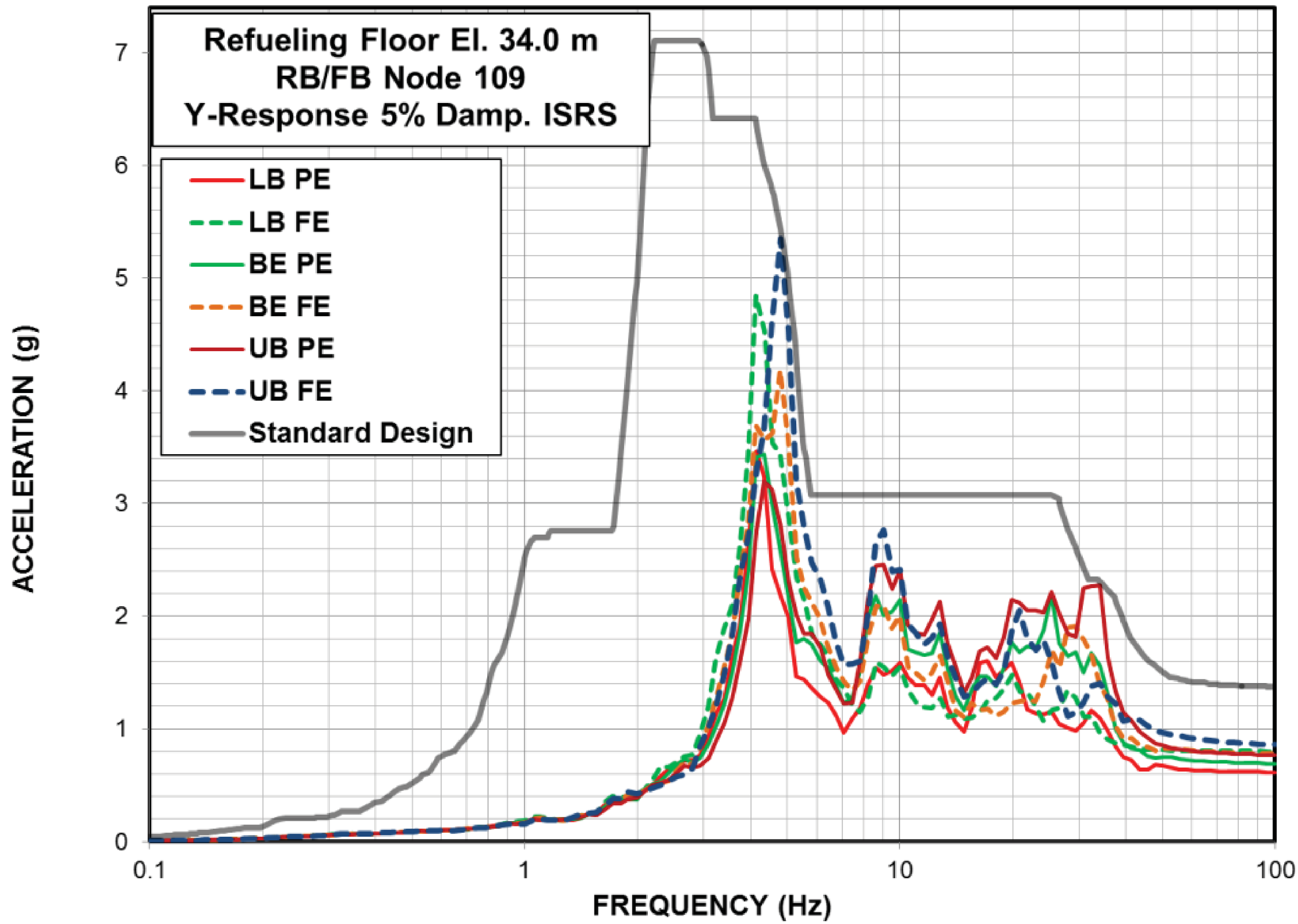
NAPS DEP 3.7-1

Figure 3A.17.12.3-201f **Comparison of ISRS - RB/FB Basemat in X-Direction**



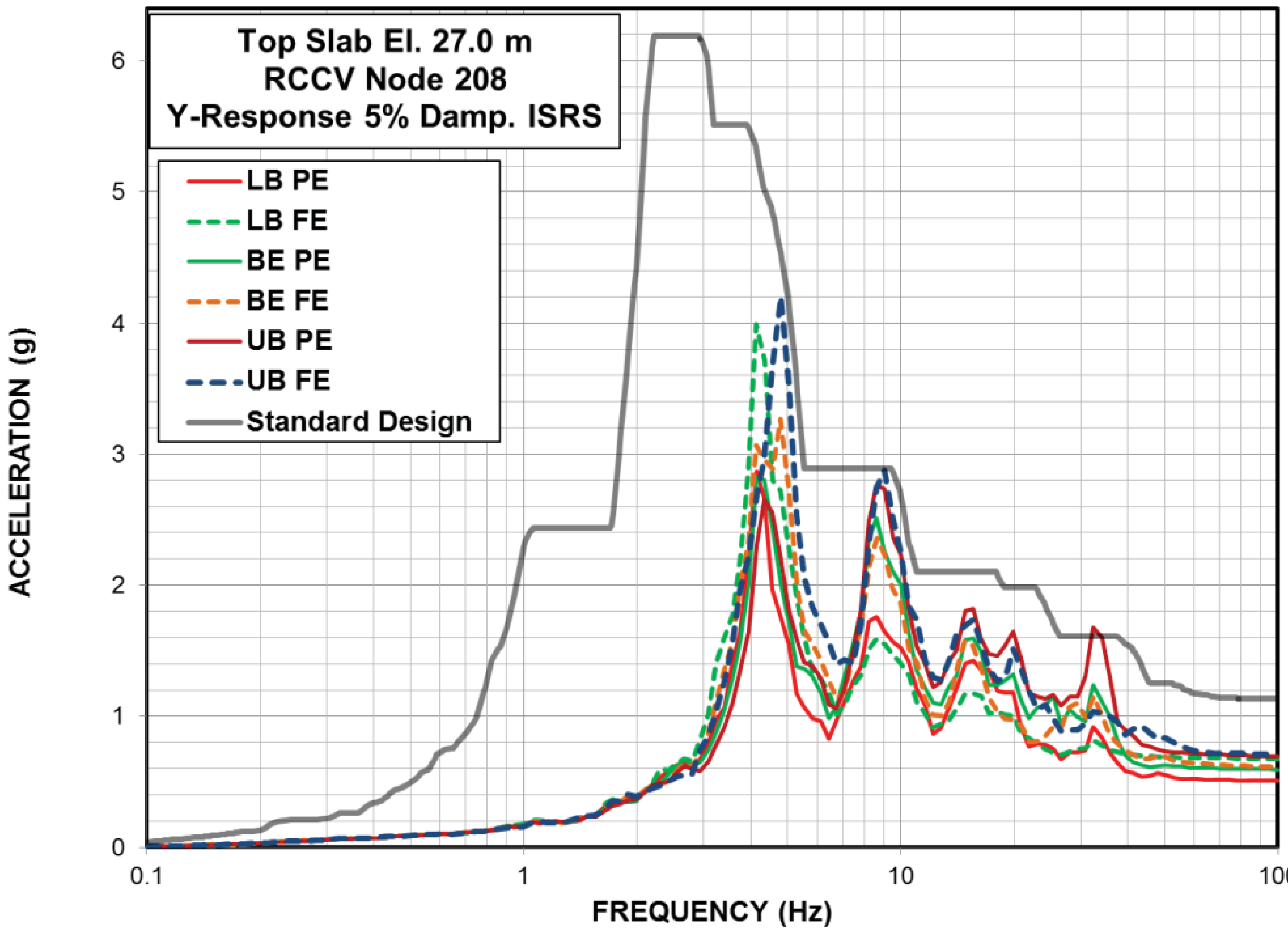
NAPS DEP 3.7-1

Figure 3A.17.12.3-202a **Comparison of ISRS - RB/FB Refueling Floor in Y-Direction**



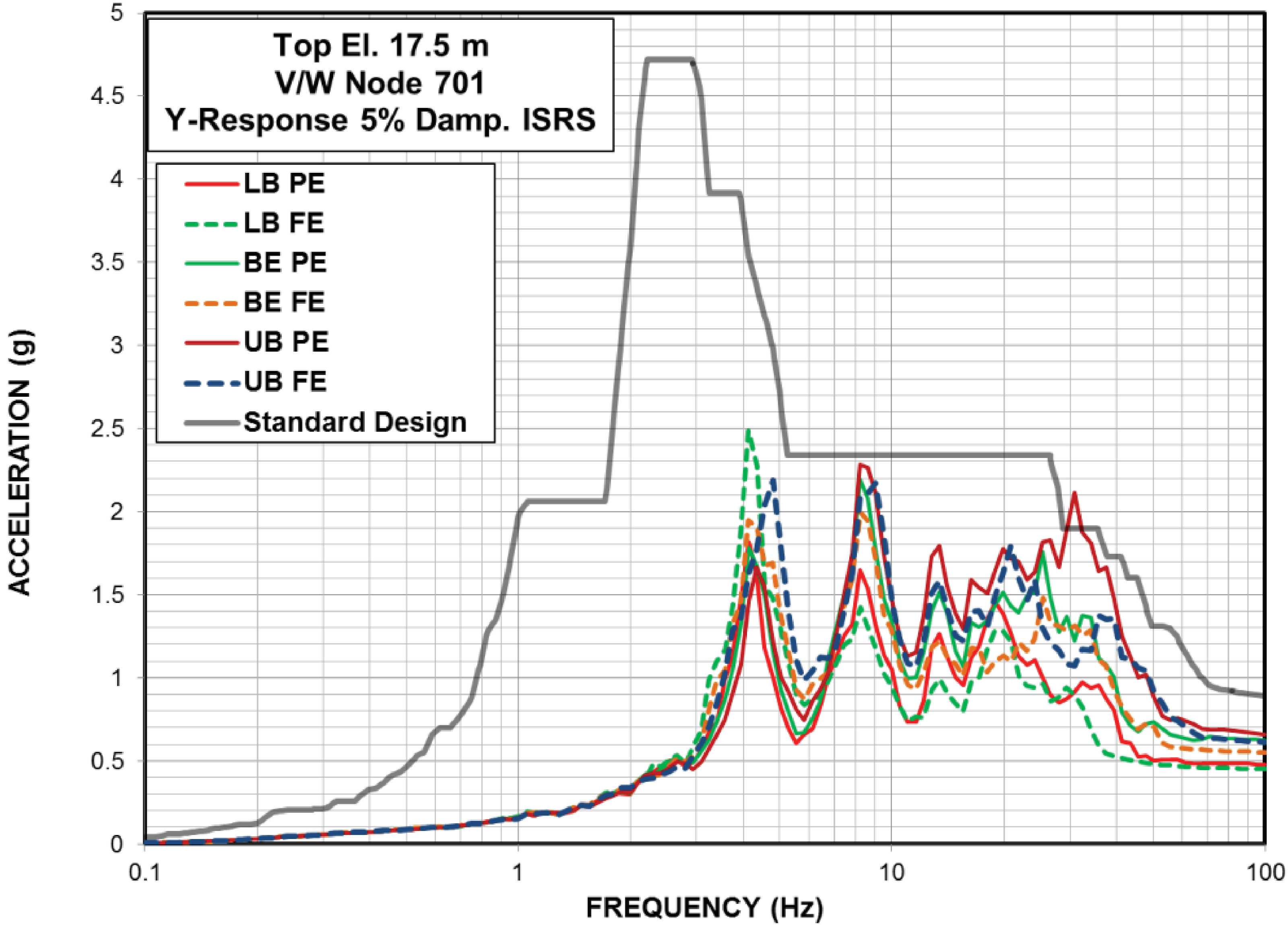
NAPS DEP 3.7-1

Figure 3A.17.12.3-202b **Comparison of ISRS - RCCV Top Slab in Y-Direction**



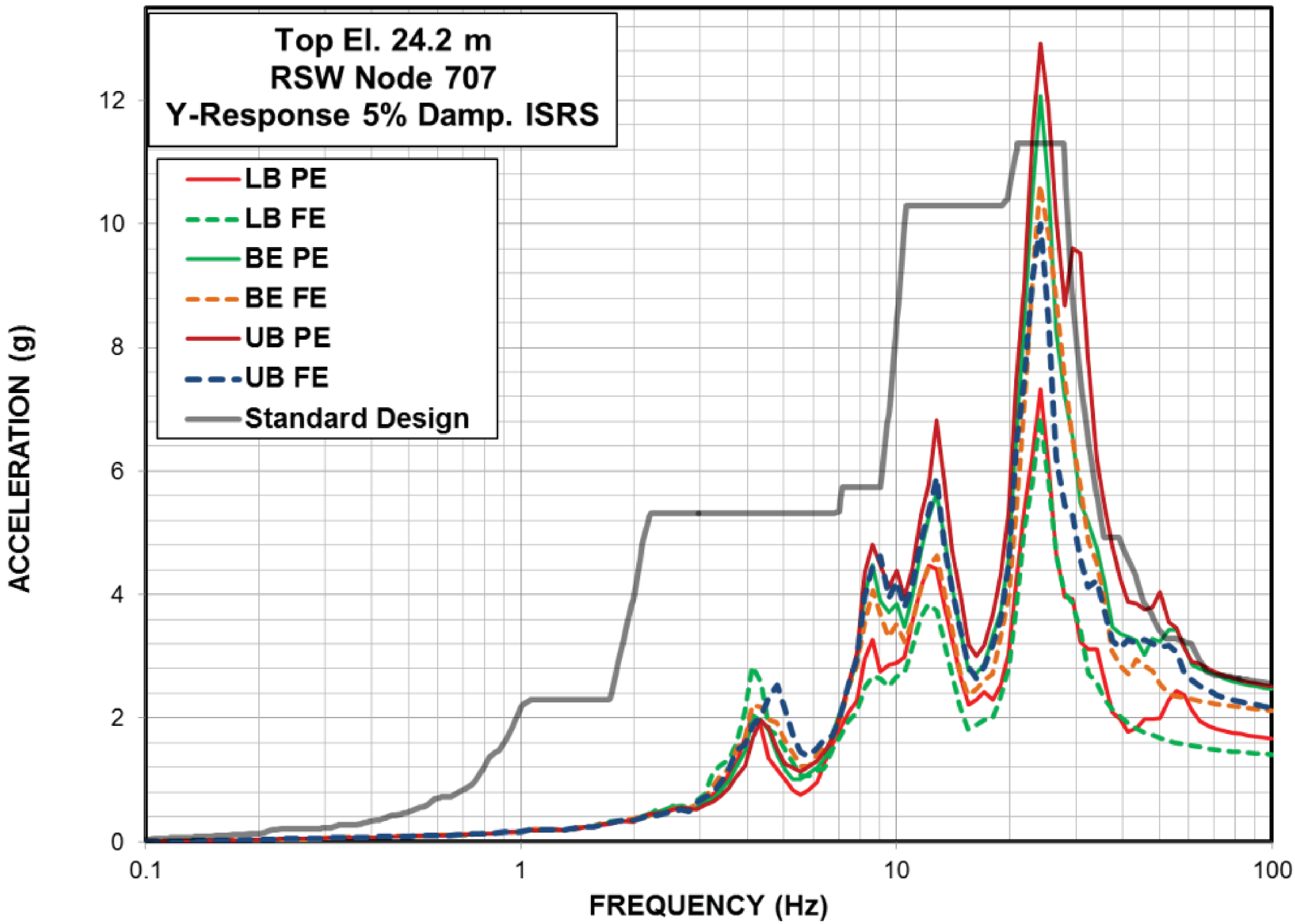
NAPS DEP 3.7-1

Figure 3A.17.12.3-202c **Comparison of ISRS - Vent Wall Top in Y-Direction**



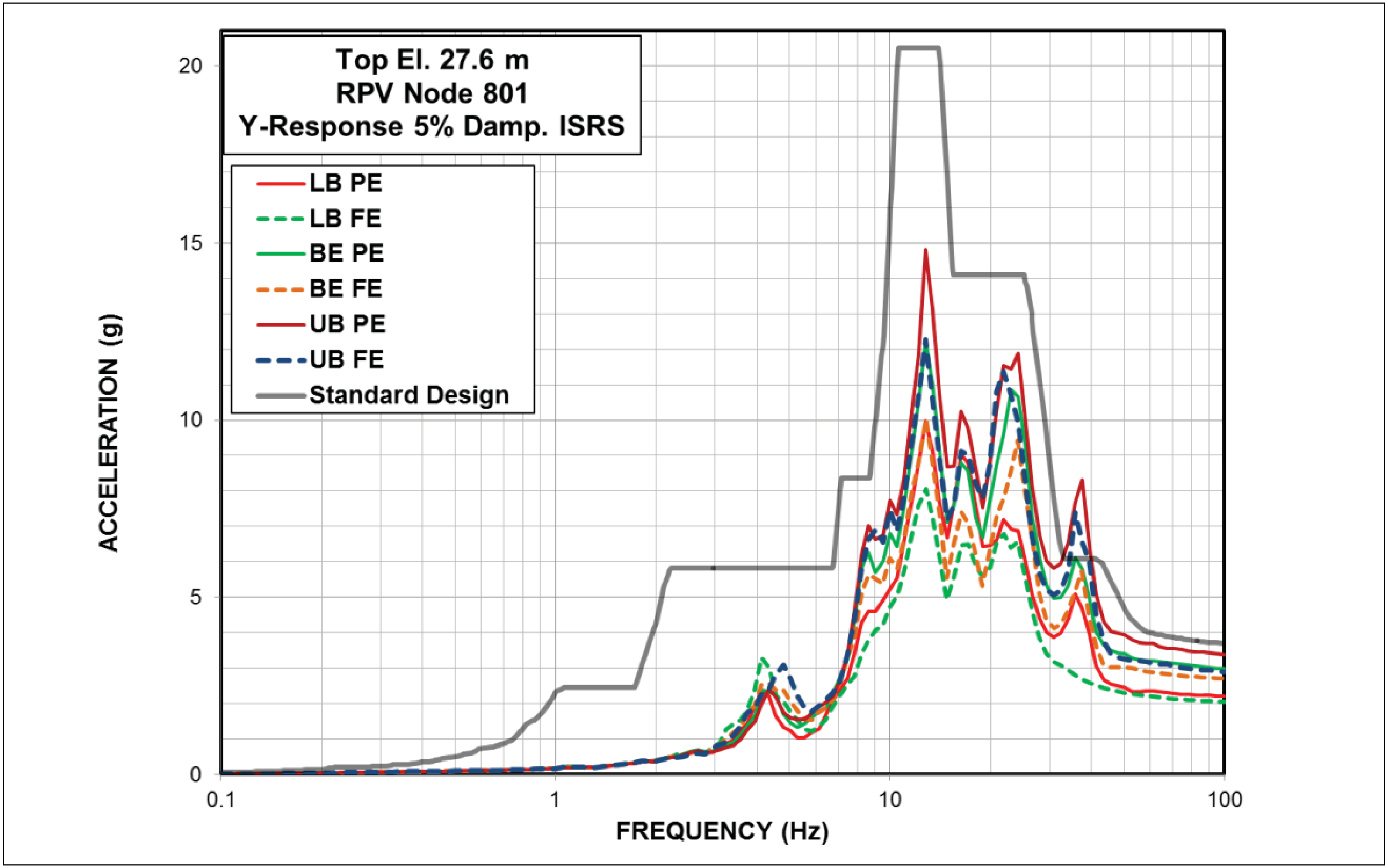
NAPS DEP 3.7-1

Figure 3A.17.12.3-202d **Comparison of ISRS - RSW Top in Y-Direction**



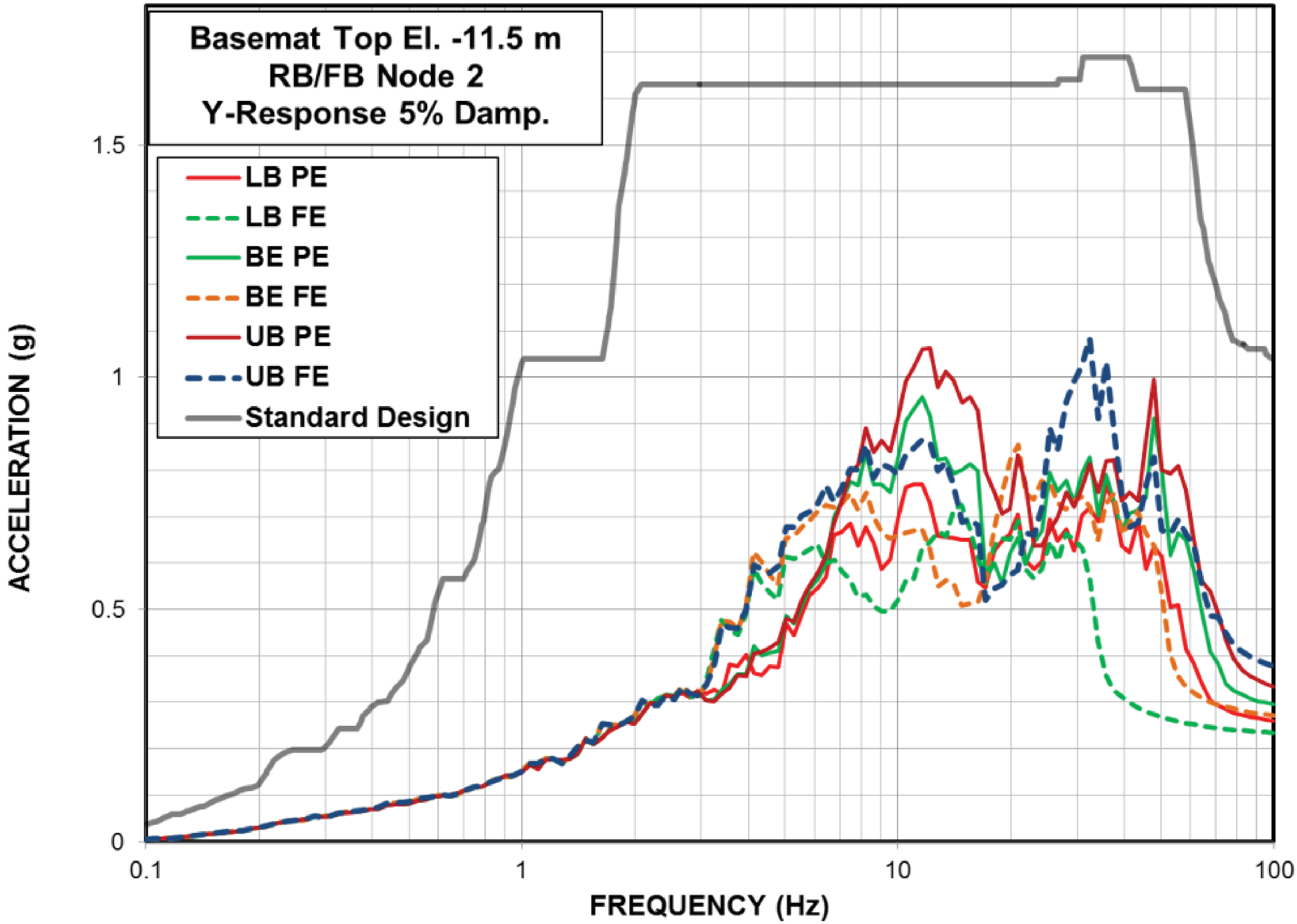
NAPS DEP 3.7-1

Figure 3A.17.12.3-202e **Comparison of ISRS - RPV Top in Y-Direction**



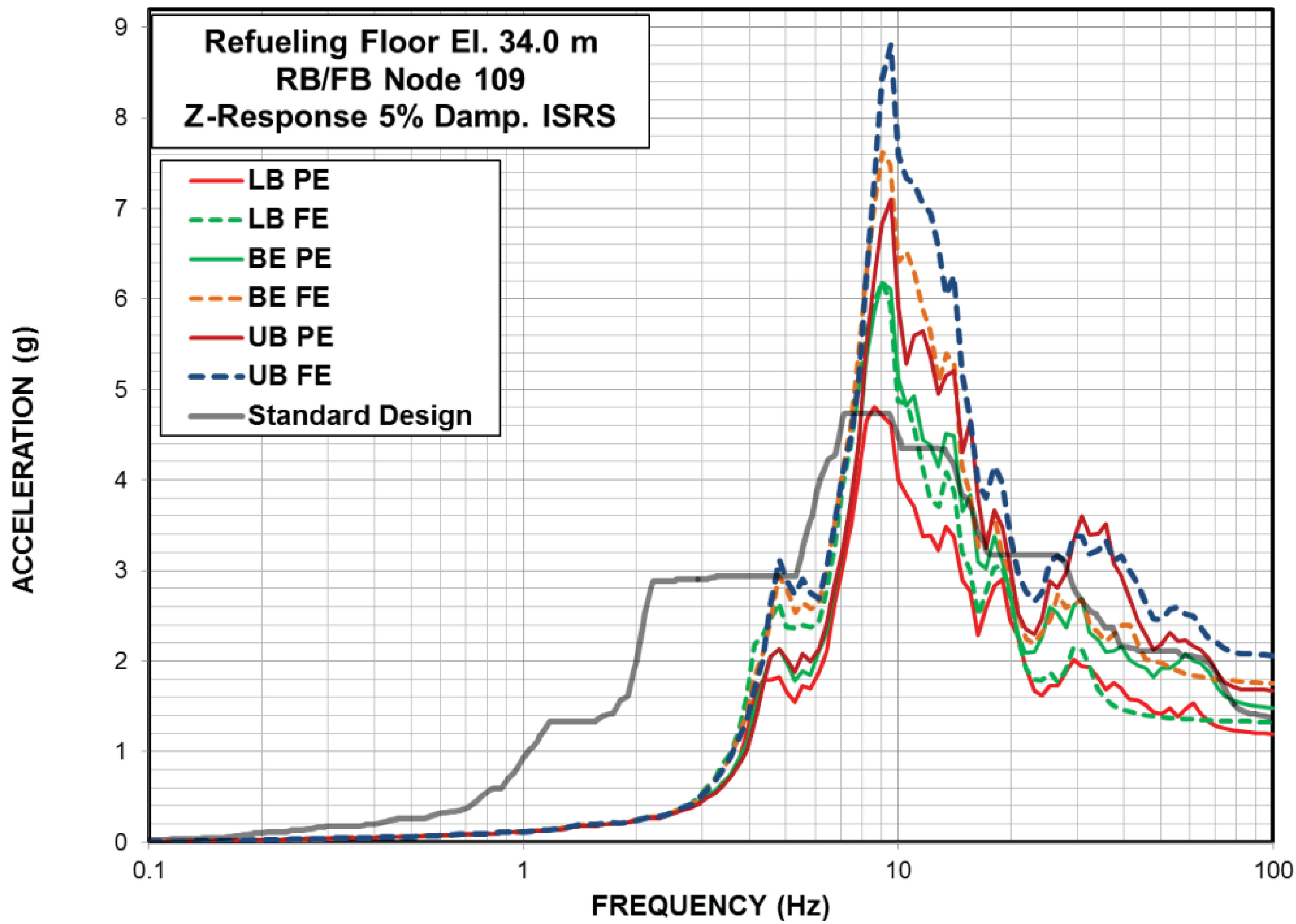
NAPS DEP 3.7-1

Figure 3A.17.12.3-202f Comparison of ISRS - RB/FB Basemat in Y-Direction



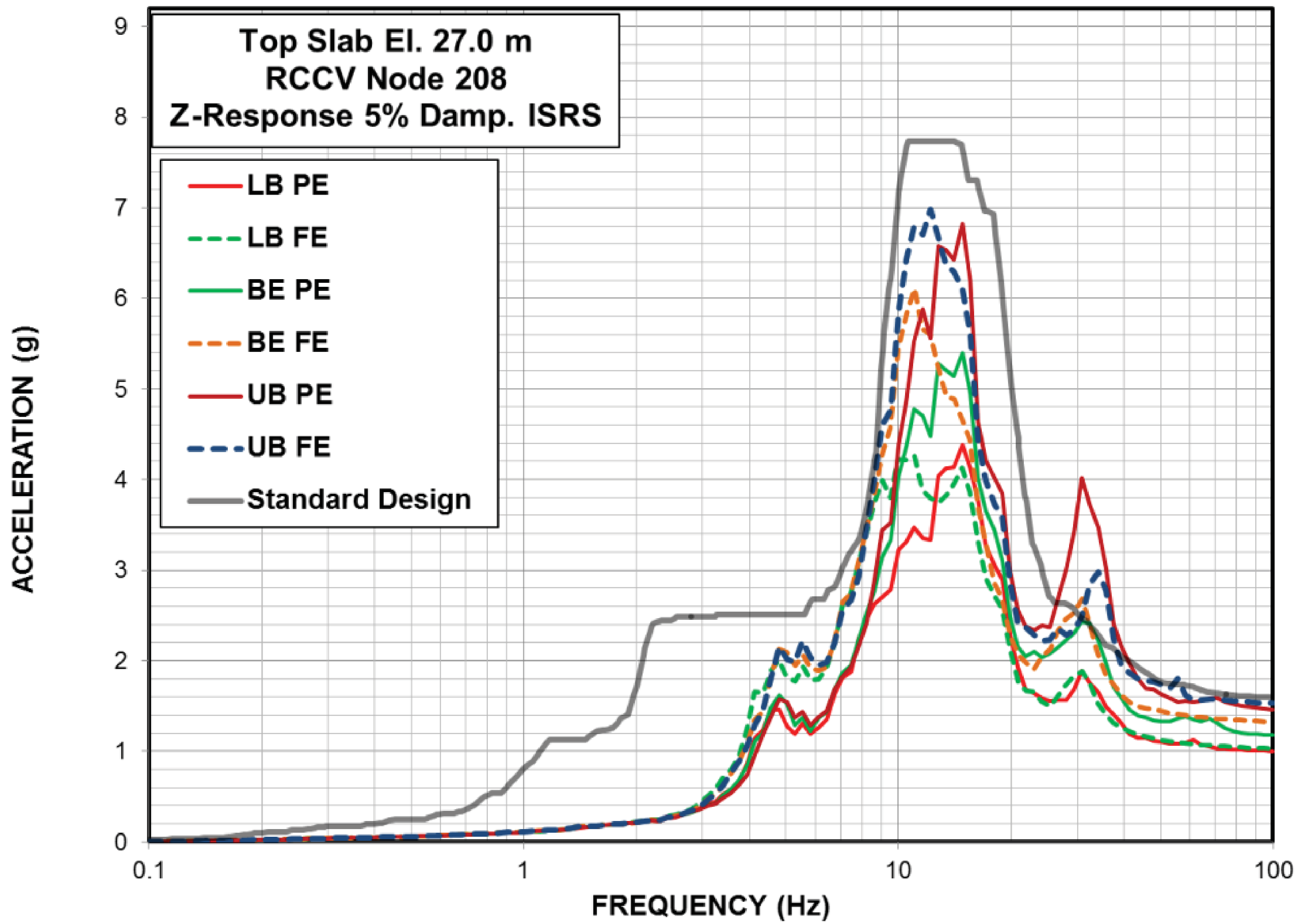
NAPS DEP 3.7-1

Figure 3A.17.12.3-203a **Comparison of ISRS - RB/FB Refueling Floor in Z-Direction**



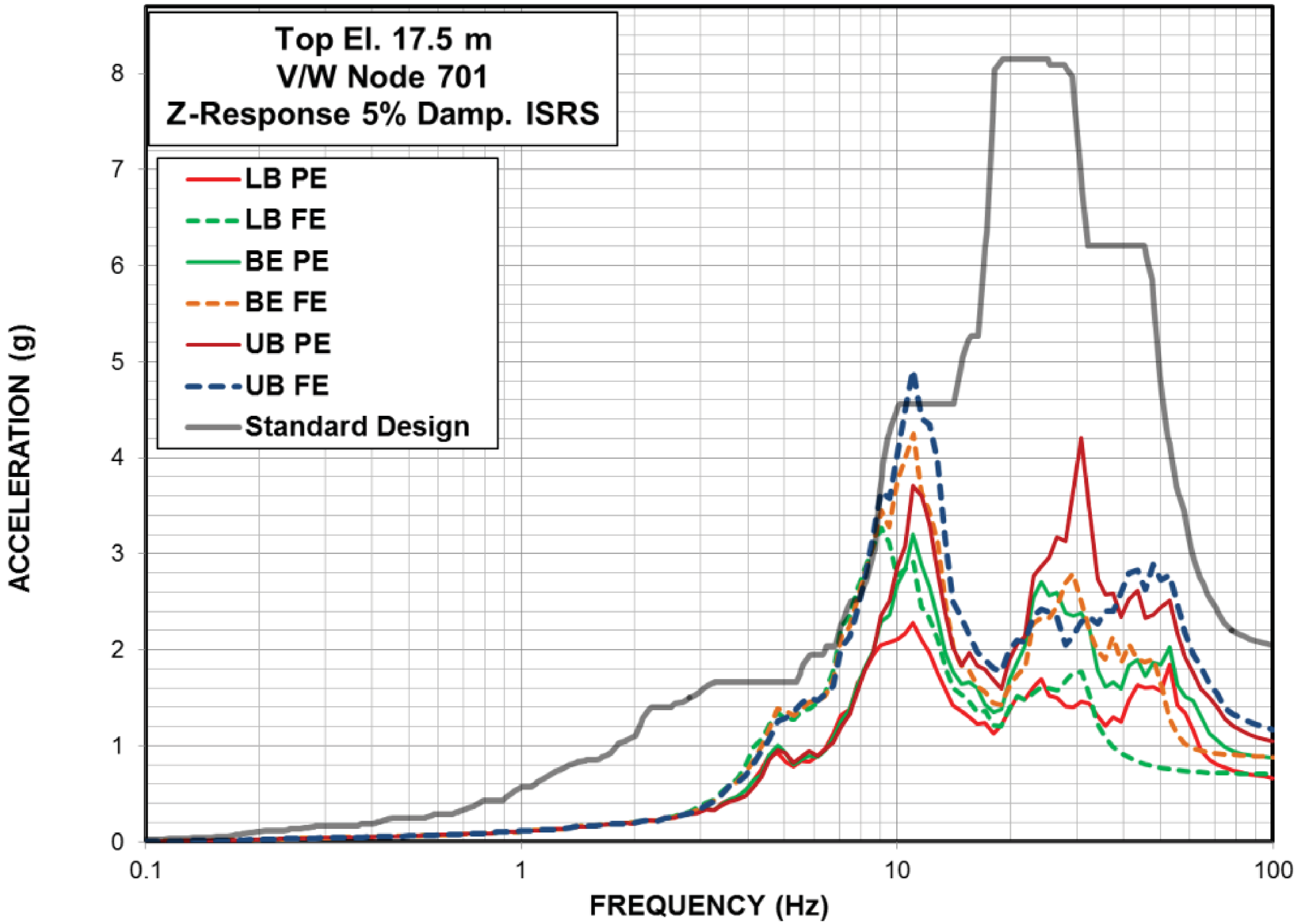
NAPS DEP 3.7-1

Figure 3A.17.12.3-203b **Comparison of ISRS - RCCV Top Slab in Z-Direction**



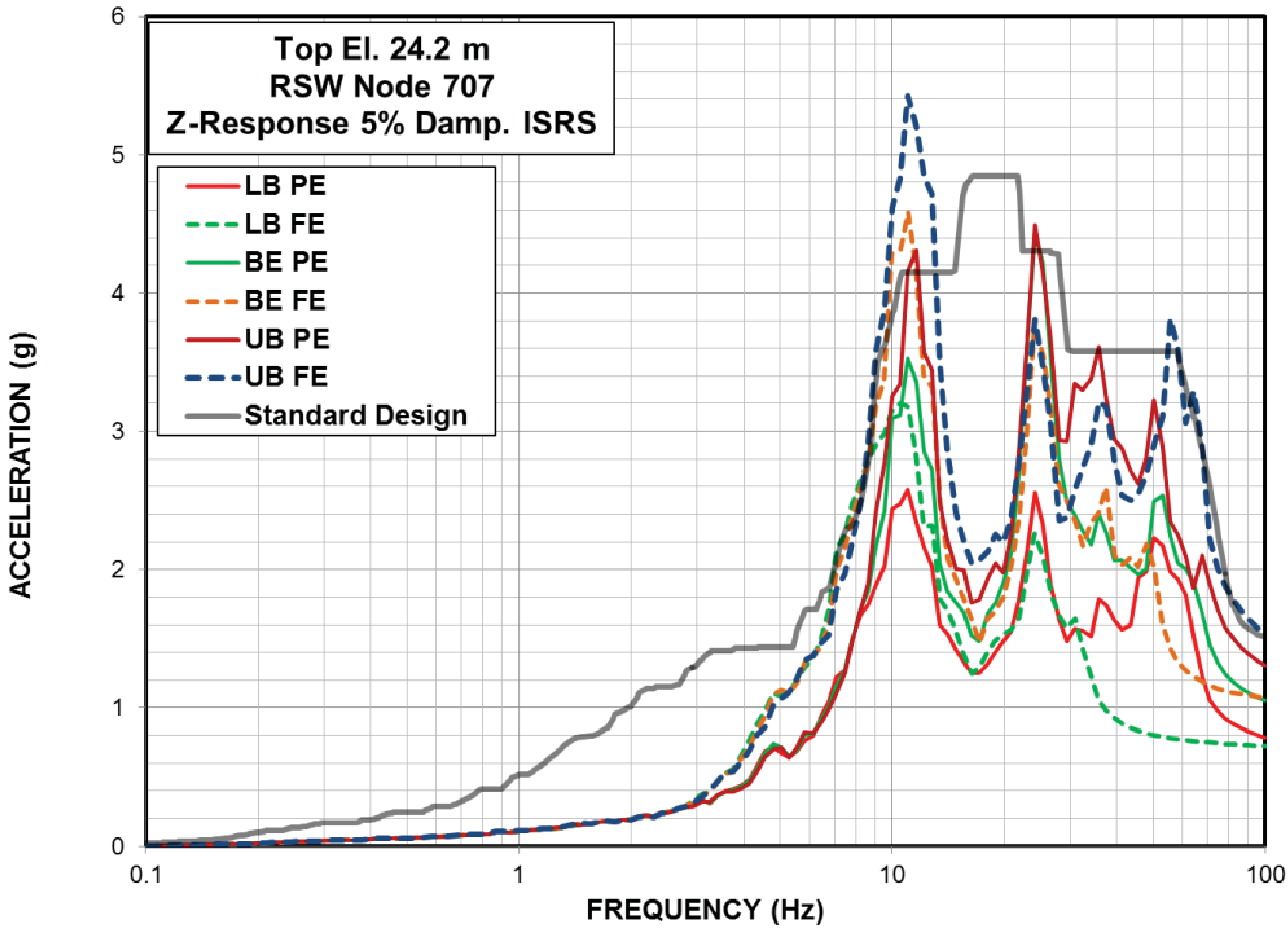
NAPS DEP 3.7-1

Figure 3A.17.12.3-203c **Comparison of ISRS - Vent Wall Top in Z-Direction**



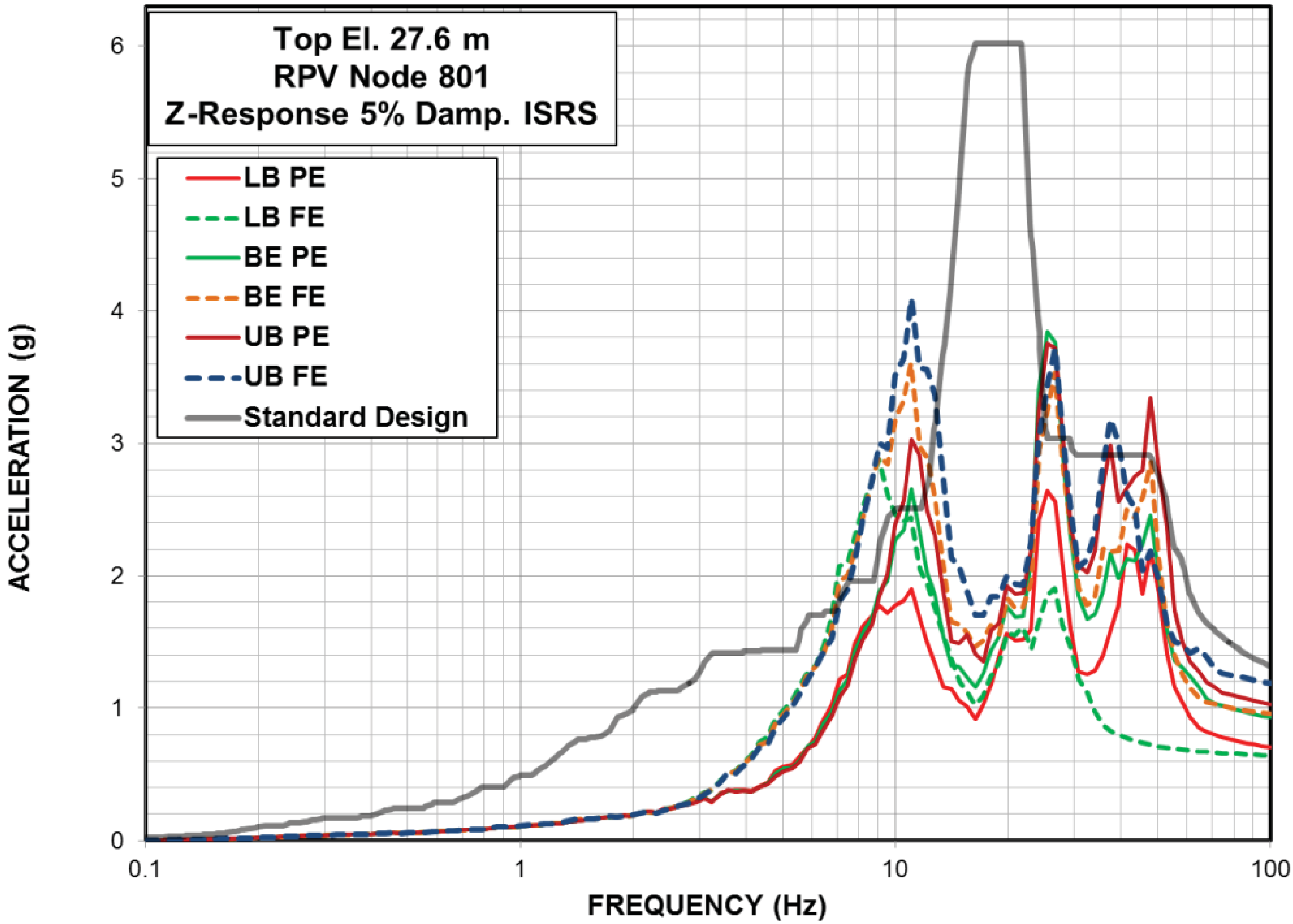
NAPS DEP 3.7-1

Figure 3A.17.12.3-203d **Comparison of ISRS - RSW Top in Z-Direction**



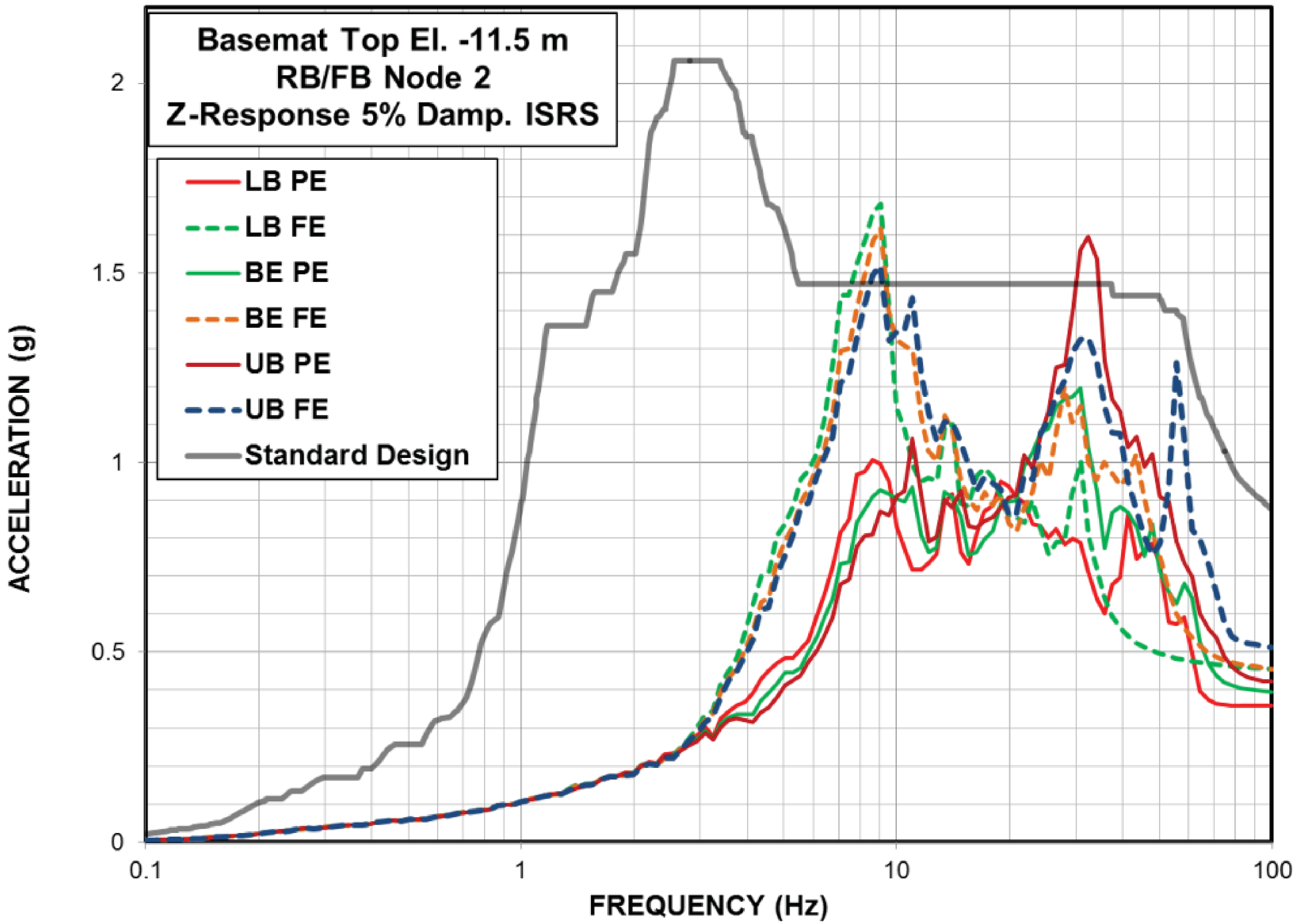
NAPS DEP 3.7-1

Figure 3A.17.12.3-203e **Comparison of ISRS - RPV Top in Z-Direction**

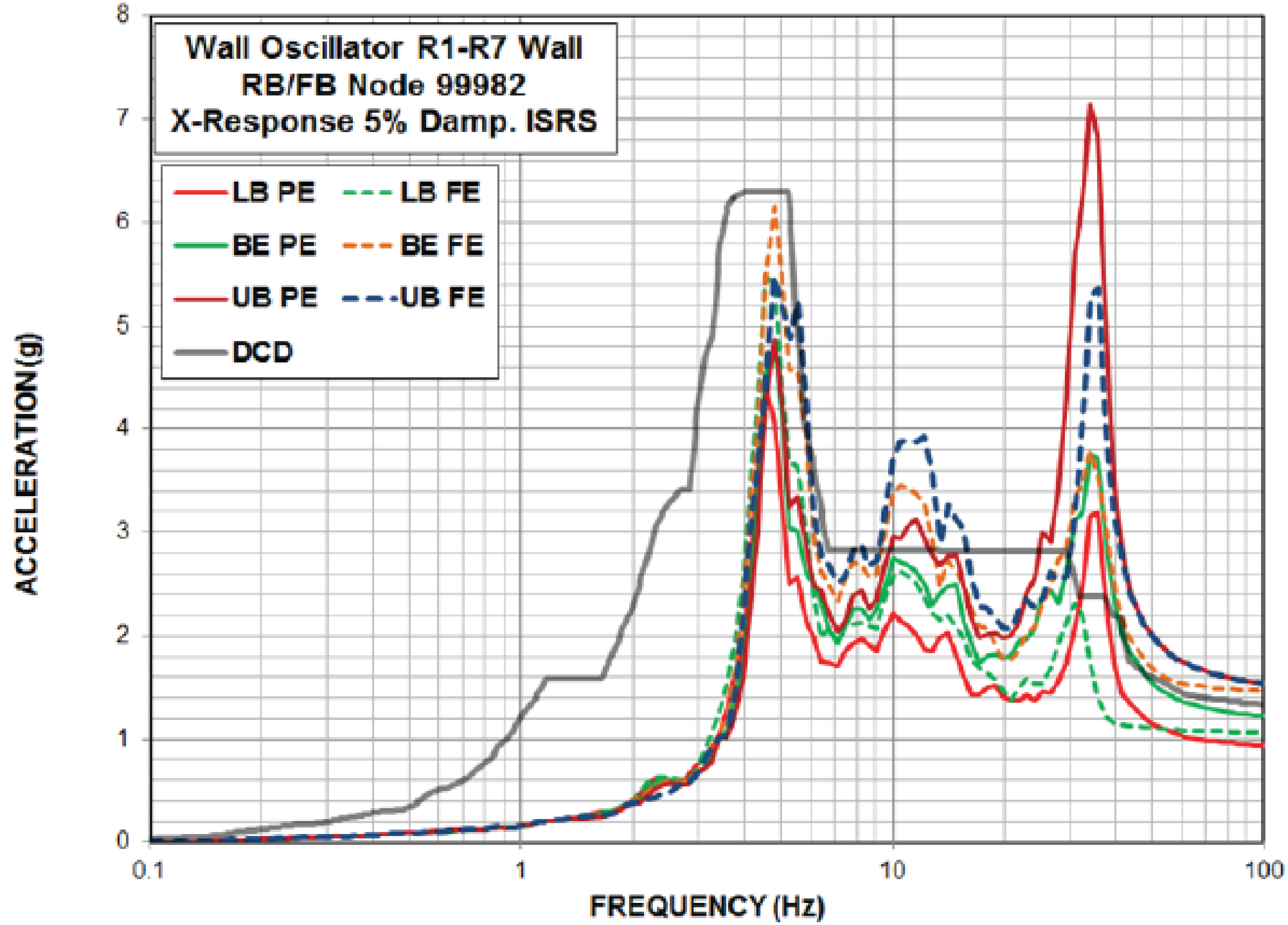


NAPS DEP 3.7-1

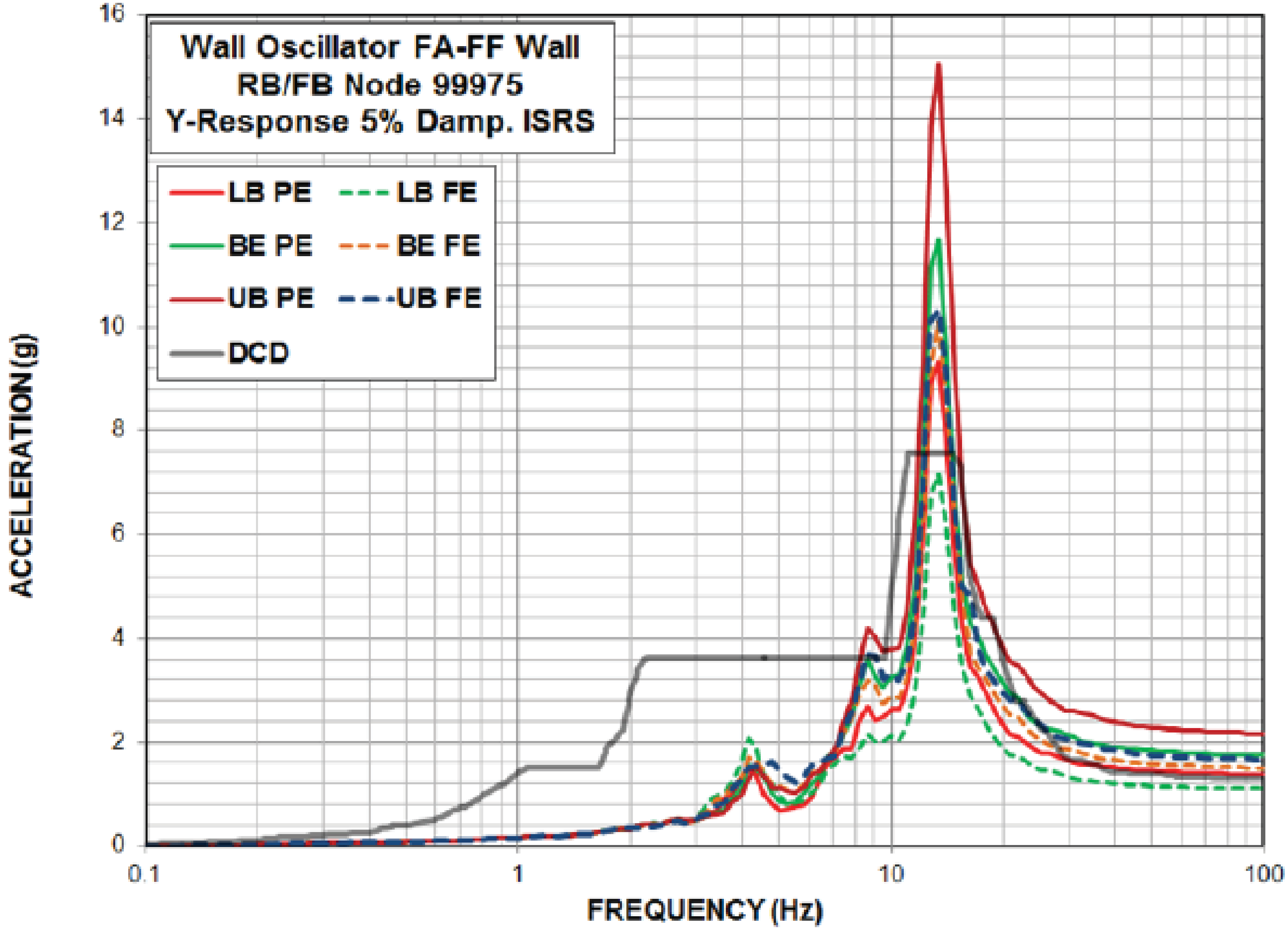
Figure 3A.17.12.3-203f **Comparison of ISRS - RB/FB Basemat in Z-Direction**



NAPS DEP 3.7-1 Figure 3A.17.12.3-204a **Comparison of ISRS - Oscillator Node RB/FB 99982X**

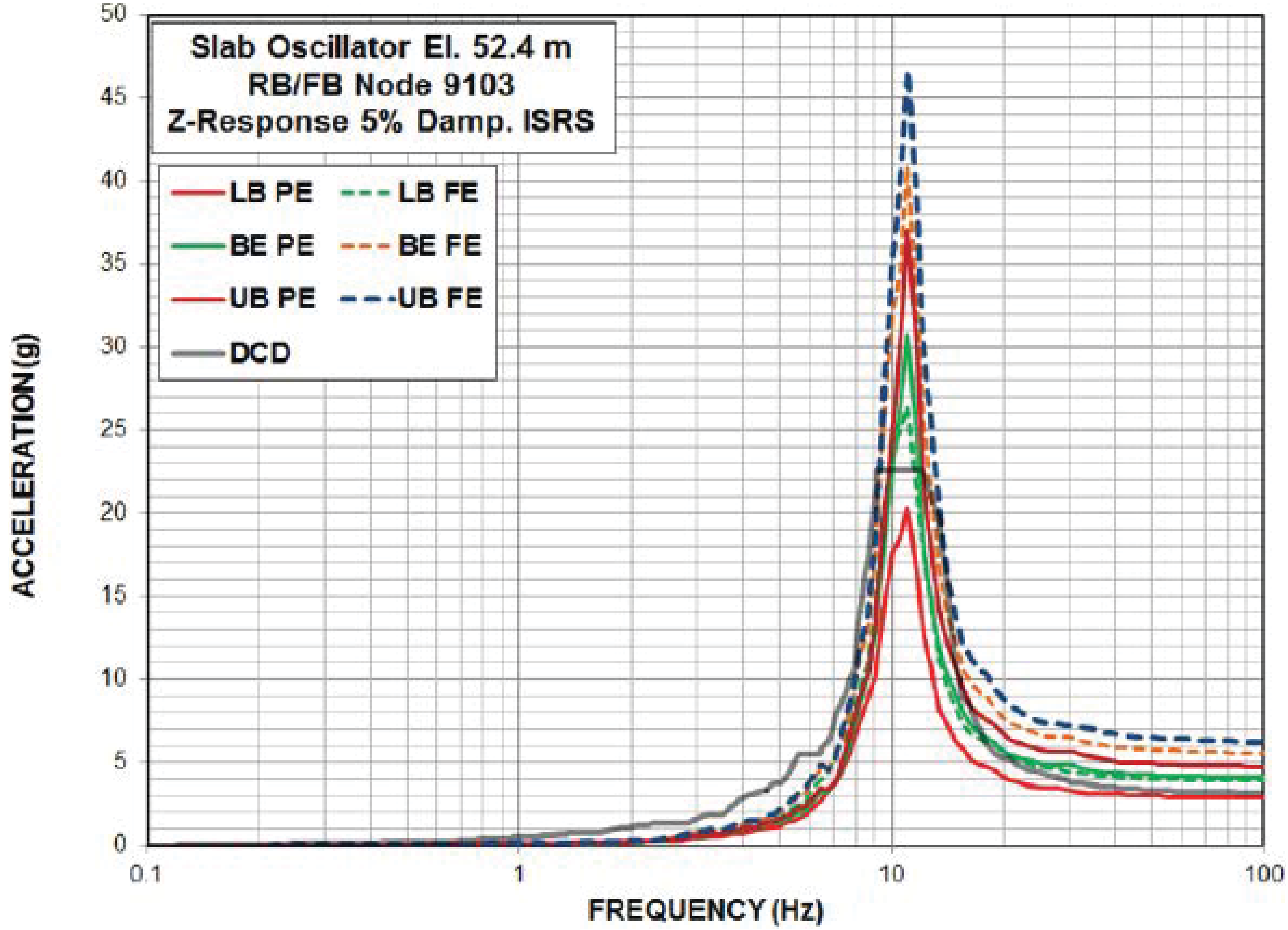


NAPS DEP 3.7-1 Figure 3A.17.12.3-204b **Comparison of ISRS - Oscillator Node RB/FB 99975Y**



NAPS DEP 3.7-1

Figure 3A.17.12.3-204c **Comparison of ISRS - Oscillator Node RB/FB 9103Z**



3A.17.12.4 Maximum Lateral Pressures on Below-Grade Exterior Walls

As discussed in Section 3A.16.3, spring elements between double nodes are installed on the foundation and soil/rock interfaces to calculate the SSI contact reactions. The lateral seismic pressures on the RB/FB below-grade walls are calculated from the SASSI analyses results for the force time histories from the springs installed between the shells on the sides of the embedded portion of RB/FB model and the surrounding near-field elements.

The absolute value of the co-directional spring force magnitudes obtained from the analyses of the three orthogonal input earthquake motion components are combined absolutely in the time domain. The absolute values of the maximum lateral forces for all spring elements at the same elevations are summed up to obtain the total maximum lateral forces at the respective elevations for the four RB/FB below-grade exterior walls separately.

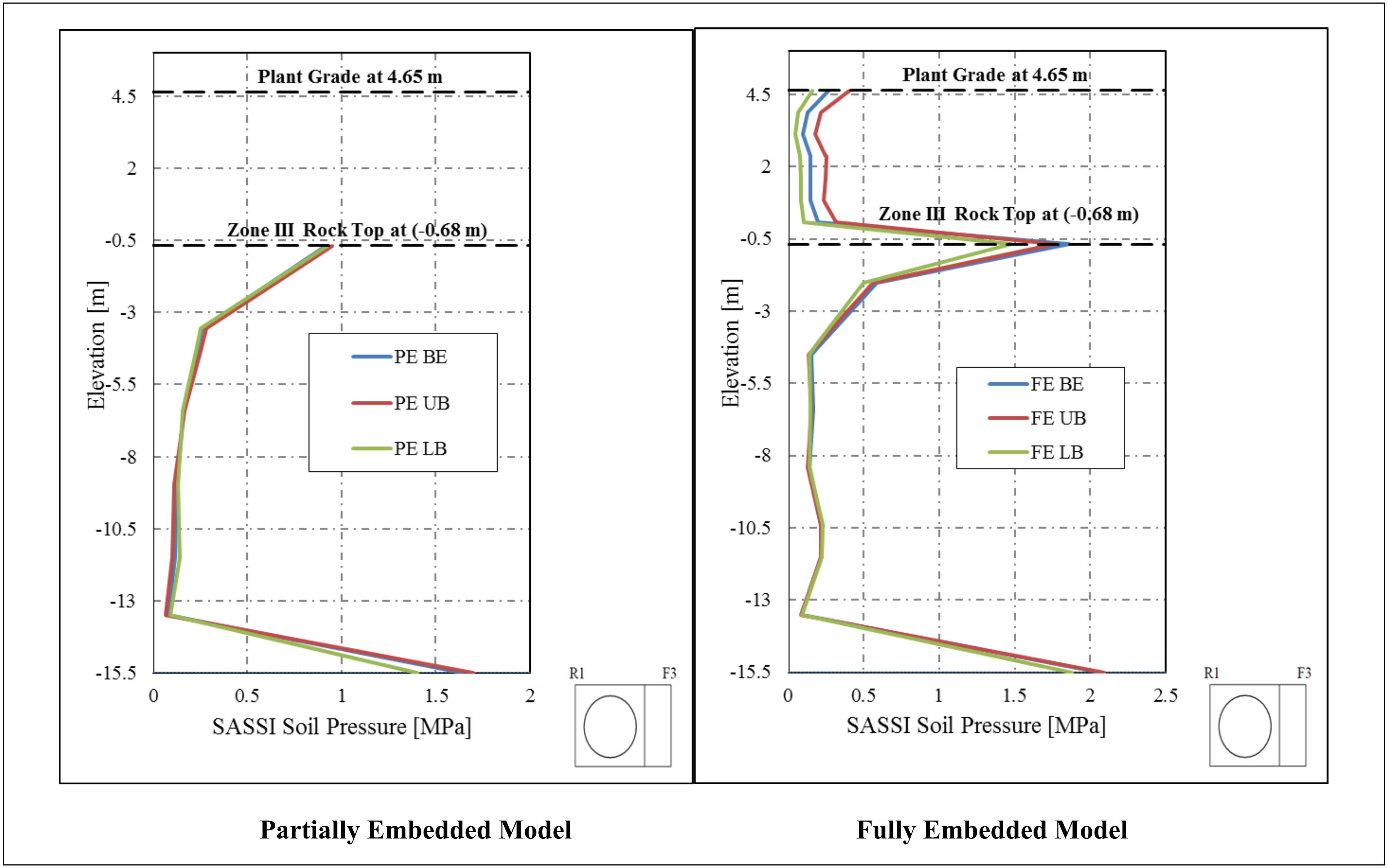
Figures 3A.17.12.4-201a through 3A.17.12.4-201d provide plots of the results of the site-specific SSI analyses of the RB/FB models with upper bound stiffness properties and OBE damping values (Cases 1 through 6 in Table 3A.15-201) for the maximum seismic lateral pressures on the RB/FB below-grade exterior walls. The comparison of the results obtained from different analyses shows that the variation of the soil properties has a very small effect on the calculated dynamic pressures on the exterior walls. Very small differences can be observed between the lateral pressures calculated from the analyses of BE, LB, and UB profiles. The results obtained from the analyses of full column profiles show that the pressures on the RB/FB exterior walls from the structural fill or saprolite are small relative to those from the concrete fill/Zone III rock. The analyses of both embedment configurations yield maximum lateral pressures at the top of Zone III rock (Elevation -0.68 m) and at the bottom of RB/FB basemat (Elevation -15.5 m).

For the site specific RB/FB stability evaluation described in Sections 3.8.4 and 3G.7, the dynamic lateral pressure results obtained from the site-specific SSI analyses are combined with site-specific static pressures to develop the total site-specific lateral load demands on RB/FB below-grade exterior walls. These total site-specific lateral loads are compared with the corresponding standard design loads in order to

determine exceedances of the Unit 3 site-specific lateral loads demands relative to the standard design.

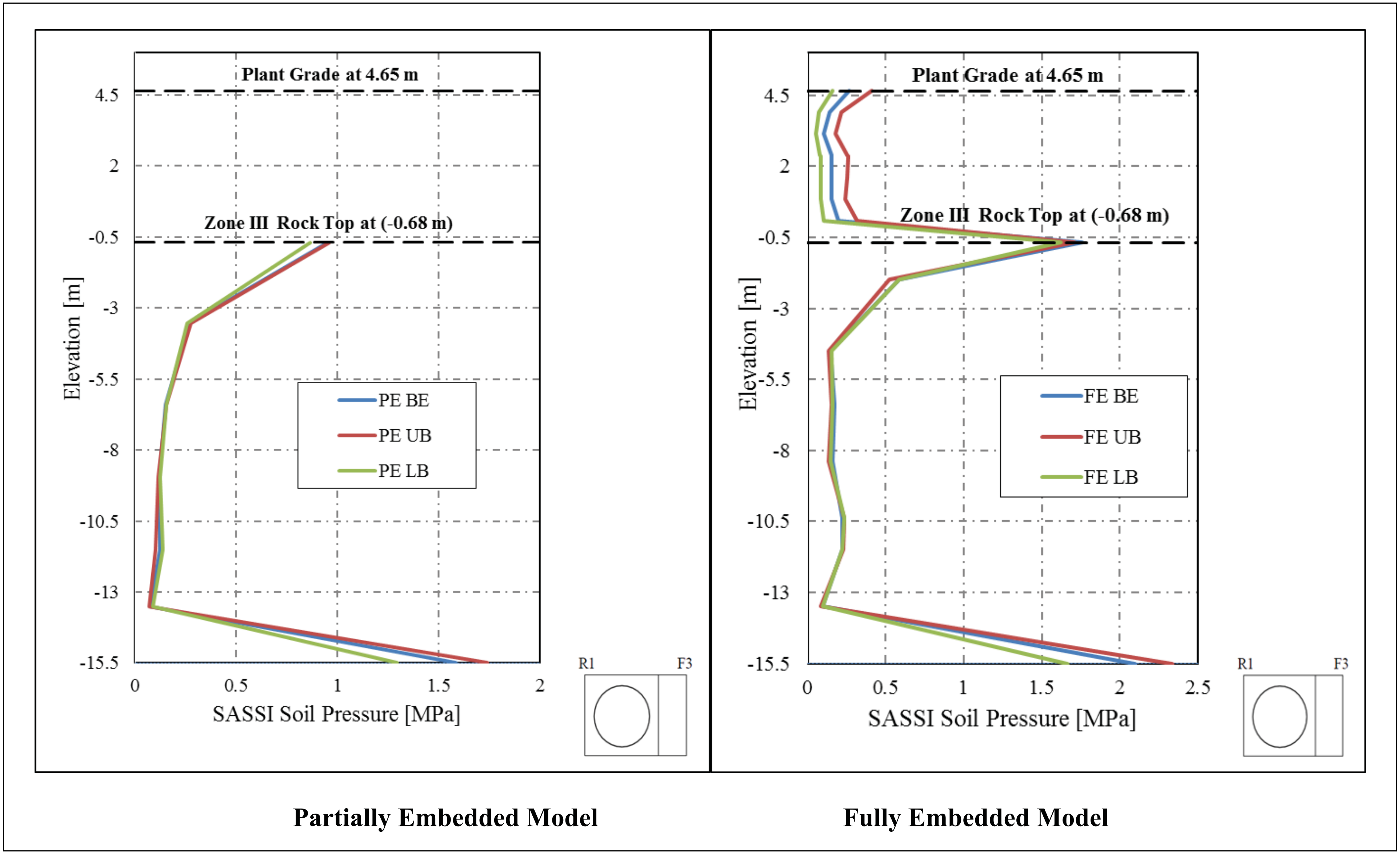
NAPS DEP 3.7-1

Figure 3A.17.12.4-201a Dynamic Lateral Pressures on Below-Grade Exterior Walls: RB/FB Wall at Column Line R1



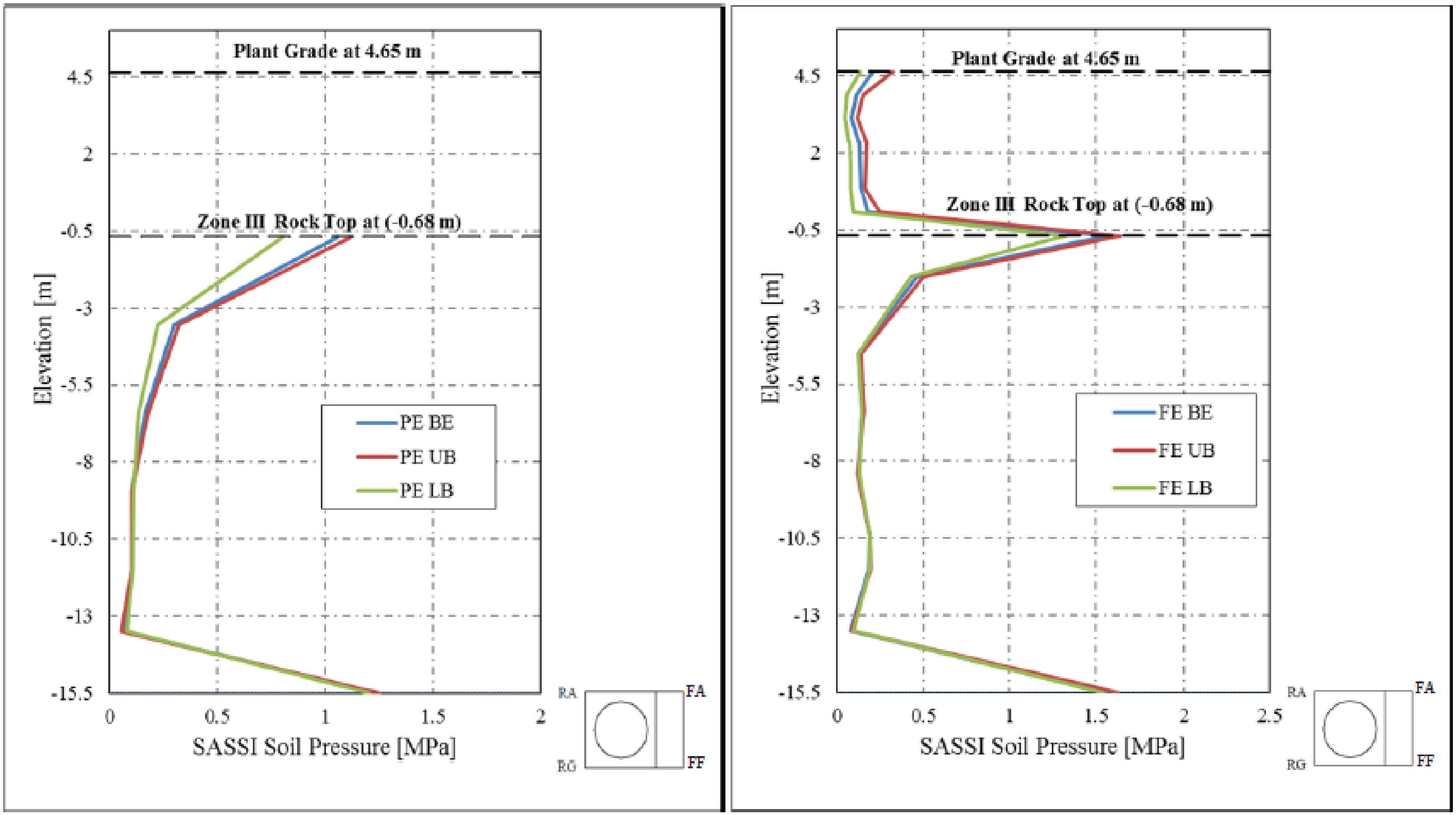
NAPS DEP 3.7-1

Figure 3A.17.12.4-201b Dynamic Lateral Pressures on Below-Grade Exterior Walls: RB/FB Wall at Column Line F3



NAPS DEP 3.7-1

Figure 3A.17.12.4-201c Dynamic Lateral Pressures on Below-Grade Exterior Walls: RB/FB Wall at Column Lines RA and FA

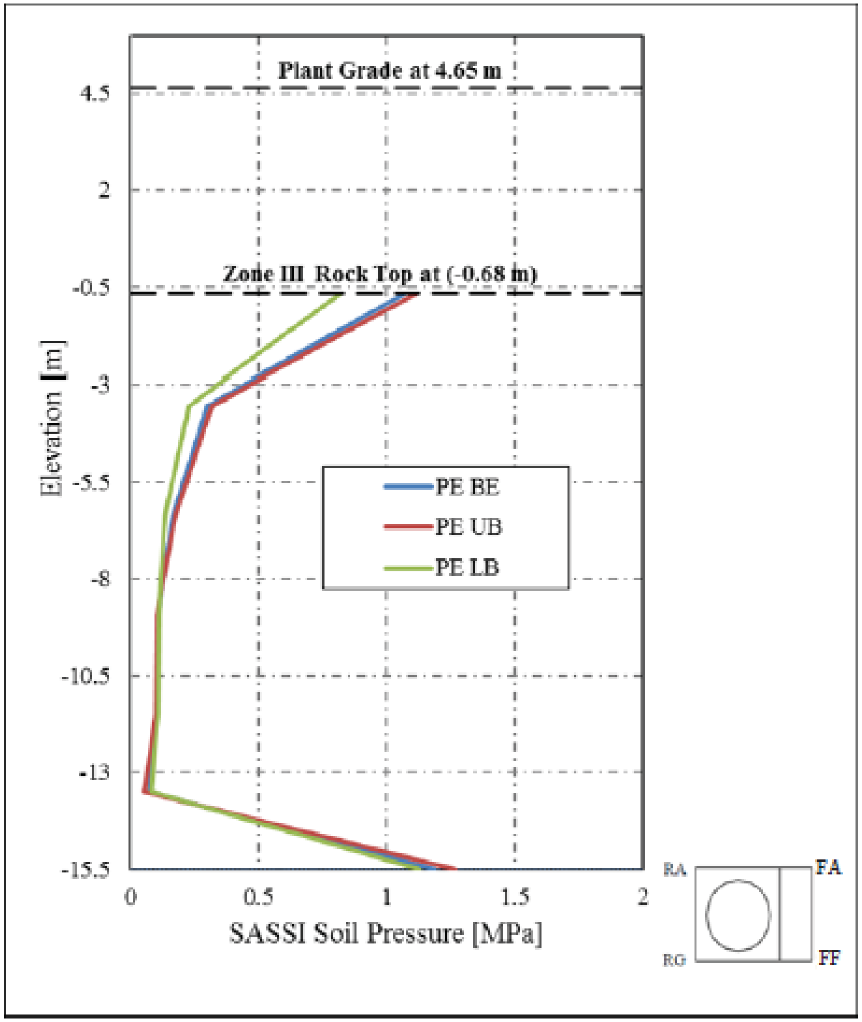


Partially Embedded Model

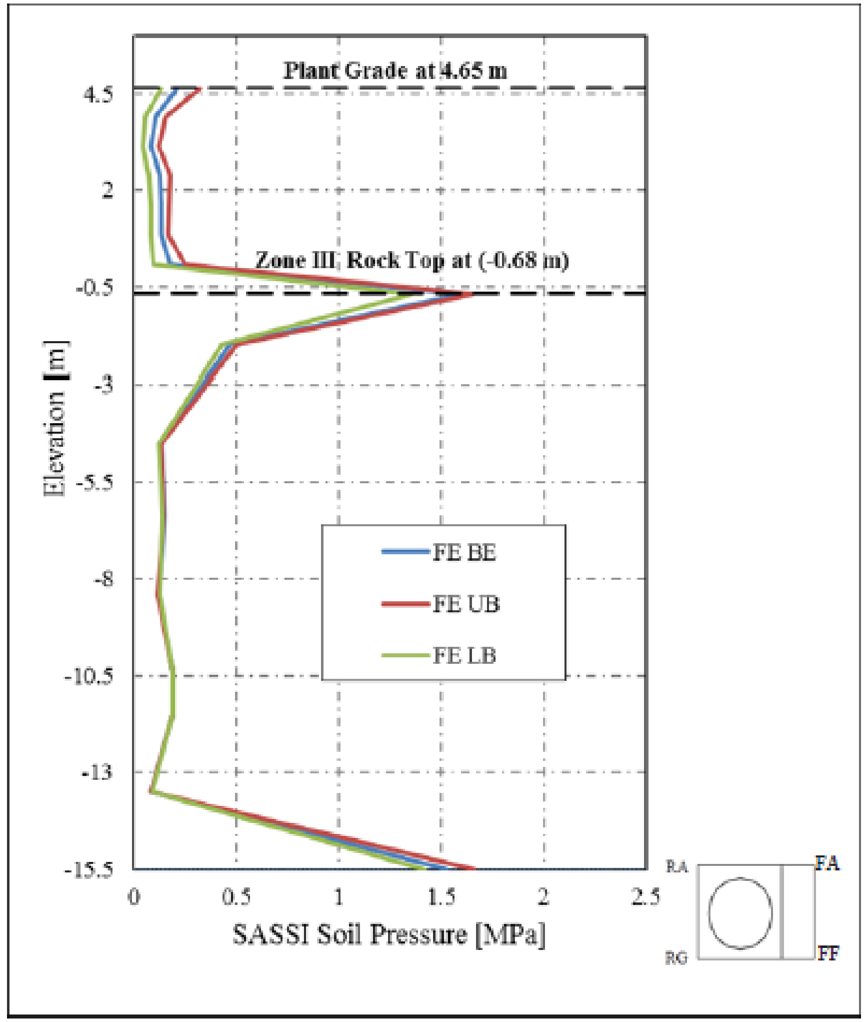
Fully Embedded Model

NAPS DEP 3.7-1

Figure 3A.17.12.4-201d Dynamic Lateral Pressures on Below-Grade Exterior Walls: RB/FB Wall at Column Line RG and FF



Partially Embedded Model



Fully Embedded Model

3A.17.12.5 Base Reactions and Contact Pressures

The results of the SSI analyses for contact spring forces are also used to calculate time histories of the seismic driving forces used as input for the sliding stability evaluations and dynamic bearing pressure calculations. The time histories of the horizontal and vertical driving seismic forces in the three orthogonal directions are calculated as the algebraic sum of the spring forces in the three directions at each time step from all contact spring elements at the interfaces between the RB/FB and the surrounding soil.

The contact spring elements at the bottom of the RB/FB basemat provide the input needed for the calculation of the dynamic bearing pressures.

The calculations of the dynamic bearing pressures are based on the time histories of:

- The overturning base moments calculated for both the horizontal directions by summing algebraically the moments generated by each contact spring reaction at the bottom of the RB/FB basemat
- The vertical driving seismic forces calculated as the algebraic sum of the vertical spring forces at each time step from all contact spring elements at the interface between the bottom of the RB/FB basemat and the rock

The results of the SSI analyses for the time histories of the vertical spring forces obtained from the contact springs at the bottom of the RB/FB basemat are used to develop base contact pressure plots for determining the minimum base contact area. These calculations are performed in the following steps:

1. Time histories of the vertical base reactions are calculated as the sum of the spring forces obtained from the contact spring elements at the bottom of the RB/FB basemat.
2. Time histories of upward base reactions RV(t) are calculated by subtracting the vertical seismic force calculated in Step 1 and the groundwater buoyancy force from the building seismic weight (total weight assigned to the RB/FB dynamic model).

3. Time histories of the overturning base moments *MNS(t)* and *MEW(t)* are calculated for the rotation about NS and EW axes by summing the moments generated by each contact spring reaction at the bottom of basemat.

4. For each soil case analysis, the time history of base reaction eccentricity *Ecc(t)* is calculated as follows:

$$Ecc(t) = \frac{1}{R_V(t)}\sqrt{{M_{NS}}^2 + {M_{EW}}^2}$$

5. The critical time step when the maximum uplift of the RB/FB basemat occurs is determined as the time instance when the value of the base reaction eccentricity *Ecc(t)* is the largest.

6. The plot of the base contact area is developed using the base spring force results at the critical time step obtained from the analysis of soil case that yields maximum value for base eccentricity.

7. The base contact pressures are calculated by dividing the base spring forces by their tributary area and then adding the uniform static pressure due to the effective weight of the building (the RB/FB seismic weight minus the groundwater buoyancy).

8. The uplifted area of the foundation is determined as the area where the value of the contact pressure is negative.

Figure 3A.17.12.5-201 presents the time histories of the eccentricities of vertical base reactions obtained from the set of six SSI analyses performed on the RB/FB model with UB stiffness properties and OBE damping (analysis Cases 1 to 6 in Table 3A.15-201). The critical instances of time when the eccentricity of the vertical base reaction has maximum value are identified with red lines. Table 3A.17.12.5-201 summarizes the calculations of maximum base reaction eccentricities and shows that the analyses of the UB partial column subgrade profile yields the maximum value for the vertical base reaction eccentricity.

Figure 3A.17.12.5-202 presents a contour plot of the base contact pressure results obtained from the analysis of the RB/FB for the BE full column profile for the instances of time t = 3.150 sec when the uplift of RB/FB basemat is the largest. The positive values of the contact pressures in the figures represent tension of the base contact springs. The negative values represent compression of the base contact springs

and are used to determine the portion of the basemat that is in contact with the underlying subgrade.

Figure 3A.17.12.5-202 shows that the contact area between the RB/FB basemat and the underlying subgrade remains larger than 84 percent. The base contact area remains larger than 80 percent which, in accordance with guidance in SRP 3.7.2, ensures that the possible uplift of the RB/FB basemat has negligible effect on the RB/FB seismic response and results of site-specific RB/FB SSI analyses that are performed on linear elastic models.

NAPS DEP 3.7-1

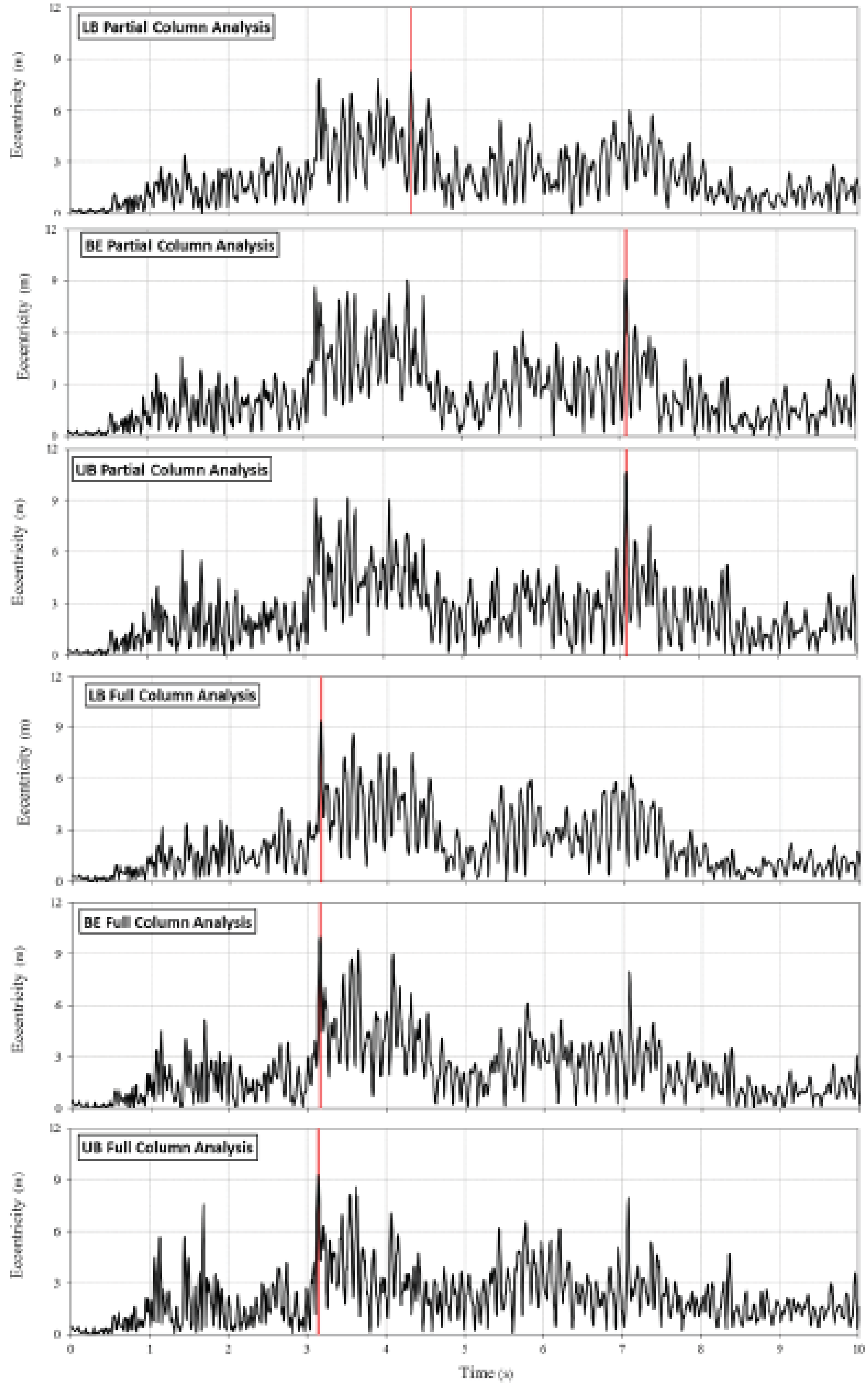
Table 3A.17.12.5-201 **Summary of Maximum Base Reaction Eccentricity Results – RB/FB**

	Full Column			**Partial Column**		
Analysis	**BE**	**UB**	**LB**	**BE**	**UB**	**LB**
Max. Eccentricity (m)	10.0	9.2	9.4	9.1	10.7	8.2
at Time (s)	3.150	3.145	3.155	7.080	7.075	4.315

Note: The shaded values in the table show the governing case.

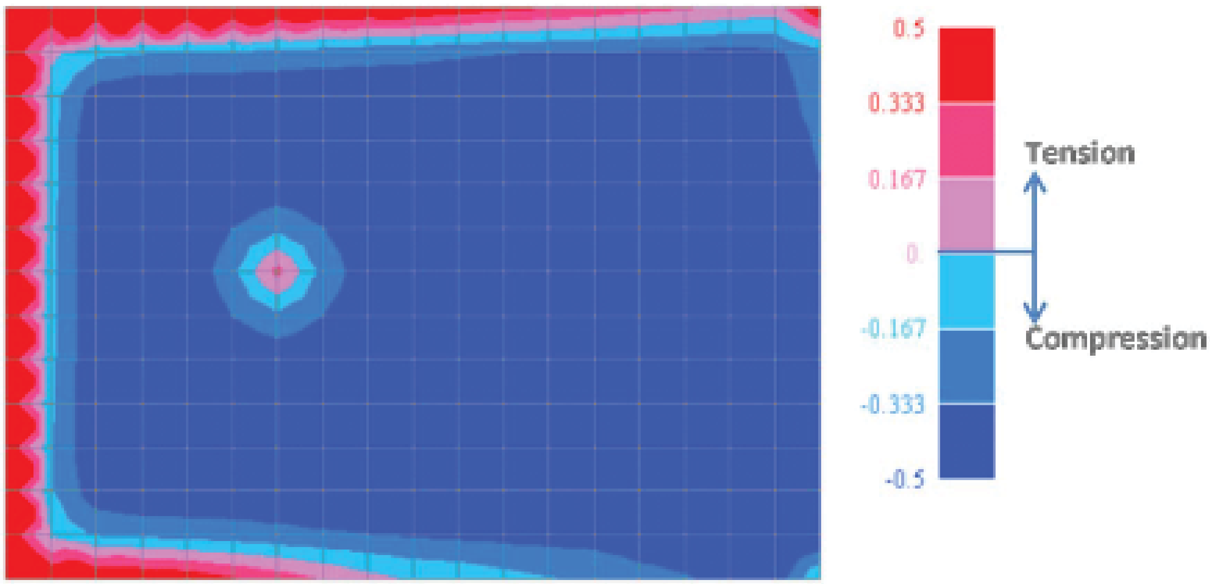
NAPS DEP 3.7-1

Figure 3A.17.12.5-201 **RB/FB Base Reaction Eccentricity**

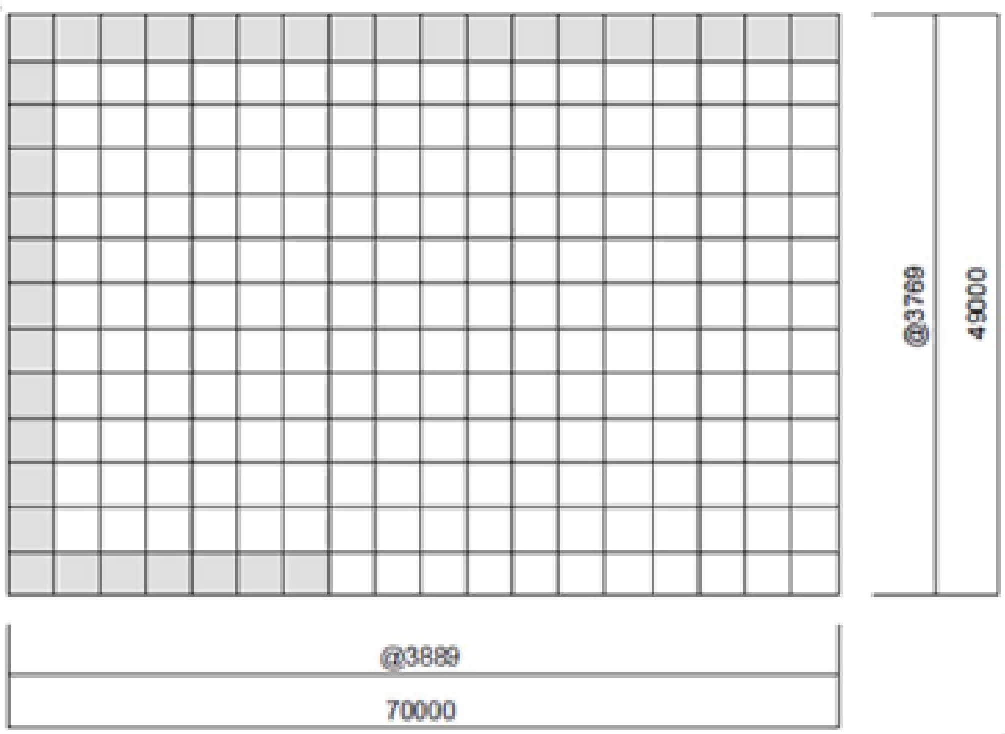


NAPS DEP 3.7-1

Figure 3A.17.12.5-202 **RB/FB Base Contact Area (BE Full Column Analysis)**



(a) Base Contact Pressures



(b) Minimum Contact Area (84% at critical time t = 3.150 sec)

3A.17.13 Unit 3 SSI Analysis Results for CB

This section presents results of the site-specific SSI analyses of the CB models with full stiffness properties (analysis Cases 1 through 12 in Table 3A.15-202).

3A.17.13.1 CB SSI Response

Results of the CB SSI analysis show that the saprolite and structural fill above the top of Unit 3 rock can affect the response of the light and deeply embedded CB resulting in shifts of the CB peak responses to higher frequencies and also reduction of responses due to higher dissipation of energy. The effects of rock subgrade properties variations on the CB response are smaller. The SSI analysis of UB partial column profile provides bounding site-specific seismic demands on the CB structure. The CB analysis of the partial column profiles yield bounding results with few exceptions in the ISRS results at lower (< 20 Hz) frequencies that are bounded by responses obtained from analyses of full column profiles.

3A.17.13.2 Maximum Accelerations and Member Forces

Figures 3A.17.13.2-201 and 3A.17.13.2-202 compare results for maximum absolute accelerations at CB floor mass locations from the analyses of CB models with full stiffness and SSE damping for the partial column profiles and full column profiles, respectively. Table 3A.17.13.2-201 compares the maximum vertical accelerations results for CB slab SDOF oscillators obtained from the analysis Cases 7 through 12 in Table 3A.15-202. Figures 3A.17.13.2-203 and 3A.17.13.2-204 compare the maximum shear forces and torsion results obtained from the analyses of CB models with full stiffness properties and SSE damping for the partial column profiles and full column profiles, respectively. Comparisons of the maximum response results show that the analysis of the UB partial column profile (analysis Case 9) govern the CB maximum responses with the exception of the maximum shear forces between Elevations -2.5 m and 4.7 m obtained from the analyses of the CB fully embedded model where the softer soil embedment amplified the CB shears.

NAPS DEP 3.7-1

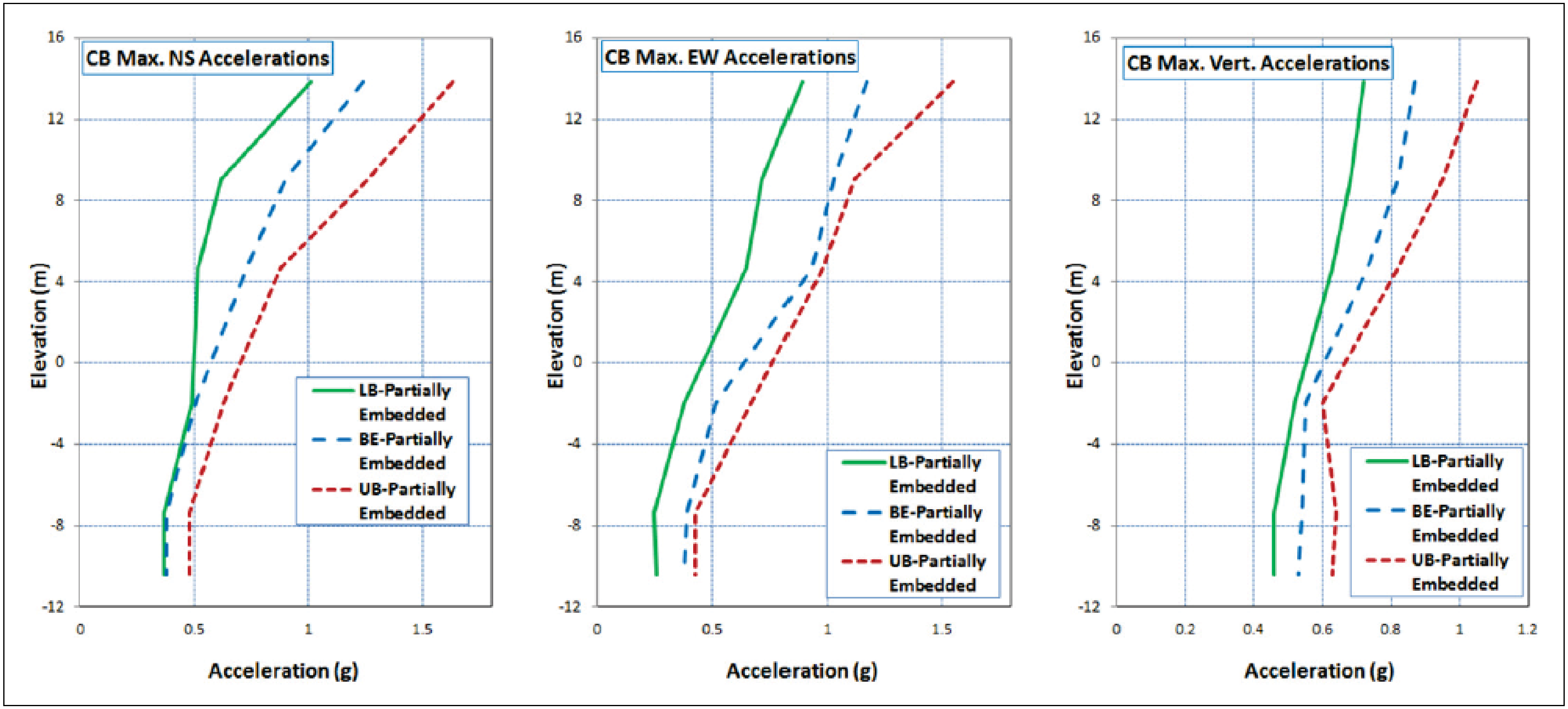
Table 3A.17.13.2-201 **Maximum Accelerations of Slab SDOF Oscillators - CB**

Elev. (m)	Node No.	Vertical Acceleration (g): Full Column LB	Full Column BE	Full Column UB	Partial Column LB	Partial Column BE	Partial Column UB	NA3 Envelope	Standard Design
13.80	9001	0.92	1.13	1.45	1.80	2.08	*2.20*	2.20	2.19
	9002	1.15	1.16	1.28	1.18	1.44	*1.69*	1.69	1.34
	9003	0.73	1.41	1.98	1.30	1.80	*1.99*	1.99	1.43
9.06	9101	1.00	1.31	1.49	1.86	2.00	*2.08*	2.08	2.00
	9102	0.77	1.11	1.30	1.16	1.33	*1.53*	1.53	1.26
	9103	0.67	1.41	*2.00*	1.28	1.77	1.91	2.00	1.43
4.65	9201	0.76	0.99	1.03	0.96	1.10	*1.24*	1.24	1.30
	9202	0.62	1.17	*1.53*	1.09	1.33	1.46	1.53	1.43
-2.00	9301	0.53	1.08	0.97	1.12	*1.16*	1.12	1.16	1.39

Note: The shaded values in the table show exceedance from standard design.
The values shown in Italic are governing case

NAPS DEP 3.7-1

Figure 3A.17.13.2-201 **Maximum Accelerations Results from Analyses of CB PE Model**



NAPS DEP 3.7-1

Figure 3A.17.13.2-202 **Maximum Accelerations Results from Analyses of CB FE Model**

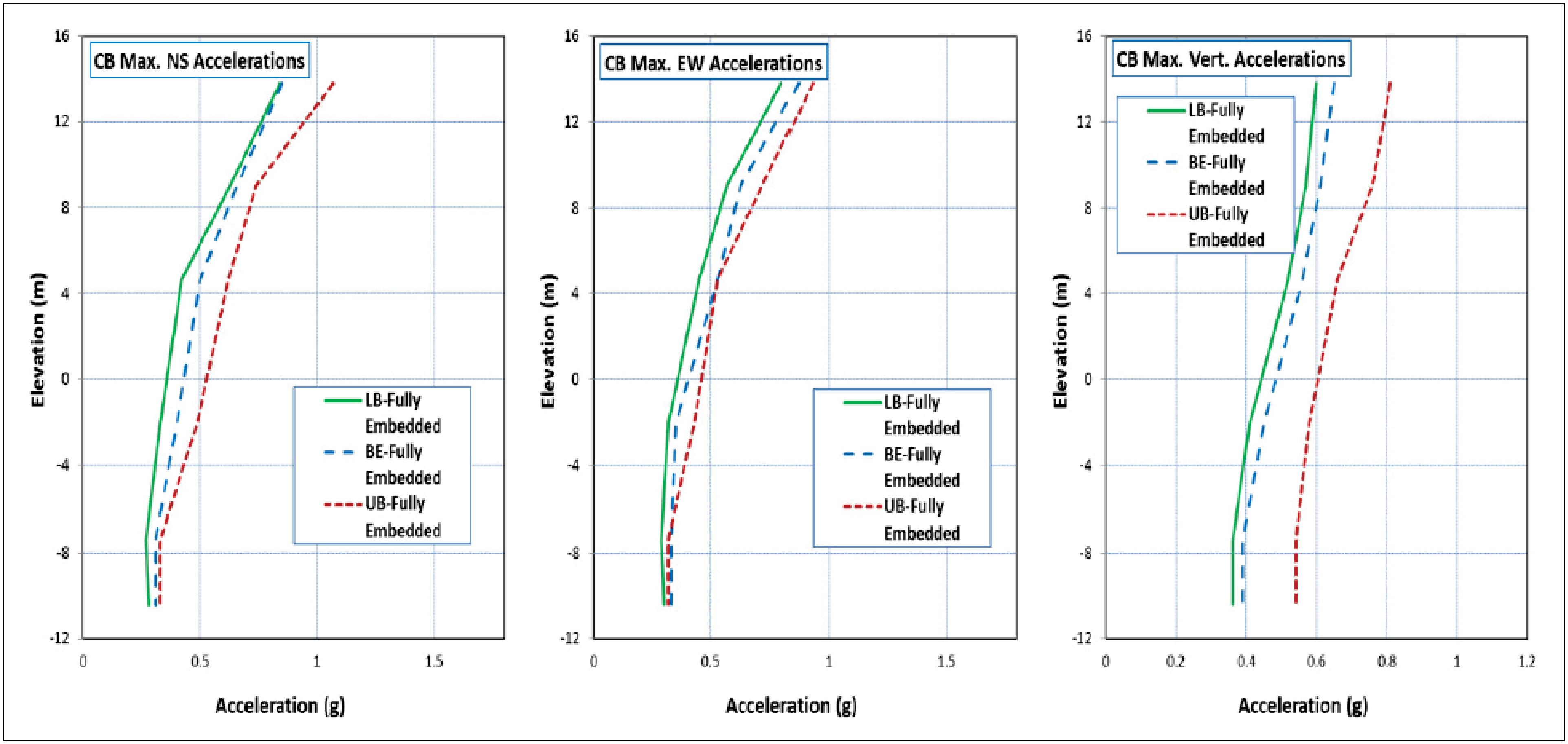


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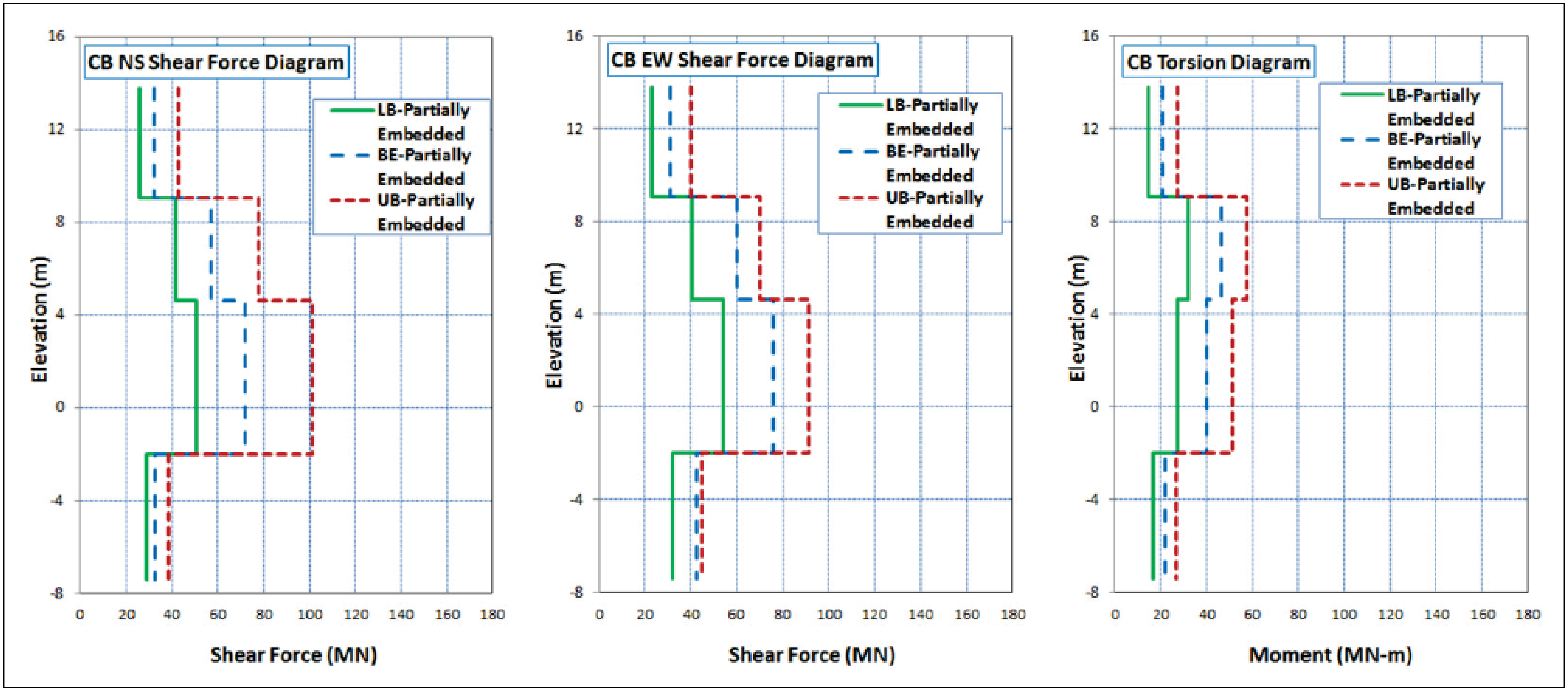
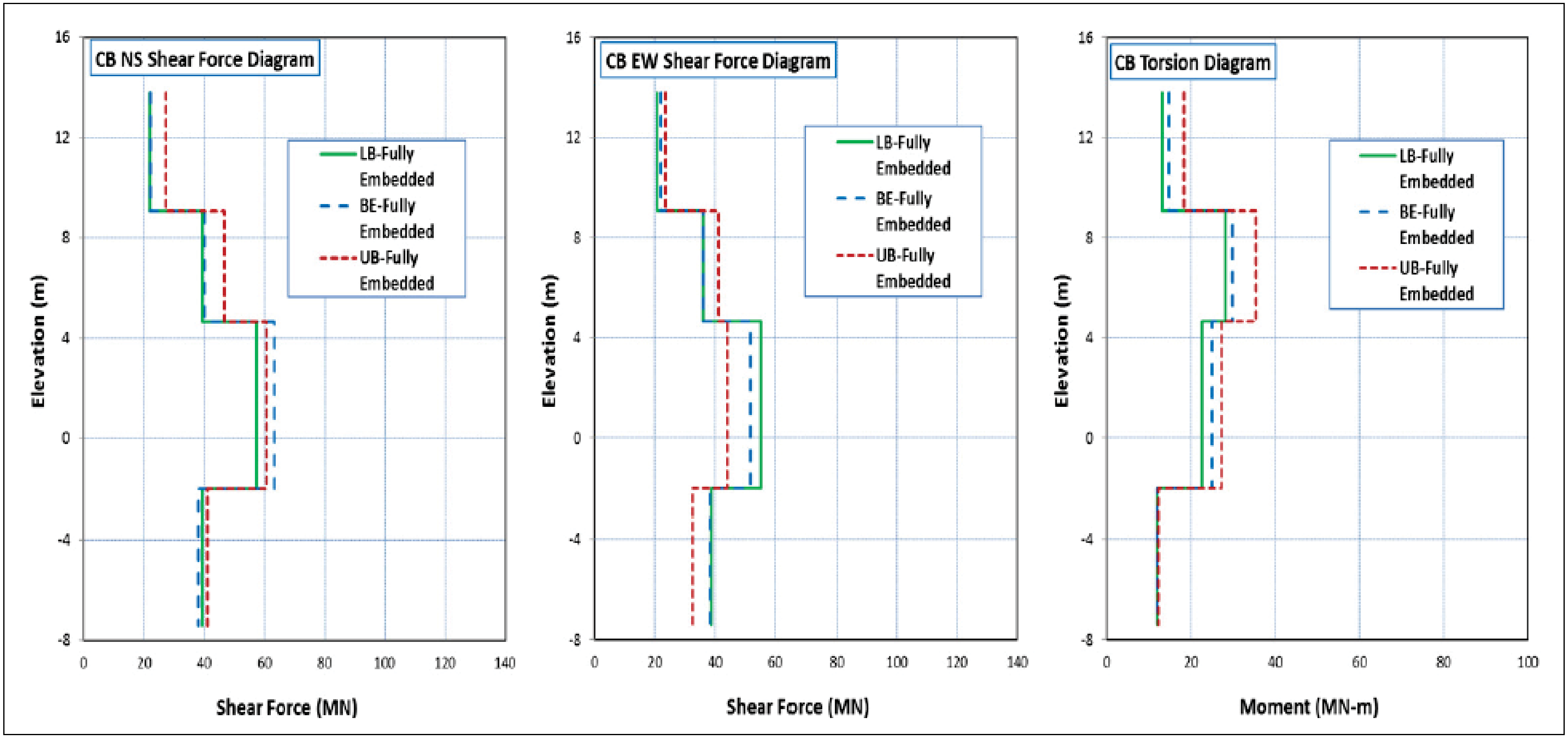
Figure 3A.17.13.2-203 **Maximum Shear Forces and Torsion Results from Analyses of CB PE Model**



NAPS DEP 3.7-1

Figure 3A.17.13.2-204 Maximum Shear Forces and Torsion Results from Analyses of CB FE Model



3A.17.13.3 Acceleration Response Spectra

Comparisons of the 5 percent damped ARS results are presented for selected locations within the CB. The ISRS are obtained from the analyses of the CB model with full (uncracked concrete) stiffness properties and OBE damping for the six subgrade profiles (analysis Cases 1 through 6 in Table 3A.15-202).

Figures 3A.17.13.3-201a and 3A.17.13.3-201b, Figures 3A.17.13.3-202a and 3A.17.13.3-202b, and Figures 3A.17.13.3-203a and 3A.17.13.3-203b, respectively, present the 5 percent damped ISRS for the response in NS(x), EW(y) and vertical (z) directions at the two key locations within the CB obtained from the SSI analyses of the CB model with full stiffness properties and OBE damping values (Cases 1 to 6 in Table 3A.15-202). The site-specific ISRS are compared with the corresponding 5 percent damped standard design ISRS in DCD Section 3A.9.

Comparisons of the ISRS results obtained from the different site-specific SSI analyses show that the responses obtained from the analyses of the partial column subgrade profiles govern the horizontal and vertical ISRS for frequencies higher than 7 Hz and 10 Hz, respectively. The exception is the ISRS for the CB basemat response in the NS(x) direction, where the analyses of the UB full column profile are bounding at frequencies between 15 and 20 Hz. The UB partial column profile also provides bounding ISRS results for frequencies higher than 31 Hz. The LB and BE full column profiles can only affect the enveloping ISRS for the response at the CB basemat top at frequencies below 18 Hz.

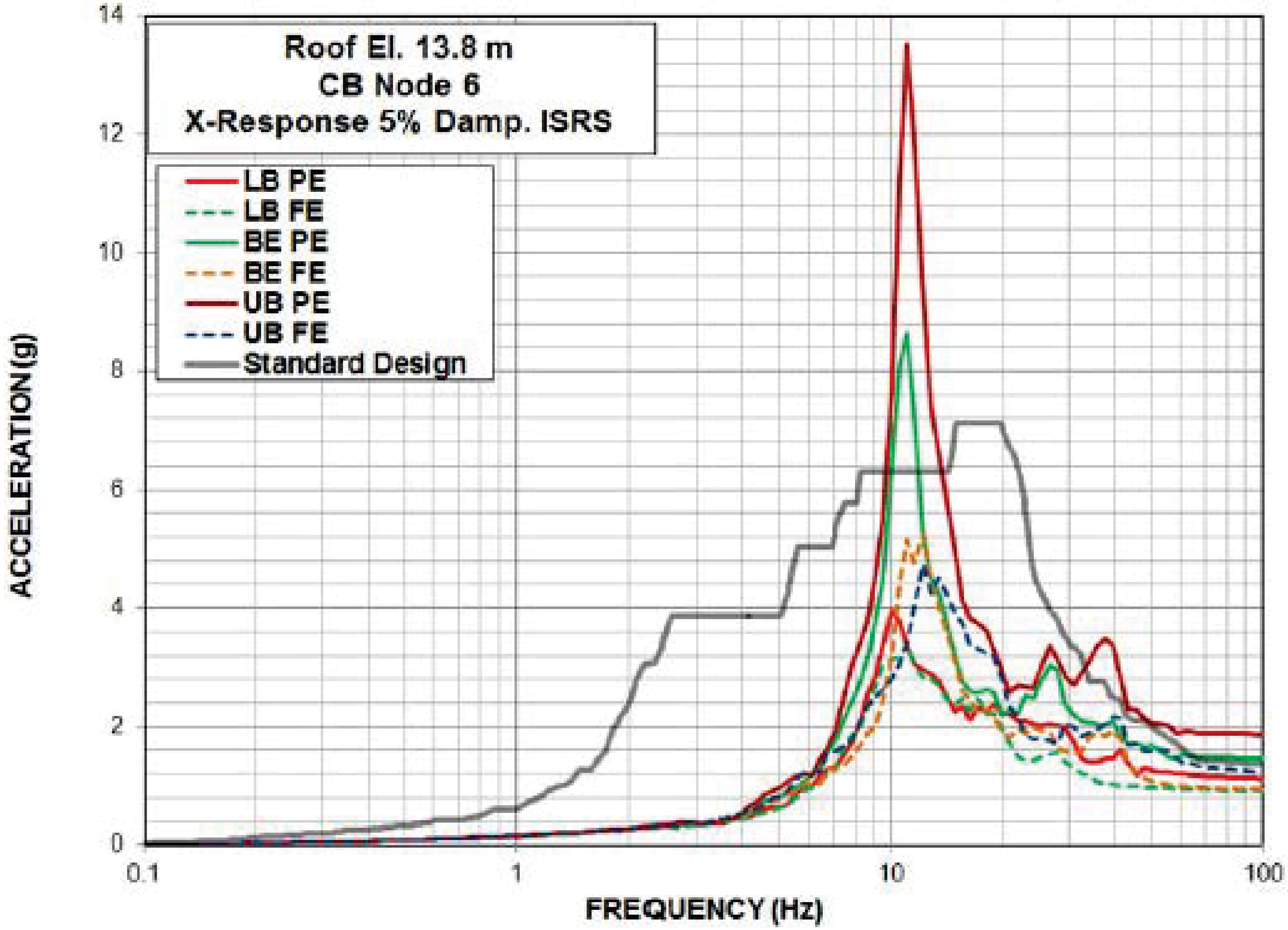
Comparisons in Figures 3A.17.13.3-201a through 3A.17.13.3-203b indicate that the CB site-specific ISRS exceed the corresponding standard design spectra. Large peak exceedances occur in the site-specific ISRS for the horizontal response of the CB roof at a relatively narrow frequency range between 10 and 15 Hz. The site-specific vertical ISRS also exceeds the corresponding standard design ISRS at frequencies from 10 to 15 Hz and 22 to 30 Hz. Certain exceedances in the SDOF oscillator ISRS occur mainly at frequencies corresponding to the frequencies of the oscillators.

The ISRS exceedances are due: 1) to the lower OBE structural damping values used for the site-specific SSI analysis versus the SSE damping values used for standard design SSI analyses and 2) the fact that the Unit 3 site-specific design motion has higher energy content than the

standard design CSDRS at frequencies close to the natural frequencies of the CB structure.

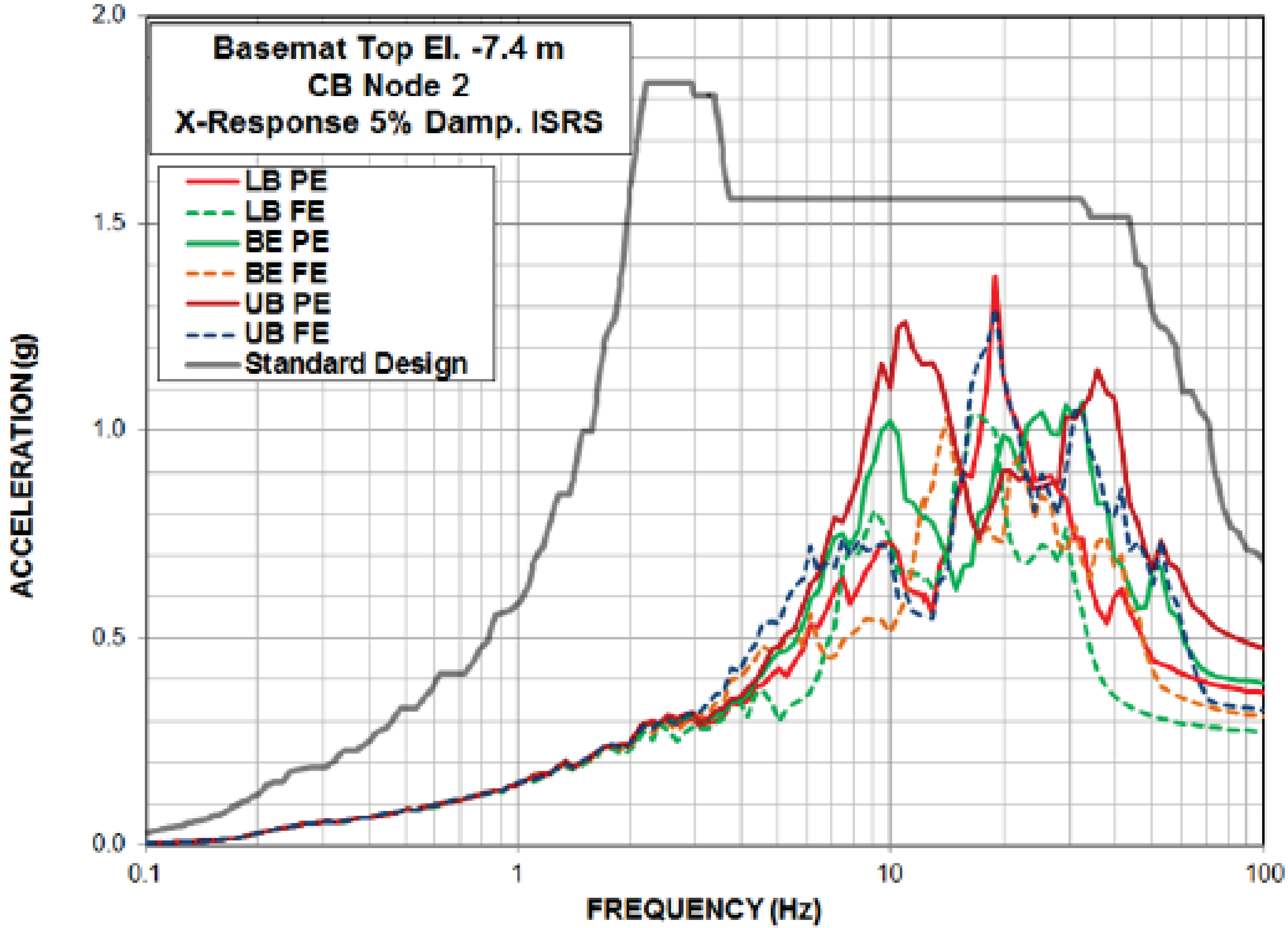
NAPS DEP 3.7-1

Figure 3A.17.13.3-201a **Comparison of ISRS - CB Top in X-Direction**



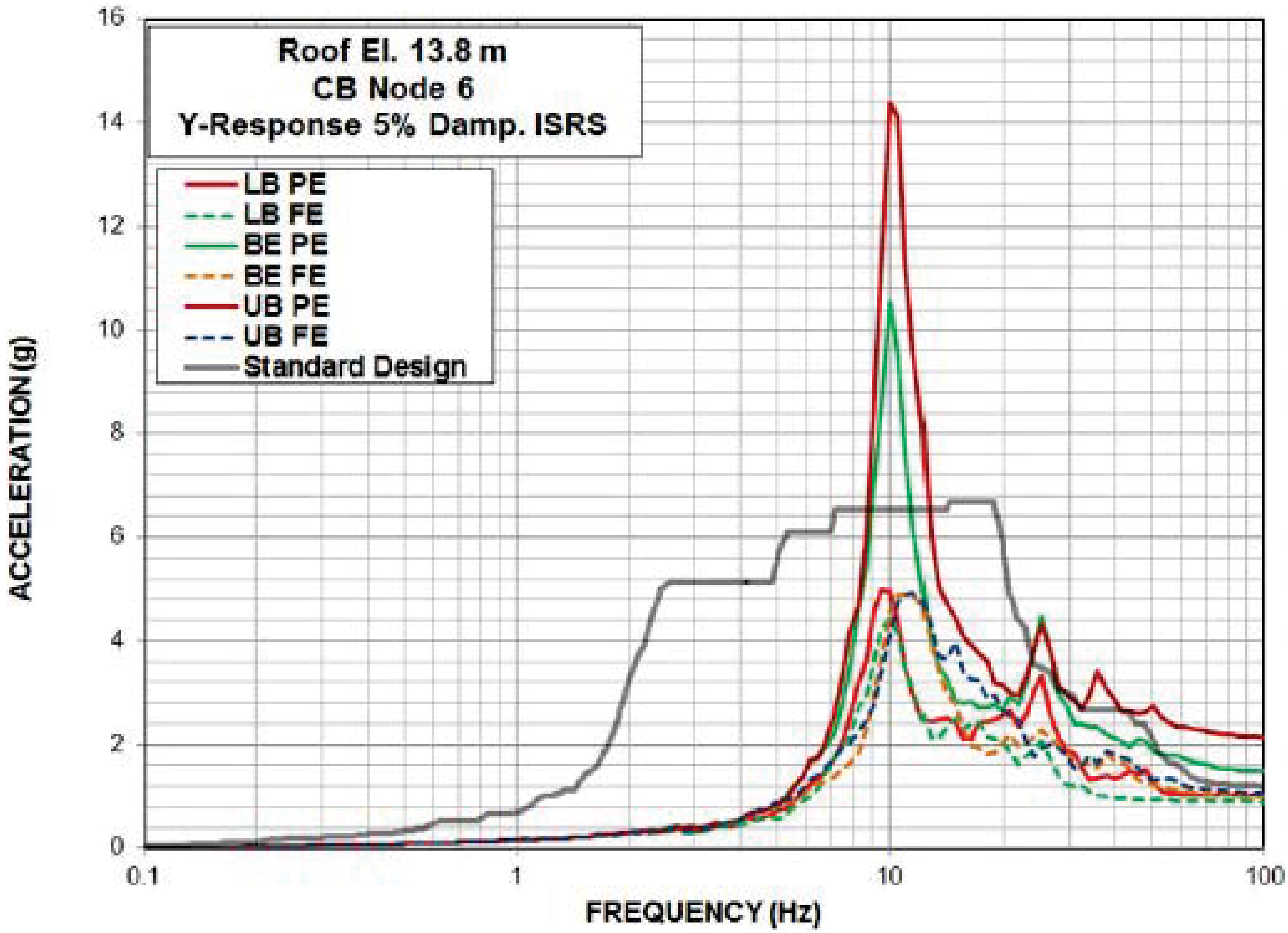
NAPS DEP 3.7-1

Figure 3A.17.13.3-201b **Comparison of ISRS - CB Basemat in X-Direction**



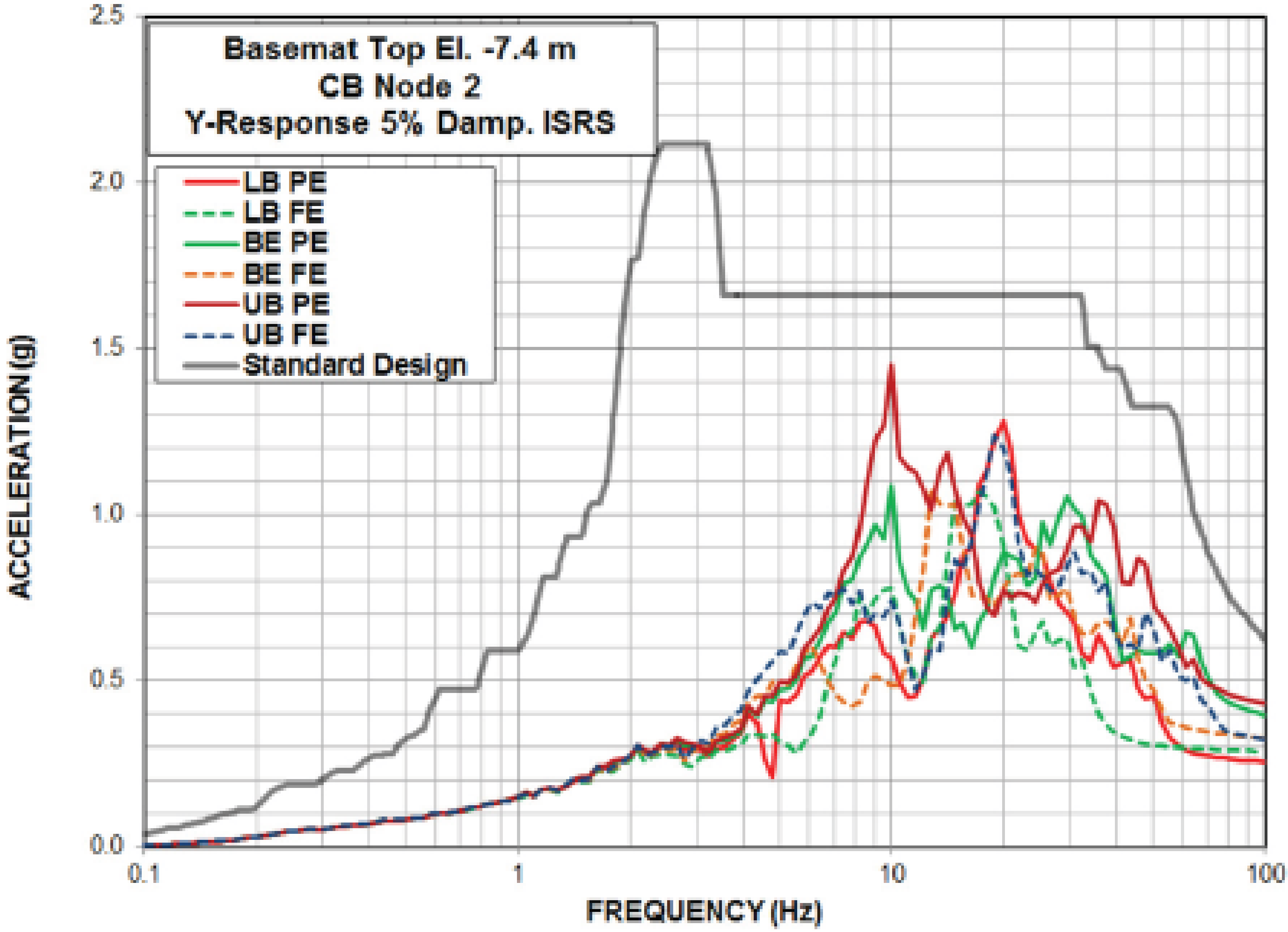
NAPS DEP 3.7-1

Figure 3A.17.13.3-202a **Comparison of ISRS - CB Top in Y-Direction**



NAPS DEP 3.7-1

Figure 3A.17.13.3-202b **Comparison of ISRS - CB Basemat in Y-Direction**



NAPS DEP 3.7-1

Figure 3A.17.13.3-203a **Comparison of ISRS - CB Top in Z-Direction**

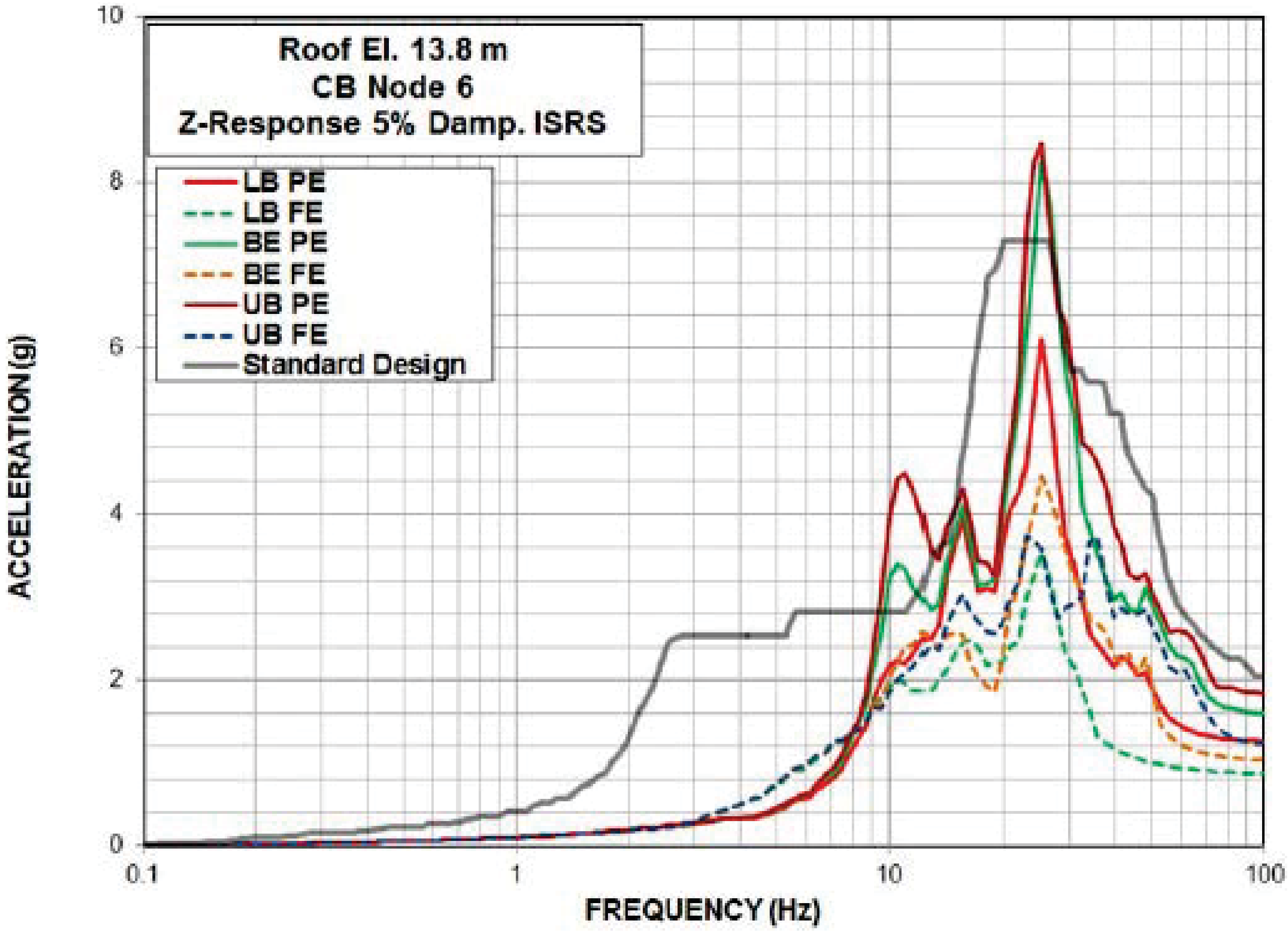


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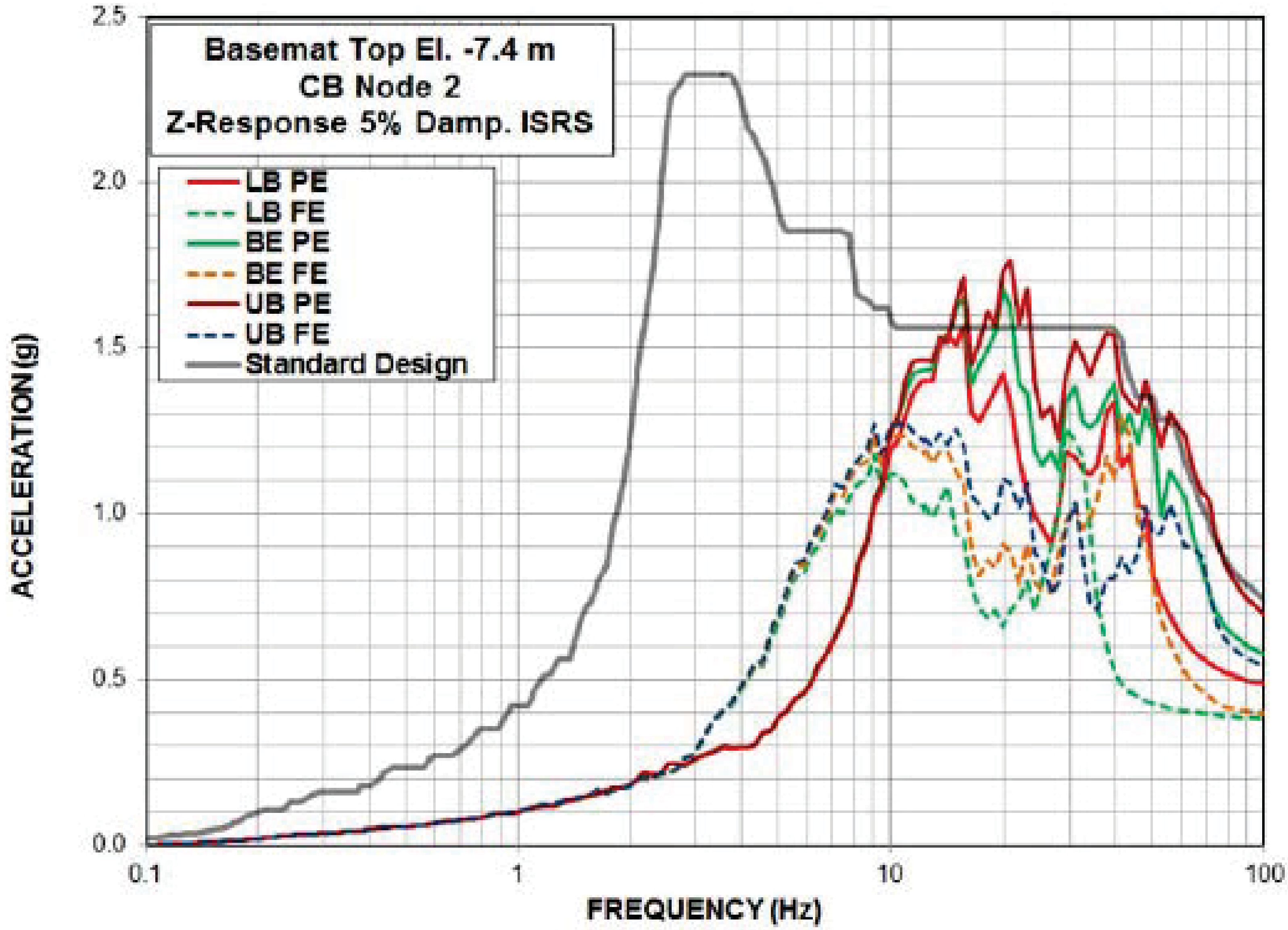
Figure 3A.17.13.3-203b **Comparison of ISRS - CB Basemat in Z-Direction**



3A.17.13.4 Maximum Lateral Pressures on Below-Grade Exterior Walls

As discussed in Section 3A.16.3, spring elements between “double nodes” are installed on the foundation and soil/rock interfaces to calculate the SSI forces. The lateral seismic pressures on the CB below-grade walls are calculated from the SASSI analysis results for the spring forces installed between the shells on the sides of the embedded portion of the CB model and the surrounding near-field elements.

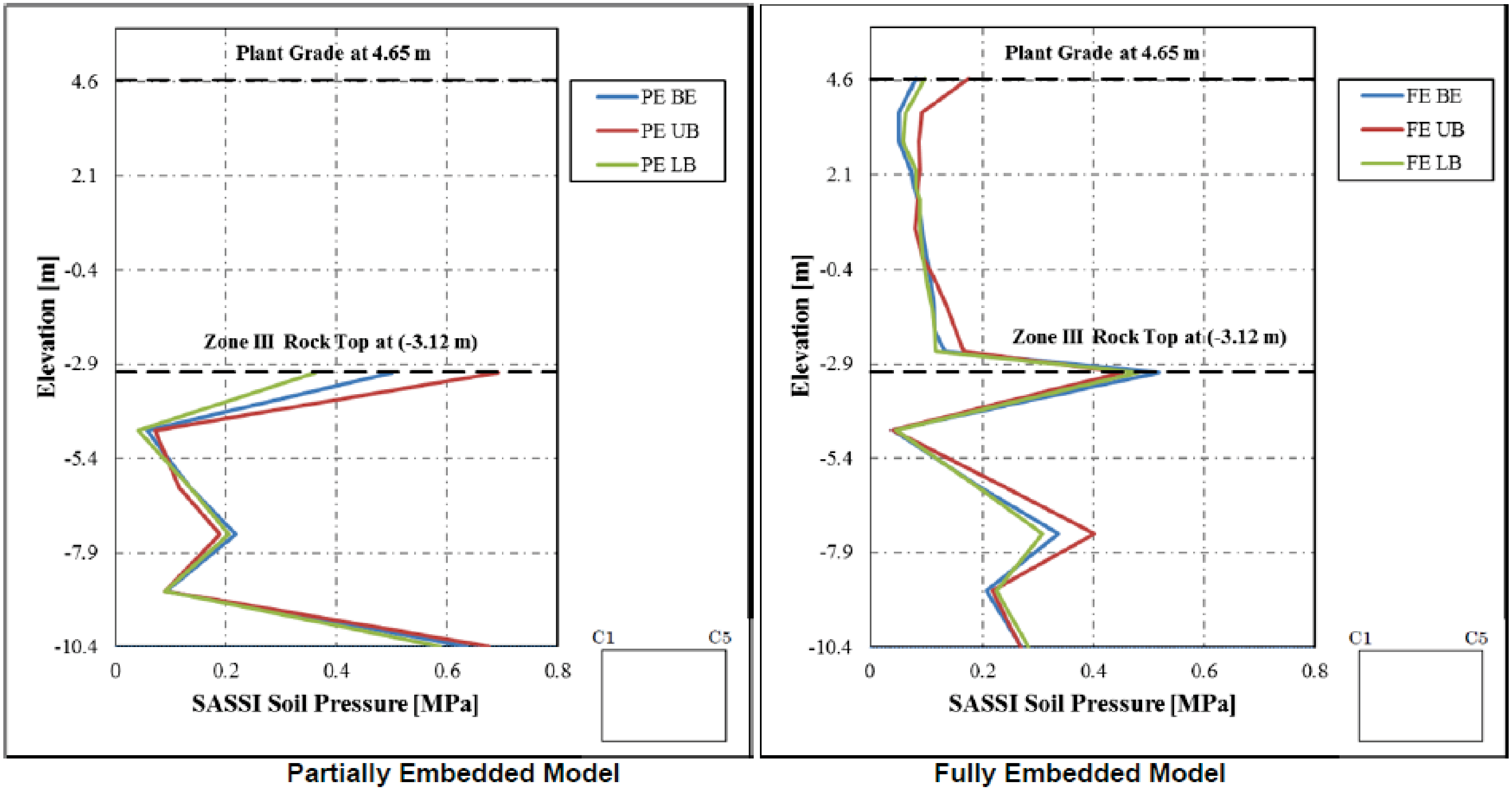
Figures 3A.17.13.4-201 through 3A.17.13.4-204 provide plots of the results from the site-specific SSI analysis of the CB models with upper bound stiffness properties and SSE damping values (Cases 7 through 12 in Table 3A.15-202) for the maximum seismic lateral pressures on the CB below-grade exterior walls.

The comparison of the results obtained from different analyses show that the variation of the soil properties has a very small effect on the calculated dynamic pressures on the exterior walls. Very small differences can be observed between the lateral pressures calculated from the analysis of LB, BE and UB profiles. Results obtained from the analyses of full column profiles show that the pressures on the CB exterior walls from the structural fill/saprolite are small relative to those from the concrete fill/Zone III rock. The analyses of both embedment configurations yield maximum lateral pressures at the top of Zone III rock (Elevation -0.68 m; Elevation 273.0 ft NAVD88) and at the bottom of CB basemat (Elevation -10.4 m; Elevation 241.1 ft NAVD88).

In the site-specific CB stability evaluation, the dynamic lateral pressure results obtained from the site-specific SSI analysis are combined with site-specific static pressures to develop the total site-specific lateral load demands on the CB below-grade exterior walls. These total site-specific lateral loads are compared with the corresponding enveloping standard design loads in order to determine exceedances of the Unit 3 site-specific lateral loads demands relative to the standard design (see Section 3G.8).

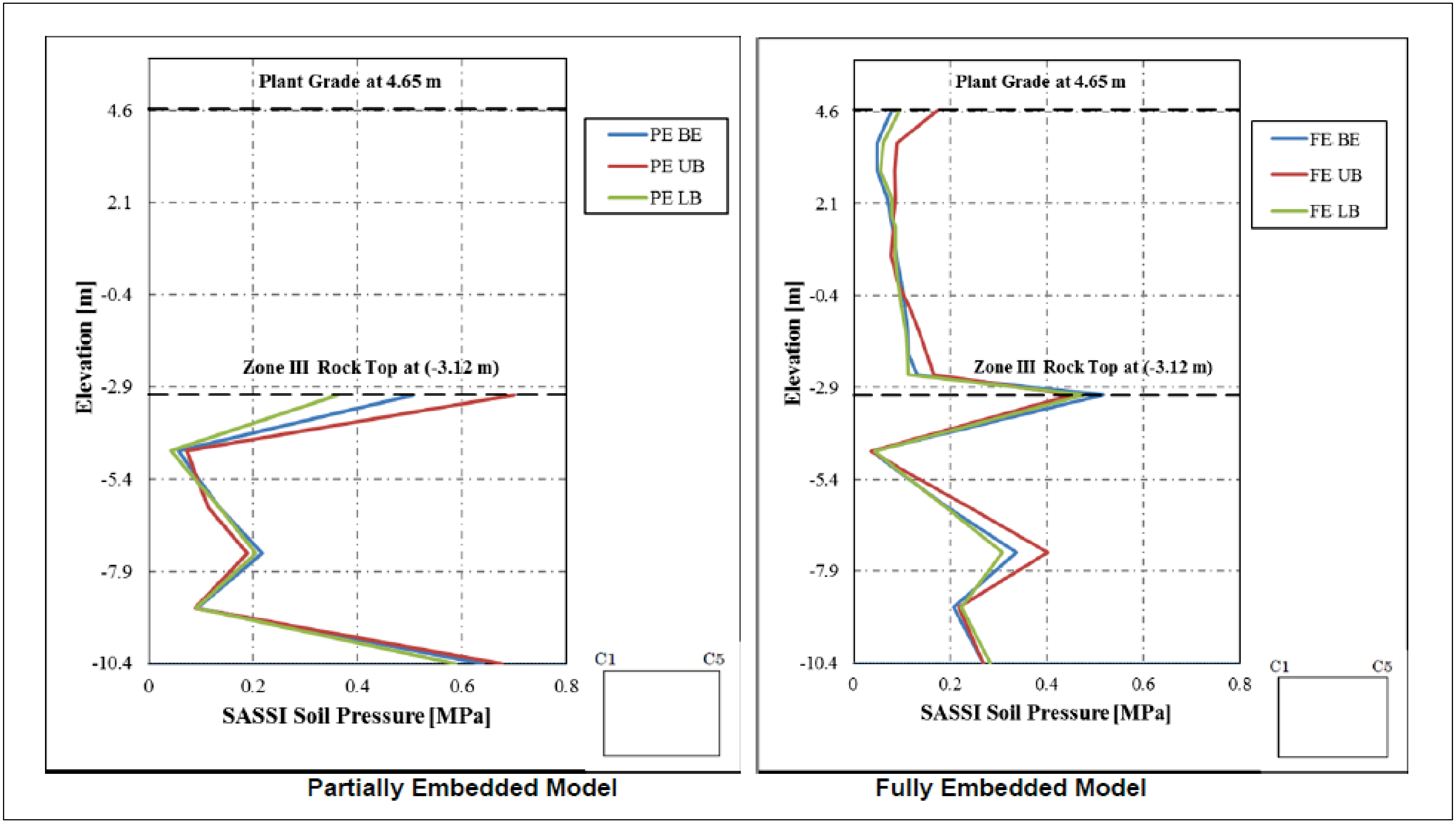
NAPS DEP 3.7-1

Figure 3A.17.13.4-201 **CB Dynamic Lateral Pressures on Below-Grade Exterior Wall at Column Line C1**



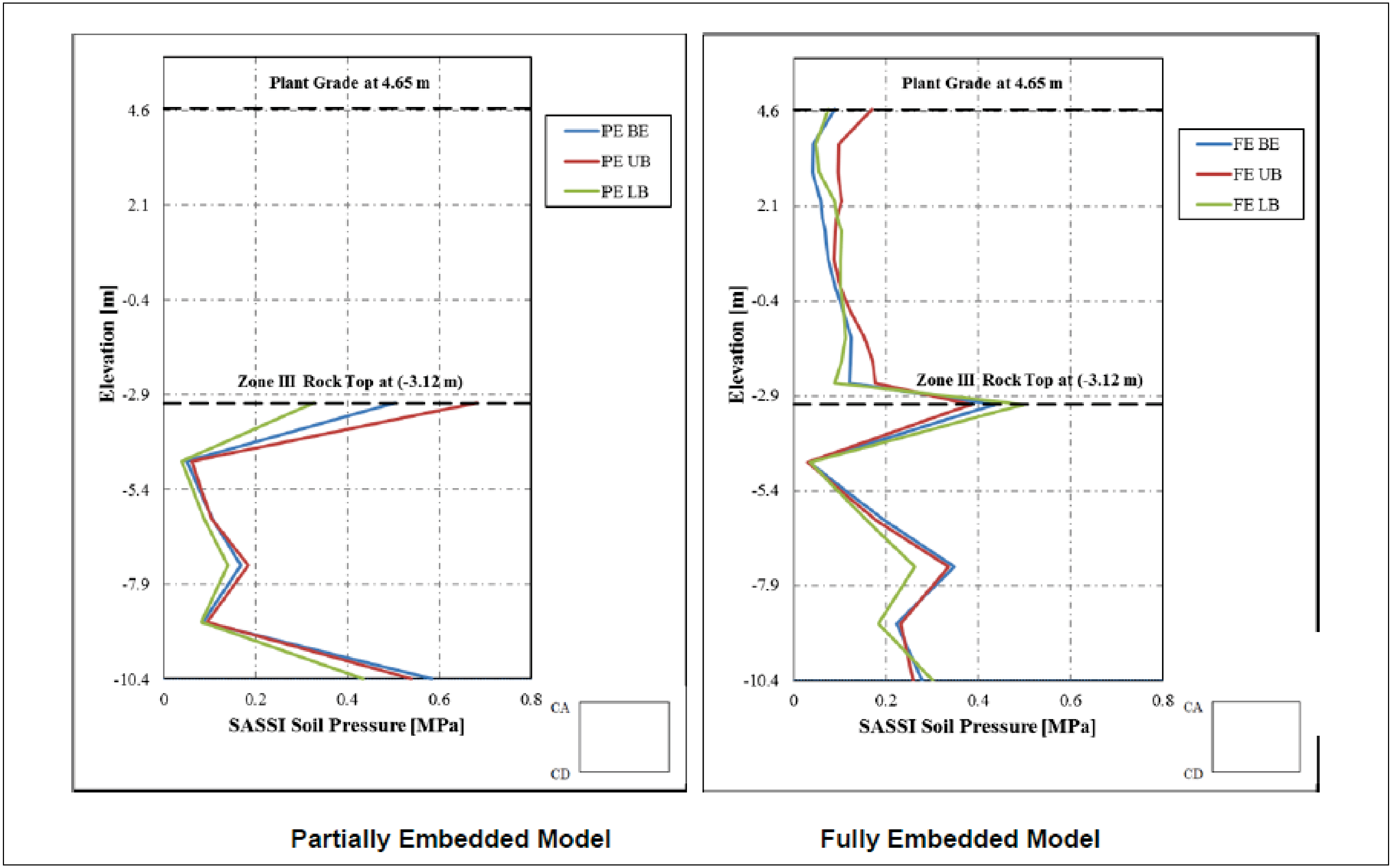
NAPS DEP 3.7-1

Figure 3A.17.13.4-202 **CB Dynamic Lateral Pressures on Below-Grade Exterior Wall at Column Line C5**



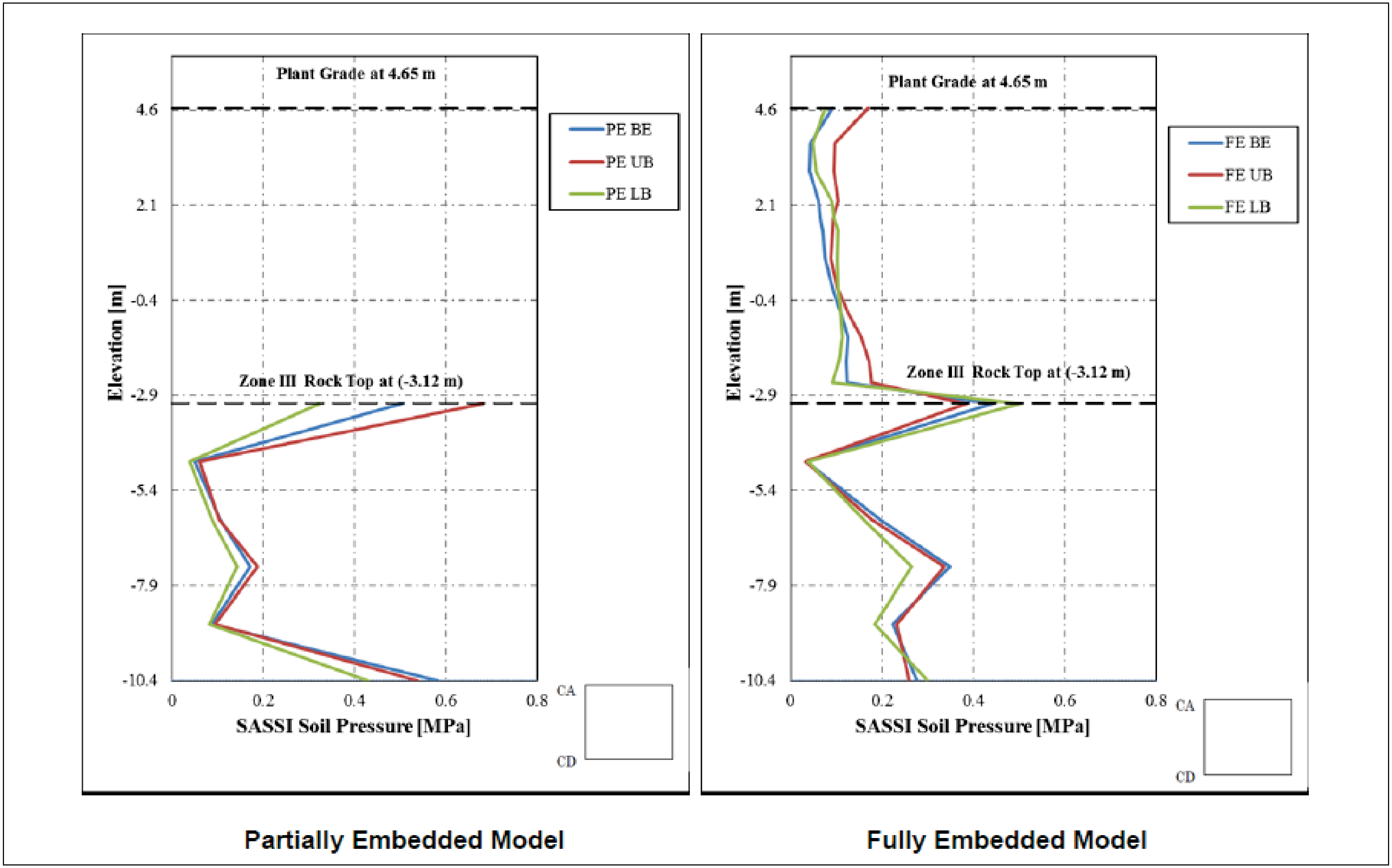
NAPS DEP 3.7-1

Figure 3A.17.13.4-203 **CB Dynamic Lateral Pressures on Below-Grade Exterior Wall Column Line CA**



NAPS DEP 3.7-1

Figure 3A.17.13.4-204 CB Dynamic Lateral Pressures on Below-Grade Exterior Wall at Column Line CD



3A.17.13.5 Base Reactions and Contact Pressures

Results of the SSI analyses for contact spring forces are also used to calculate the time histories of the seismic driving forces used as input for the sliding stability evaluations and dynamic bearing pressure calculations. The time histories of the horizontal and vertical driving seismic forces and overturning base moments are calculated as described in Section 3A.17.12.5.

The time histories of the vertical spring forces obtained from the contact springs at the bottom of the CB basemat are also used to develop the base contact pressure plots that are used to calculate the minimum base contact area. These calculations are performed as described in Section 3A.17.12.5.

Figure 3A.17.13.5-201 presents the time histories of the eccentricities of vertical base reactions obtained from the SSI analyses of the CB model with upper bound stiffness properties and SSE damping for the LB, BE and UB partial column profiles (analysis Cases 7 through 9 in Table 3A.15-202) that provided bounding results. The critical instances of time when the eccentricity of the vertical base reaction has maximum value are identified with red lines. Table 3A.17.13.5-201, which summarizes the calculations of maximum base reaction eccentricities, shows that, as expected, the analysis of the UB partial column subgrade profile yields the maximum value for the vertical base reaction eccentricity. Figure 3A.17.13.5-202 presents a contour plot of the SASSI2010 base contact pressure results for the critical UB partial column analysis at the critical instance of time t = 1.810 sec when the uplift of the CB basemat is the largest. The positive values of the contact pressures in the figures represent tension of the base contact springs. The negative values represent compression of the base contact springs and are used to determine the portion of the basemat that is in contact with the underlying subgrade. The contour plot of the base contact stresses in Figure 3A.17.13.5-202(a) indicates uplift at the center of the CB basemat that is due to the vertical force transmitted through the rigid beam connection between the CB LMSM and the basemat shell elements. This artificial uplift force that is transferred from the LMSM to the foundation by the rigid beam is redistributed uniformly to the other elements of the basemat to calculate the actual contact area. Plot (b) in Figure 3A.17.13.5-202(b) presents the actual contact area after being adjusted for the artificial uplift at the center of the basemat shows that the

base contact area remains larger than 80 percent, which, per the guidance of SRP 3.7.2, ensures that the possible uplift of the CB basemat has a negligible effect on the CB seismic response and the results of site-specific CB SSI analyses documented in this report that are performed on linear elastic models.

NAPS DEP 3.7-1

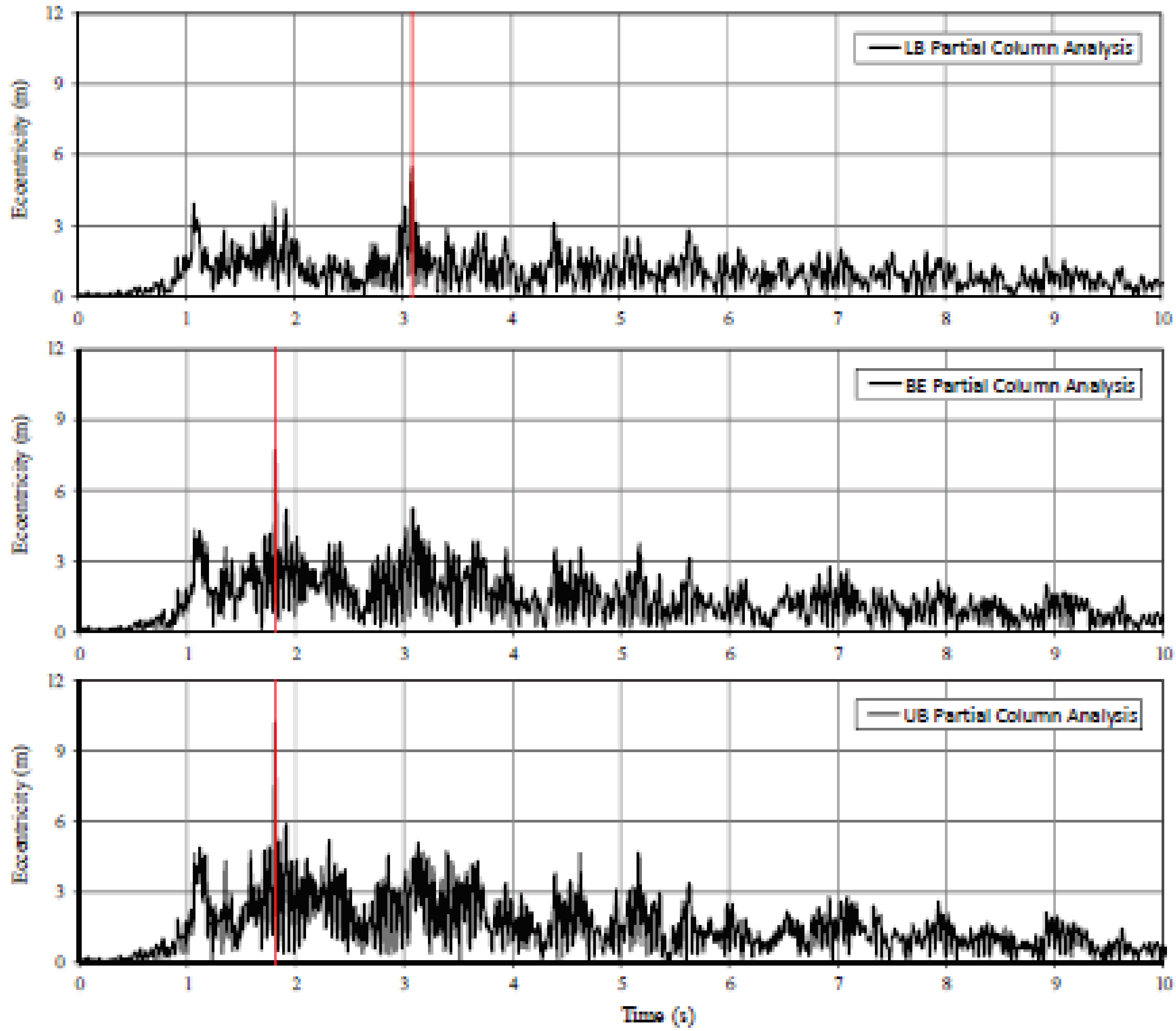
Table 3A.17.13.5-201 **Summary of Maximum Base Reaction Eccentricity Results - CB**

	Partial Column		
Analysis	**BE**	**UB**	**LB**
Max. Eccentricity (m)	7.7	10.2	5.5
at Time (s)	1.815	1.810	3.085

Note: The shaded values are the governing case.

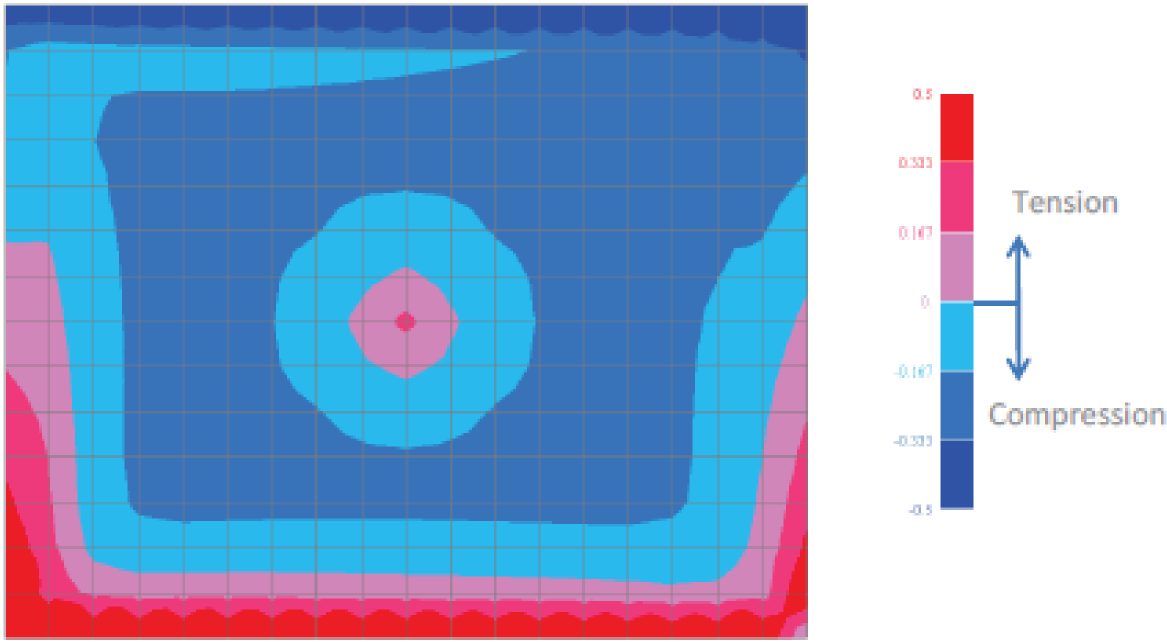
NAPS DEP 3.7-1

Figure 3A.17.13.5-201 **CB Base Reaction Eccentricity**

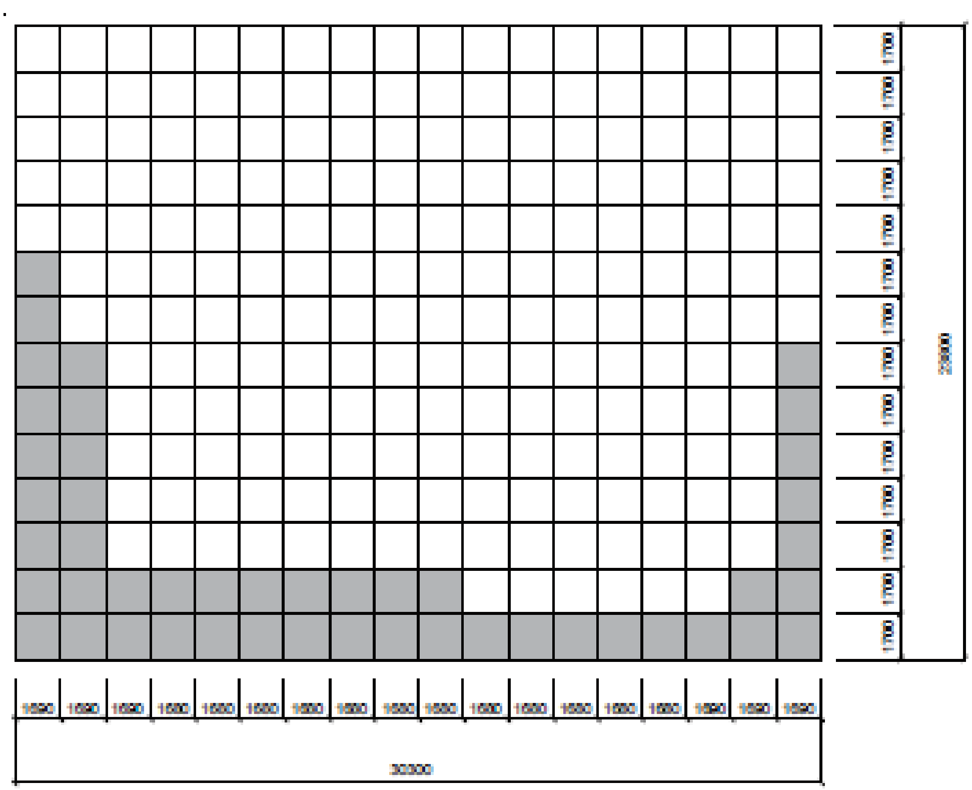


NAPS DEP 3.7-1

Figure 3A.17.13.5-202 **CB Base Contact Area (UB Partial Column Analysis)**



(a) Base Contact Pressures



(b) Minimum Contact Area (81% at critical time t = 1.81s)

3A.17.14 Unit 3 SSI Analysis Results for FWSC

This section presents the results of the site-specific SSI analyses of the FWSC analysis Cases 1 through 9 in Table 3A.15-203.

3A.17.14.1 FWSC SSI Response

Comparisons of SSI analyses results for analyses of the FWSC model with OBE damping for all subgrade profiles and the two different elevations of the input control motion show that the analyses performed using input control motion at the bottom of the concrete fill Elevation 220 ft NAVD88 provide bounding results for the maximum responses of the FWSC structures. Analyses of the UB profile using deep input control motion at the bottom of the concrete fill provides bounding site-specific seismic demands on the FWSC structures. The only exception is the torsional moment demand on the FWS tank structure that is governed by the results of the analysis of the BE profile with deep input control motion at the bottom of the concrete fill.

Development of the site-specific seismic load demands on the FWSC structures in Section 3A.18 are based on the envelope of the maximum force and acceleration results obtained from the SSI analyses of the model with full (uncracked concrete) properties and SSE damping using the in-column control motion that is input at the bottom of the concrete fill Elevation 220 ft NAVD88 (analysis Cases 7 through 9 in Table 3A.15-203). The envelope of the results from the set of six analyses performed on the FWSC model with full (uncracked concrete) stiffness properties and OBE damping (analysis Cases 1 through 6 in Table 3A.15-203) are used for development of the FWSC site-specific ISRS as described in Section 3A.17.14.3.

3A.17.14.2 Maximum Accelerations and Member Forces

Figures 3A.17.14.2-201a through 3A.17.14.2-202b present comparisons of the results of the analyses of LB, BE, and UB subgrade profiles for maximum absolute accelerations at FWS and FPE lumped mass locations, respectively. Figures 3A.17.14.2-201a and 3A.17.14.2-202a compare the results obtained from the analyses with surface input motion at Elevation 282 ft NAVD88 (analysis Cases 1 through 3 in Table 3A.15-203). Figures 3A.17.14.2-201b and 3A.17.14.2-202b compare the results obtained from the analyses with deep input motion at Elevation 220 ft NAVD88 (analysis Cases 4 through 6 in Table 3A.15-203). Table 3A.17.14.2-201 compares the maximum

accelerations results for the FWSC SDOF oscillators representing the out-of-plane response of the FWS roof and the horizontal responses of the sloshing and impulsive hydrodynamic modes of vibration of the water in the FWS tank.

Figures 3A.17.14.2-203a through 3A.17.14.2-204b present comparisons of the maximum shear forces and torsional moments on the FWS and FPE structures obtained from the analyses of the FWSC model for LB, BE and UB subgrade profiles. Figures 3A.17.14.2-203a and 3A.17.14.2-204a compare the results obtained from the analyses with surface input motion at Elevation 282 ft NAVD88 (analysis Cases 1 through 3 in Table 3A.15-203). Figures 3A.17.14.2-203b and 3A.17.14.2-204b compare the results obtained from the analyses with deep input motion at Elevation 220 ft NAVD88 (analysis Cases 4 through 6 in Table 3A.15-203).

Figures 3A.17.14.2-201a through 3A.17.14.2-204b show the analysis of UB subgrade profile with deep input control motion (analysis Case 6 in Table 3A.15-203) governs the maximum responses of the FWSC structures with the exception of torsional moment demand on the FWS that are governed by the analysis of the BE profile with deep input motion at Elevation 220 ft NAVD88 (analysis Case 5 in Table 3A.15-203). Comparisons in Table 3A.17.14.2-201 show that analysis Case 6 also governs the maximum responses of the FWSC SDOF oscillators.

The comparisons of the results obtained from the analysis with surface and deep input motions show that the high frequency content of the input motion transmitted through the concrete fill block resulted in significantly higher results for the FWS and FPE maximum accelerations and forces.

NAPS DEP 3.7-1

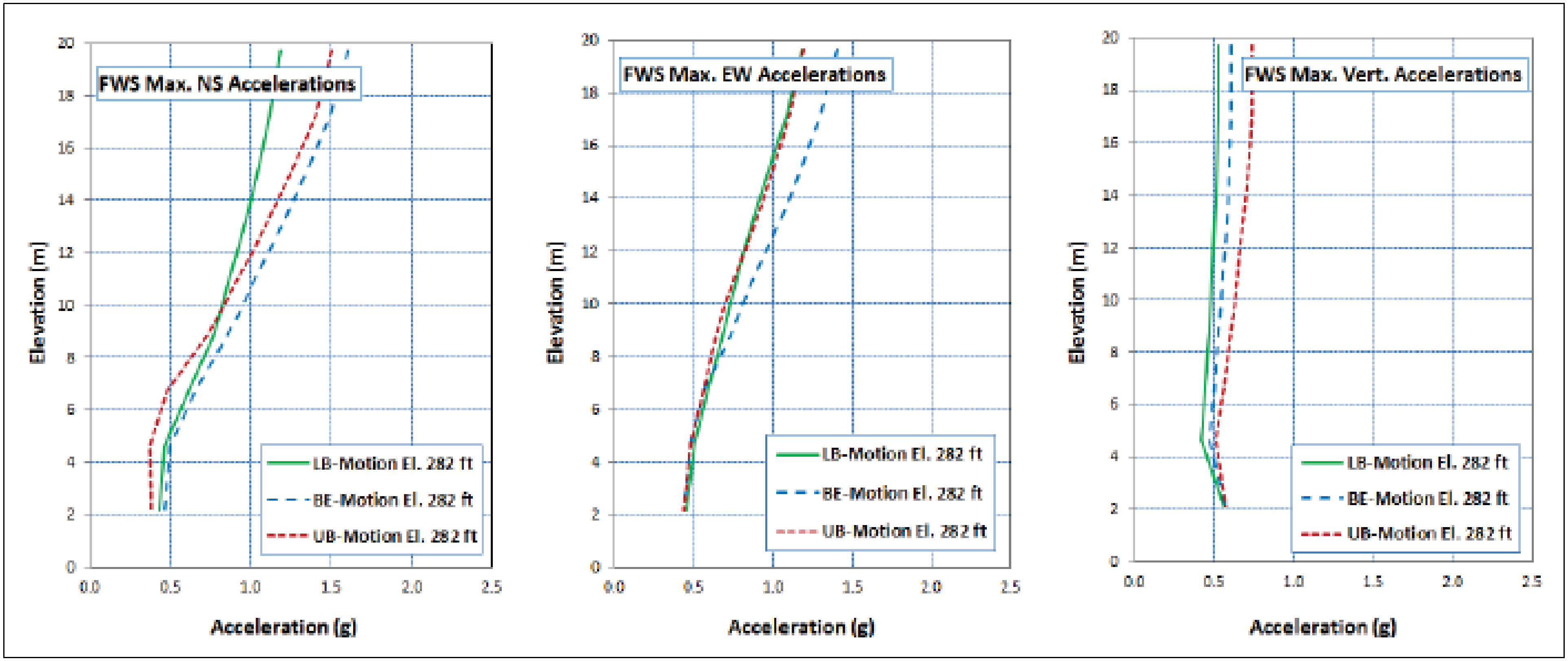
Table 3A.17.14.2-201 **Maximum Accelerations of SDOF Oscillators - FWSC**

SDOF Oscillator				Acceleration (g)						
				Motion Elevation 282 ft.			Motion Elevation 220 ft.			
Elev. (m)	**Node No.**	**Description**	**Direction**	**LB**	**BE**	**UB**	**LB**	**BE**	**UB**	**Envelope**
19.70	11	FWS Roof	Vert. (Z)	1.34	1.62	1.58	2.10	3.26	*4.22*	4.22
12.10	60	FWS Water Sloshing Mode	NS (X)	*0.10*	*0.10*	*0.10*	*0.10*	*0.10*	*0.10*	0.10
			EW (Y)	0.07	0.07	0.07	*0.09*	*0.09*	*0.09*	0.09
8.81	30	FWS Water Impulsive Mode	NS (X)	0.77	0.85	0.72	0.80	0.85	1.02	1.02
			EW (Y)	0.68	0.73	0.64	0.74	0.81	1.08	1.08

Note: The shaded values are the governing case(s)

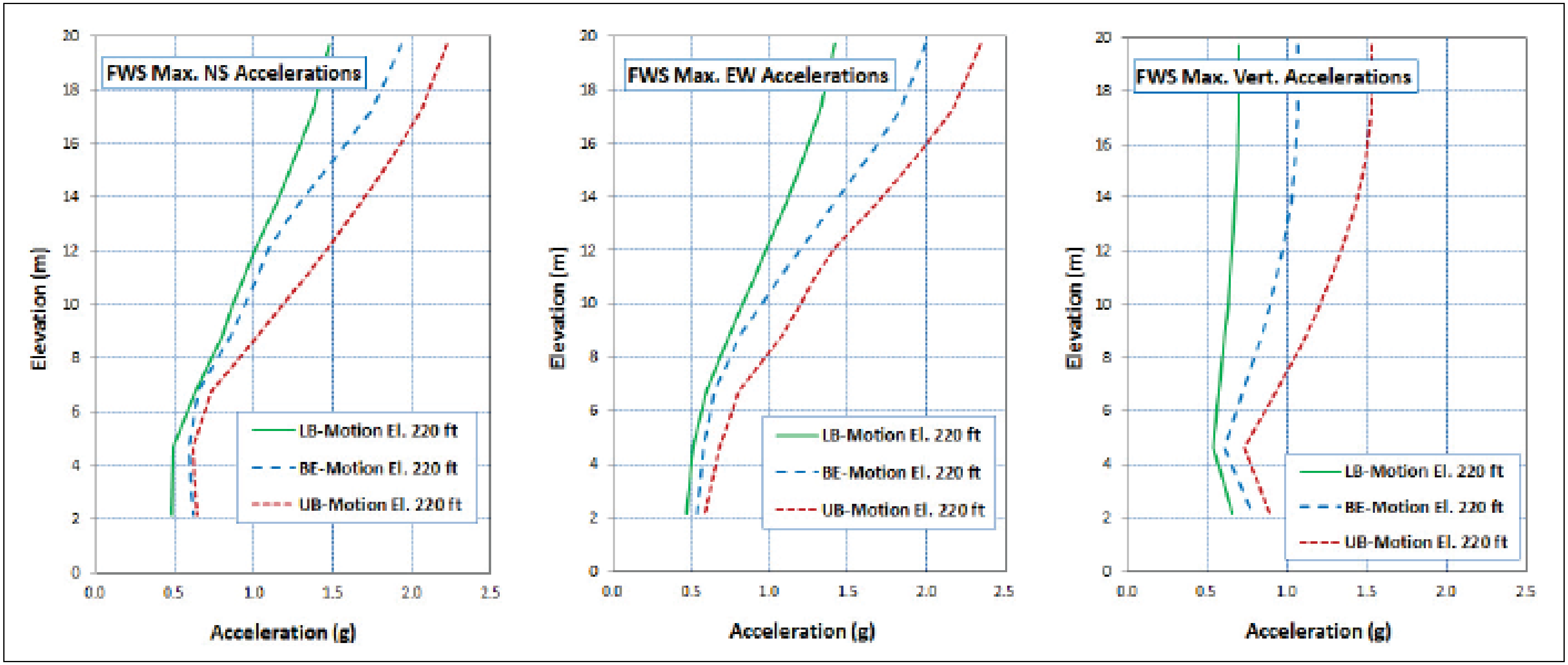
NAPS DEP 3.7-1

Figure 3A.17.14.2-201a FWS Maximum Accelerations from Analyses of FWSC UC_{OBE} Model with Surface Input Motion at Elevation 282 ft



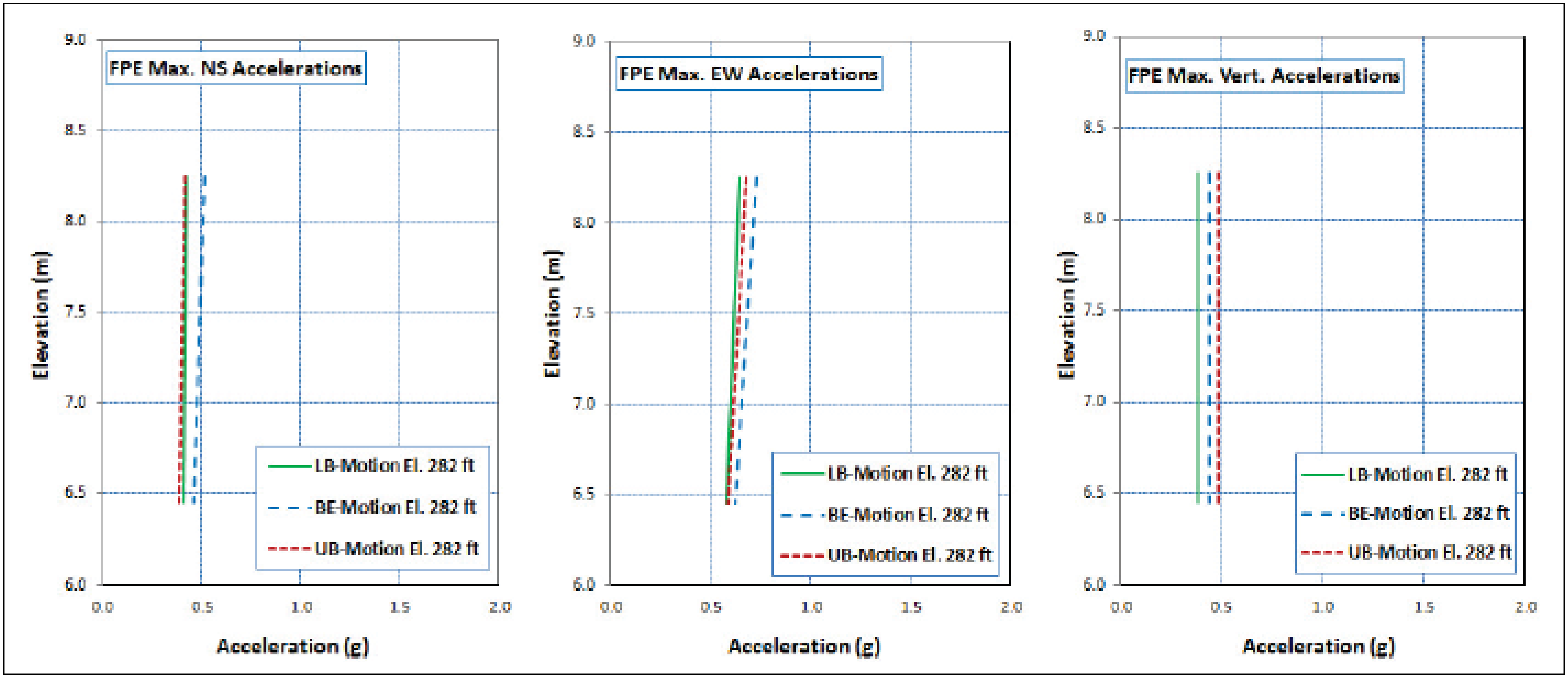
NAPS DEP 3.7-1

Figure 3A.17.14.2-201b **FWS Maximum Accelerations from Analyses of FWSC UC$_{OBE}$ Model with Deep Input Motion at Elevation 220 ft**



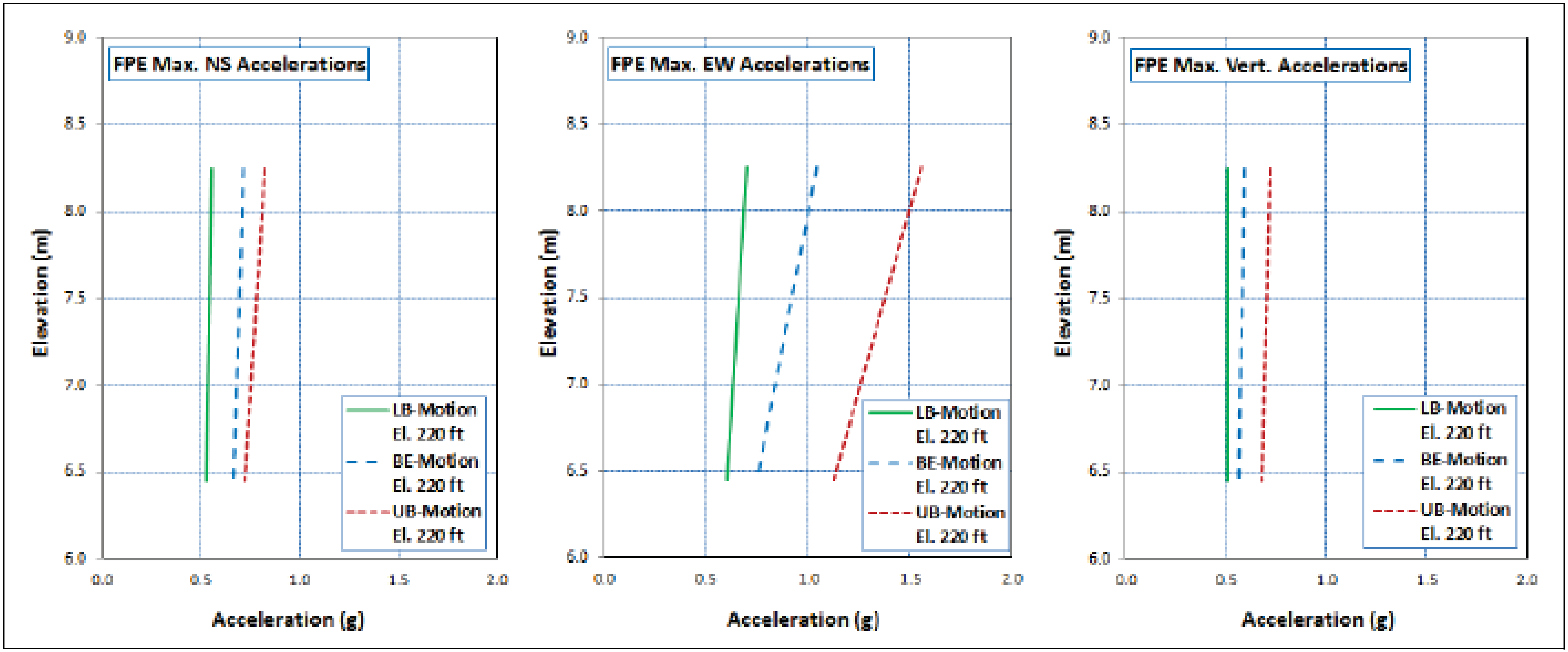
NAPS DEP 3.7-1

Figure 3A.17.14.2-202a **FPE Maximum Accelerations from Analyses of FWSC UC_{OBE} Model with Surface Input Motion at Elevation 282 ft**



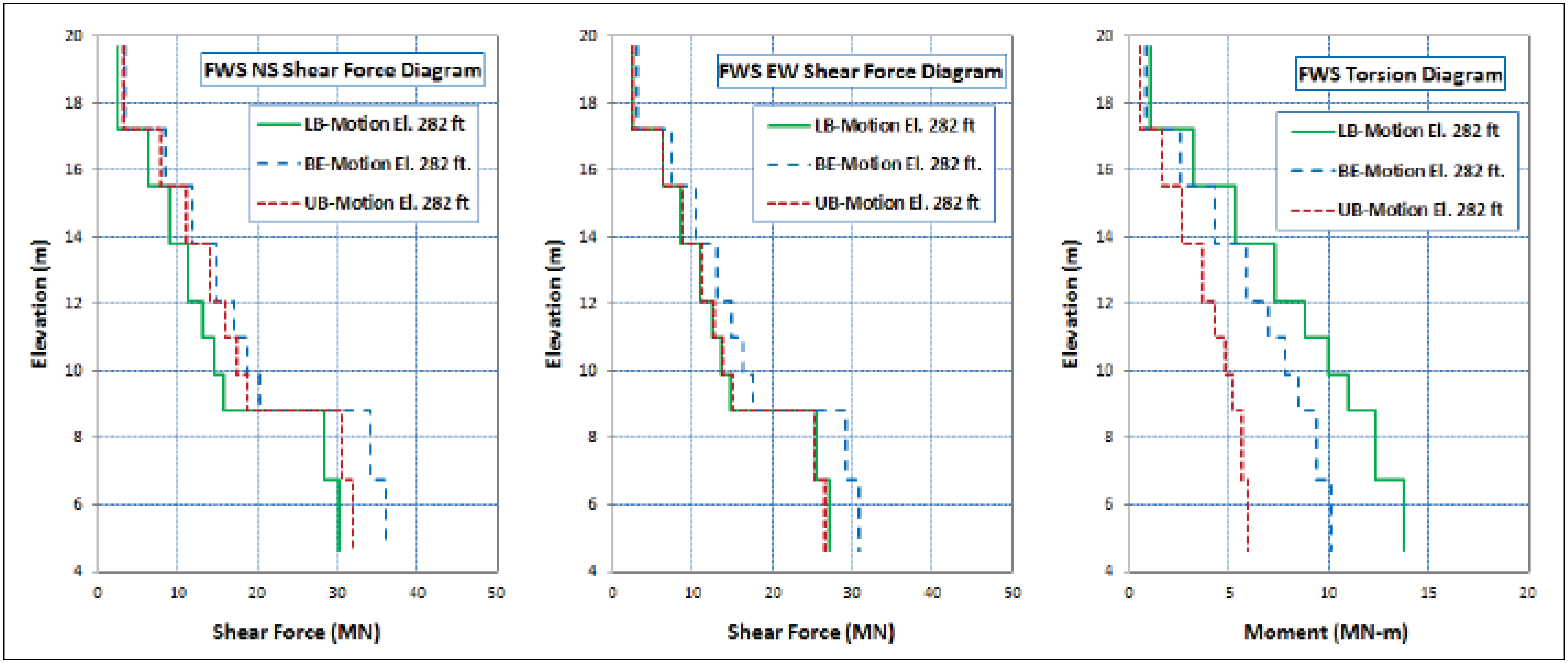
NAPS DEP 3.7-1

Figure 3A.17.14.2-202b **FPE Maximum Accelerations from Analyses of FWSC UC_{OBE} Model with Deep Input Motion at Elevation 220 ft**



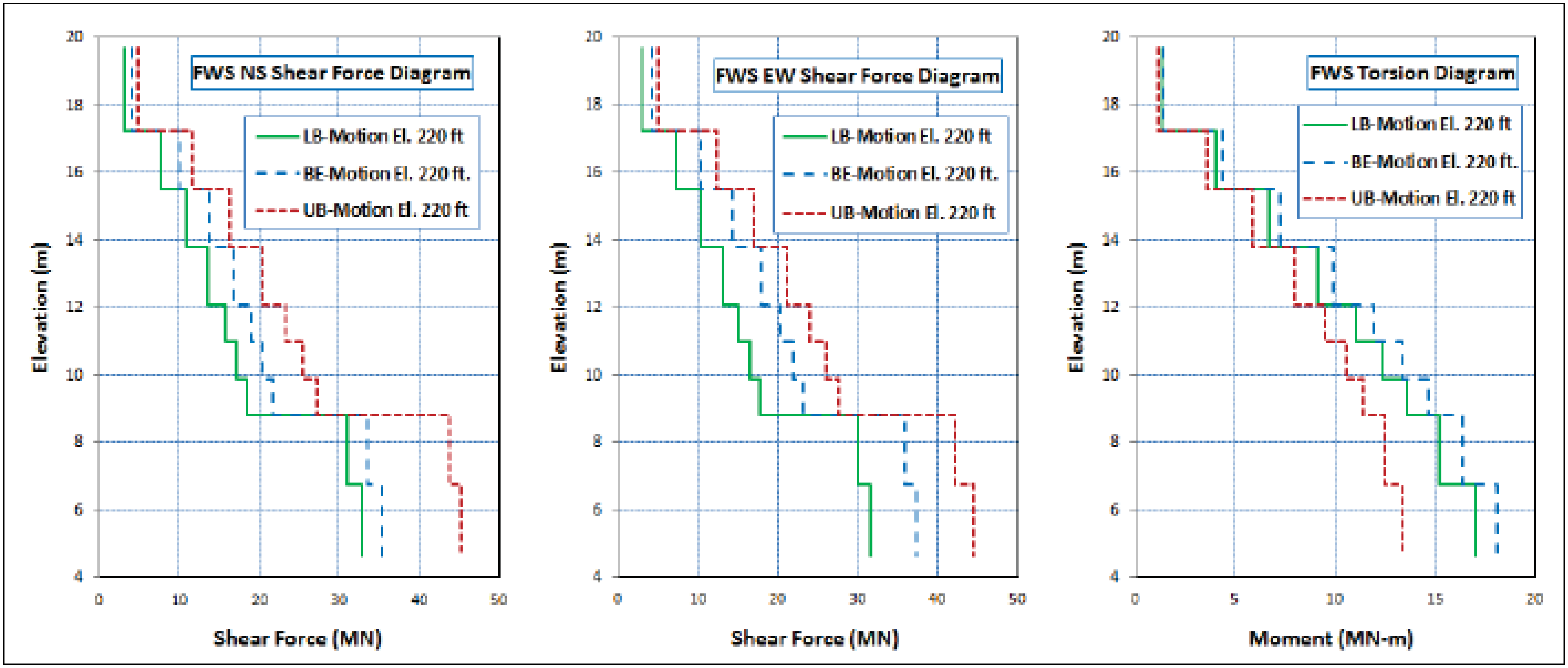
NAPS DEP 3.7-1

Figure 3A.17.14.2-203a **FWS Maximum Shear Forces and Torsion from Analyses of FWSC UC$_{OBE}$ Model with Surface Input Motion at Elevation 282 ft**



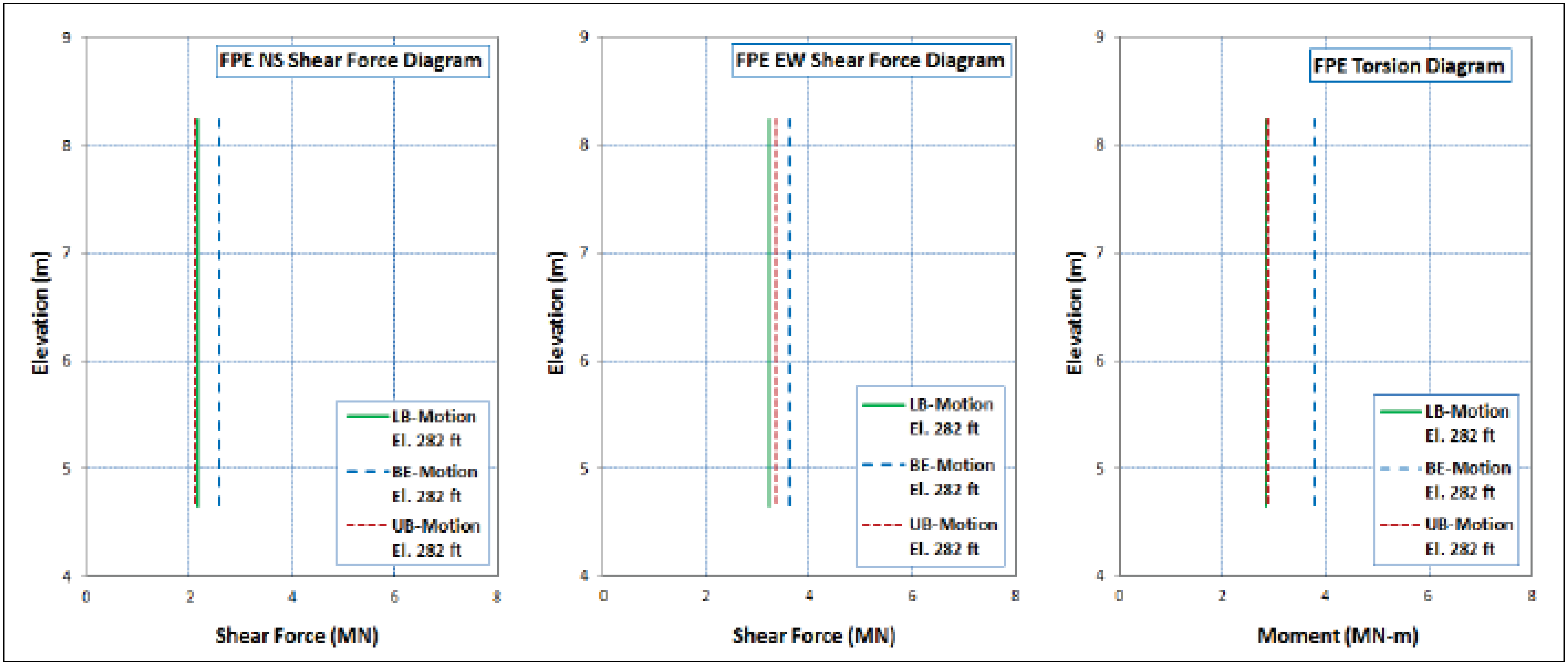
NAPS DEP 3.7-1

Figure 3A.17.14.2-203b **FWS Maximum Shear Forces and Torsion from Analyses of FWSC UC_{OBE} Model with Deep Input Motion at Elevation 220 ft**



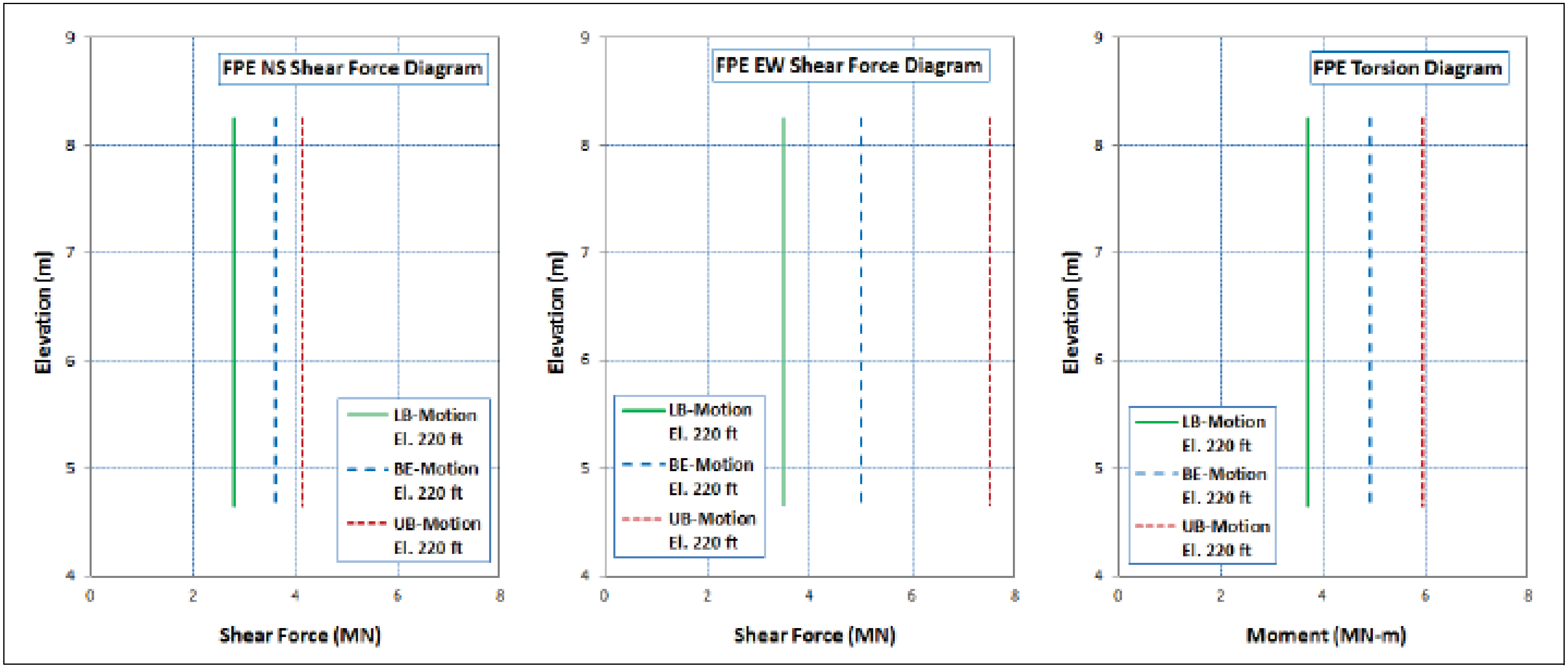
NAPS DEP 3.7-1

Figure 3A.17.14.2-204a FPE Maximum Shear Forces and Torsion from Analyses of FWSC UC_{OBE} Model with Surface Input Motion at Elevation 282 ft



NAPS DEP 3.7-1

Figure 3A.17.14.2-204b FPE Maximum Shear Forces and Torsion from Analyses of FWSC UC_{OBE} Model with Deep Input Motion at Elevation 220 ft



3A.17.14.3 Acceleration Response Spectra

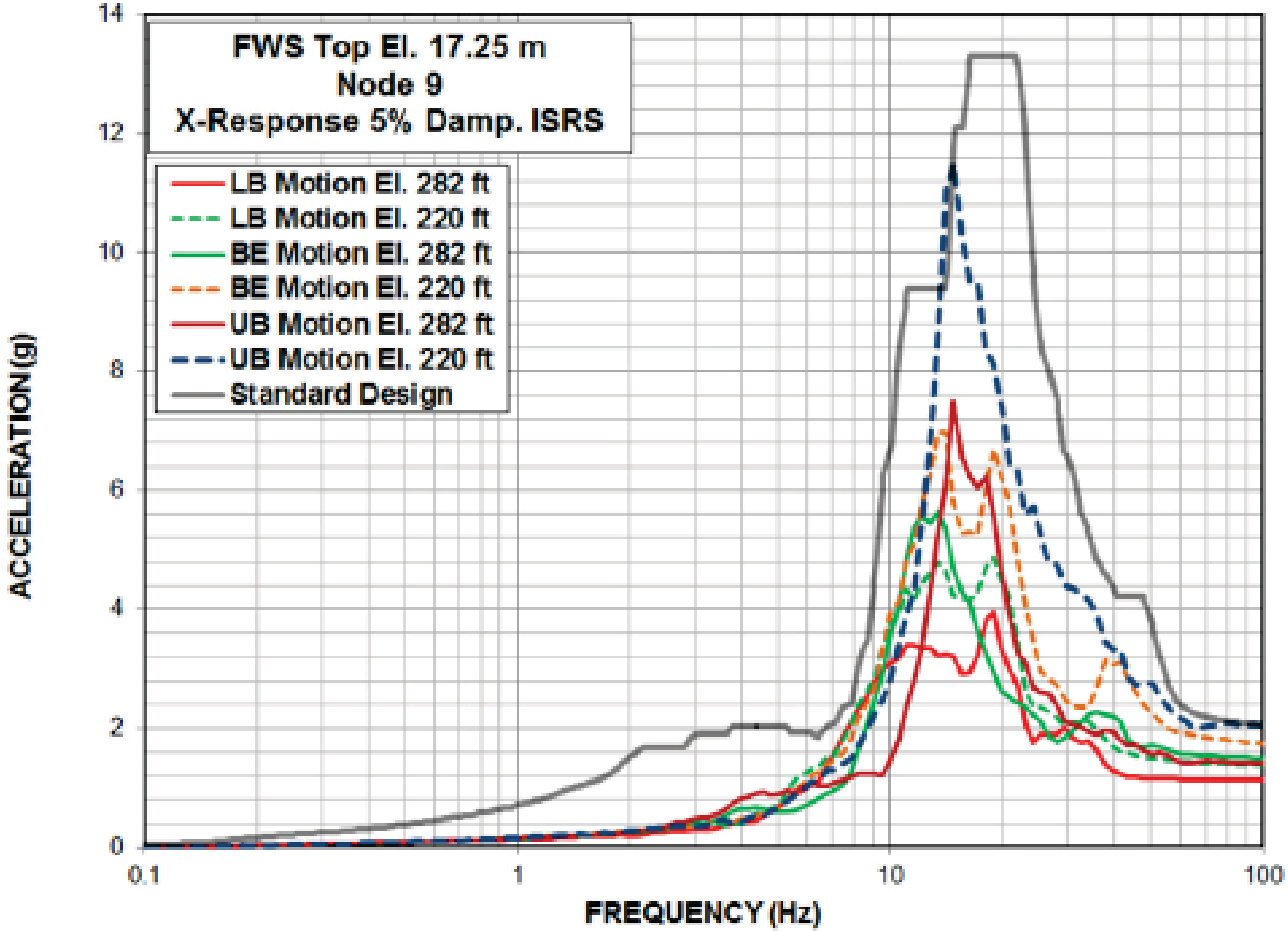
Figures 3A.17.14.3-201a through 3A.17.14.3-201d, Figures 3A.17.14.3-202a through 3A.17.14.3-202d, and Figures 3A.17.14.3-203a through 3A.17.14.3-203d, respectively, compare the 5 percent damped ISRS for the response in NS(x), EW(y), and vertical (z) directions at the four key locations within the FWSC with the corresponding 5 percent damped standard design ISRS in DCD Section 3A.9. These spectra are obtained from the analyses of the UC_{OBE} model for the LB, BE, and UB subgrade profiles using surface and deep input control motions at Elevation 282 ft NAVD88 and Elevation 220 ft NAVD88.

Comparison of the ISRS results obtained from the different site-specific SSI analyses show that the responses obtained from the analyses with deep input motion at Elevation 220 ft NAVD88 govern the horizontal and vertical ISRS for frequencies higher than 10 Hz and 18 Hz, respectively. The analyses with surface input motion at Elevation 282 ft NAVD88 (analysis Case 1 in Table 3A.15-203) govern the ISRS at frequencies lower than 9 Hz.

Comparisons in Figures 3A.17.14.3-201a through 3A.17.14.3-203d indicate that the FWSC site-specific ISRS can exceed the corresponding standard design spectra. Large exceedances can be observed in the site-specific ISRS for the EW response of the FWSC basemat and the FPE at frequencies between 5 and 15 Hz. An exceedance in the FWS roof SDOF oscillator ISRS occurs at frequencies corresponding to the frequencies of the oscillator.

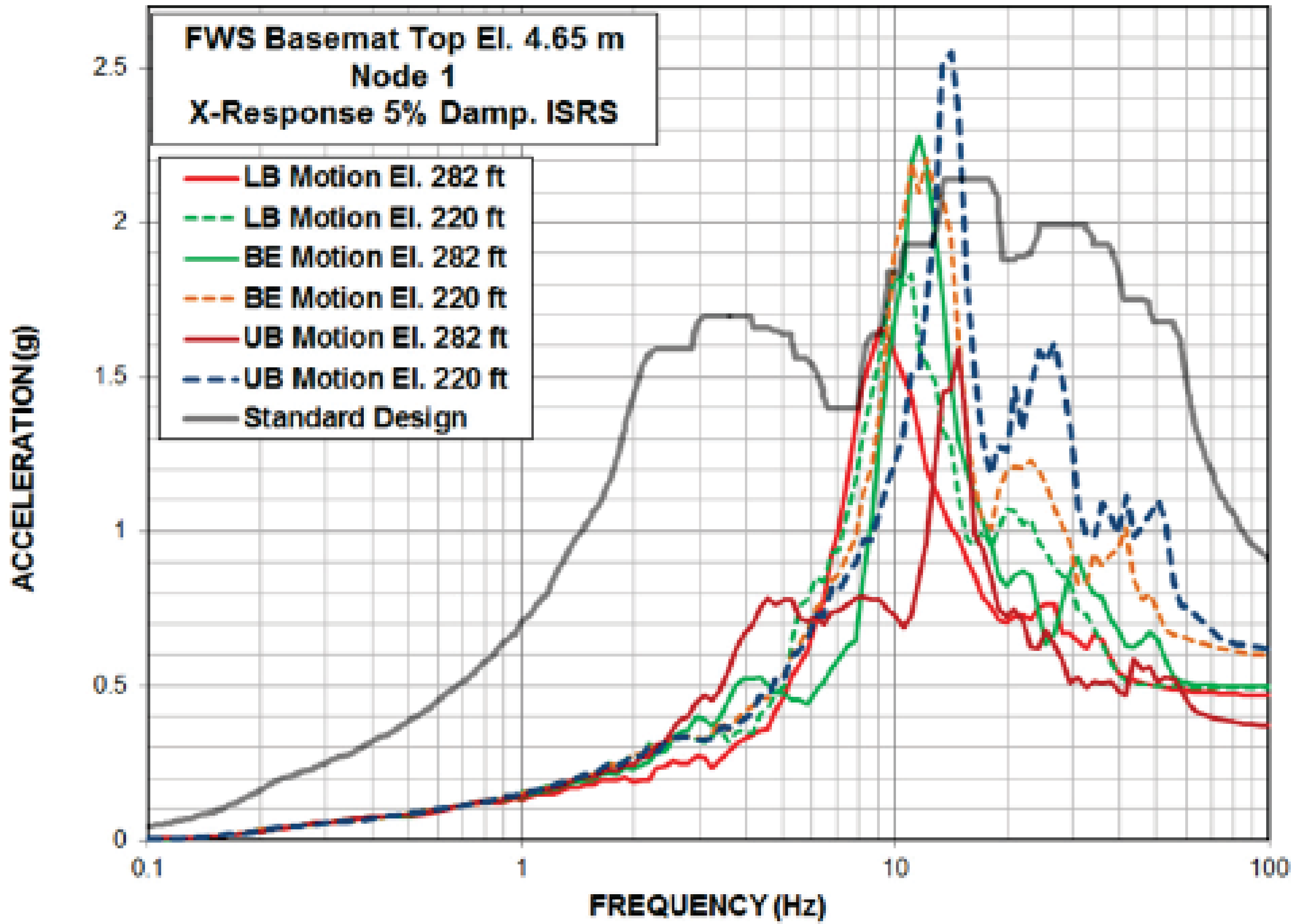
The ISRS exceedances are due: 1) to the higher energy content of the Unit 3 site-specific design motion as compared to the standard design CSDRS at mid-range frequencies and 2) to the lower OBE structural damping values used for the site-specific SSI analyses versus the SSE damping values used for the standard design SSI analyses. Such exceedances will be considered in the site-specific design for Unit 3 as described in Section 3A.18.

NAPS DEP 3.7-1 Figure 3A.17.14.3-201a **Comparison of ISRS for Response of FWS Wall Top in NS (X) Direction**



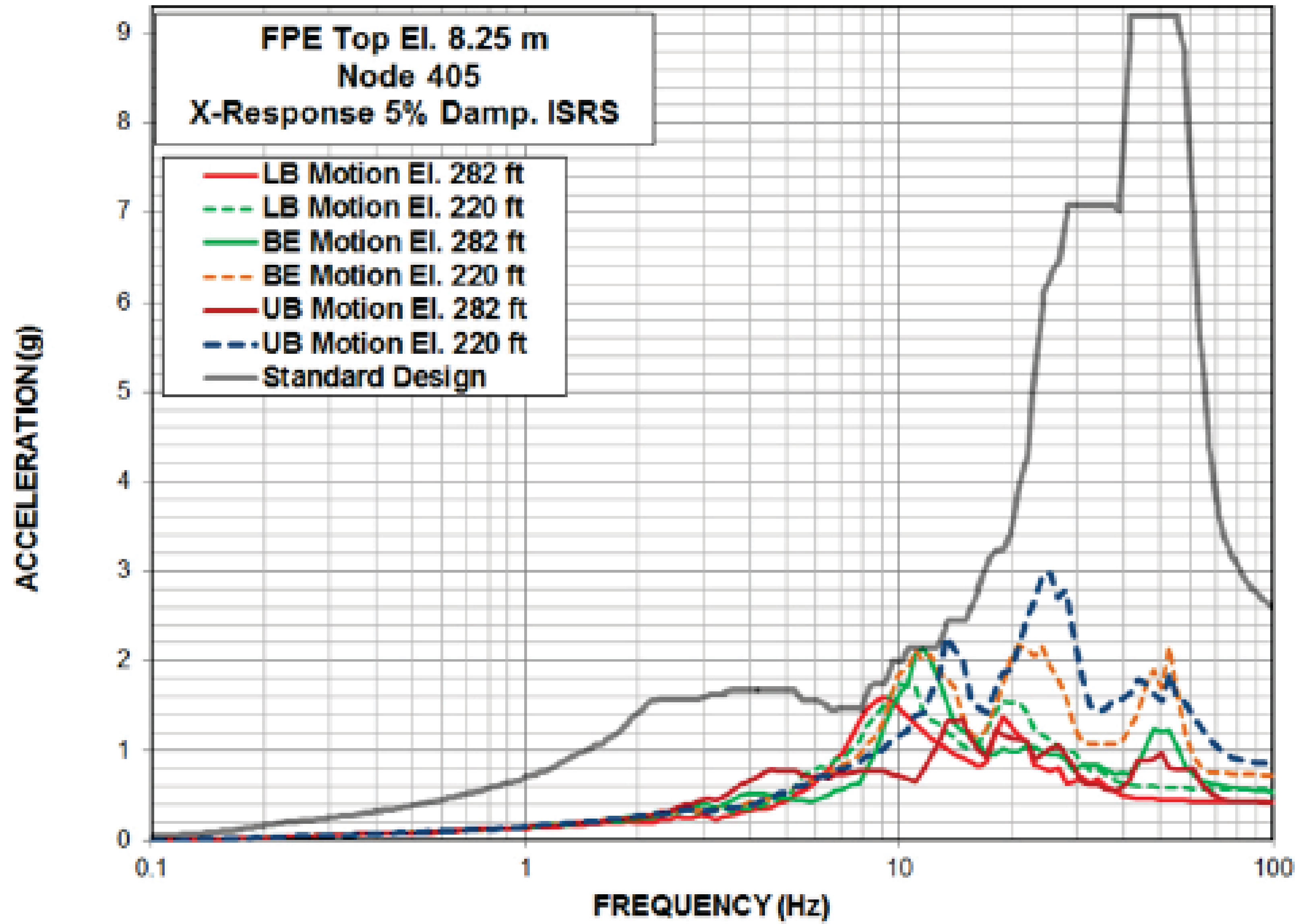
NAPS DEP 3.7-1

Figure 3A.17.14.3-201b **Comparison of ISRS for Response of FWS Basemat in NS (X) Direction**



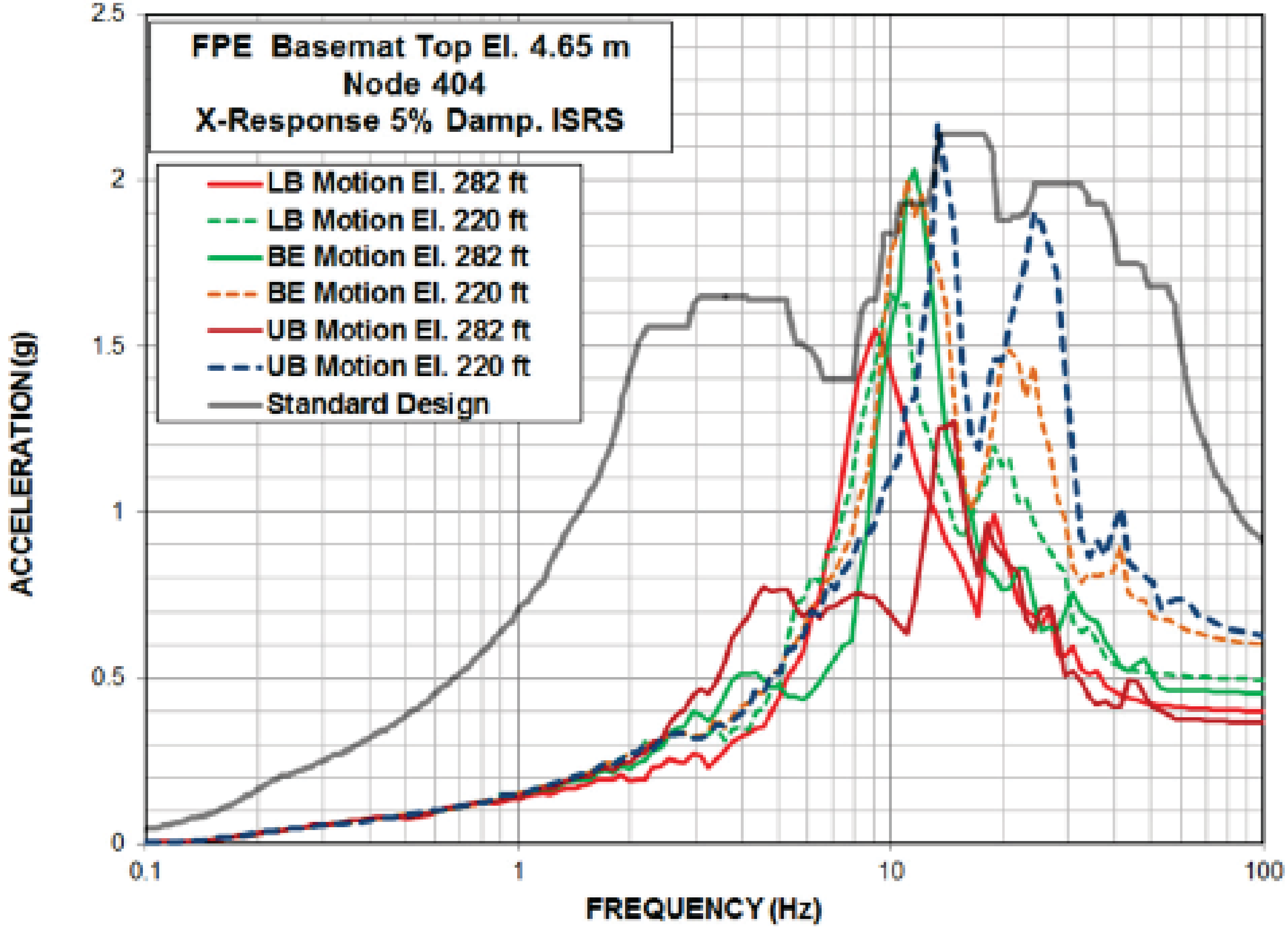
NAPS DEP 3.7-1

Figure 3A.17.14.3-201c **Comparison of ISRS for Response of FPE Top in NS (X) Direction**



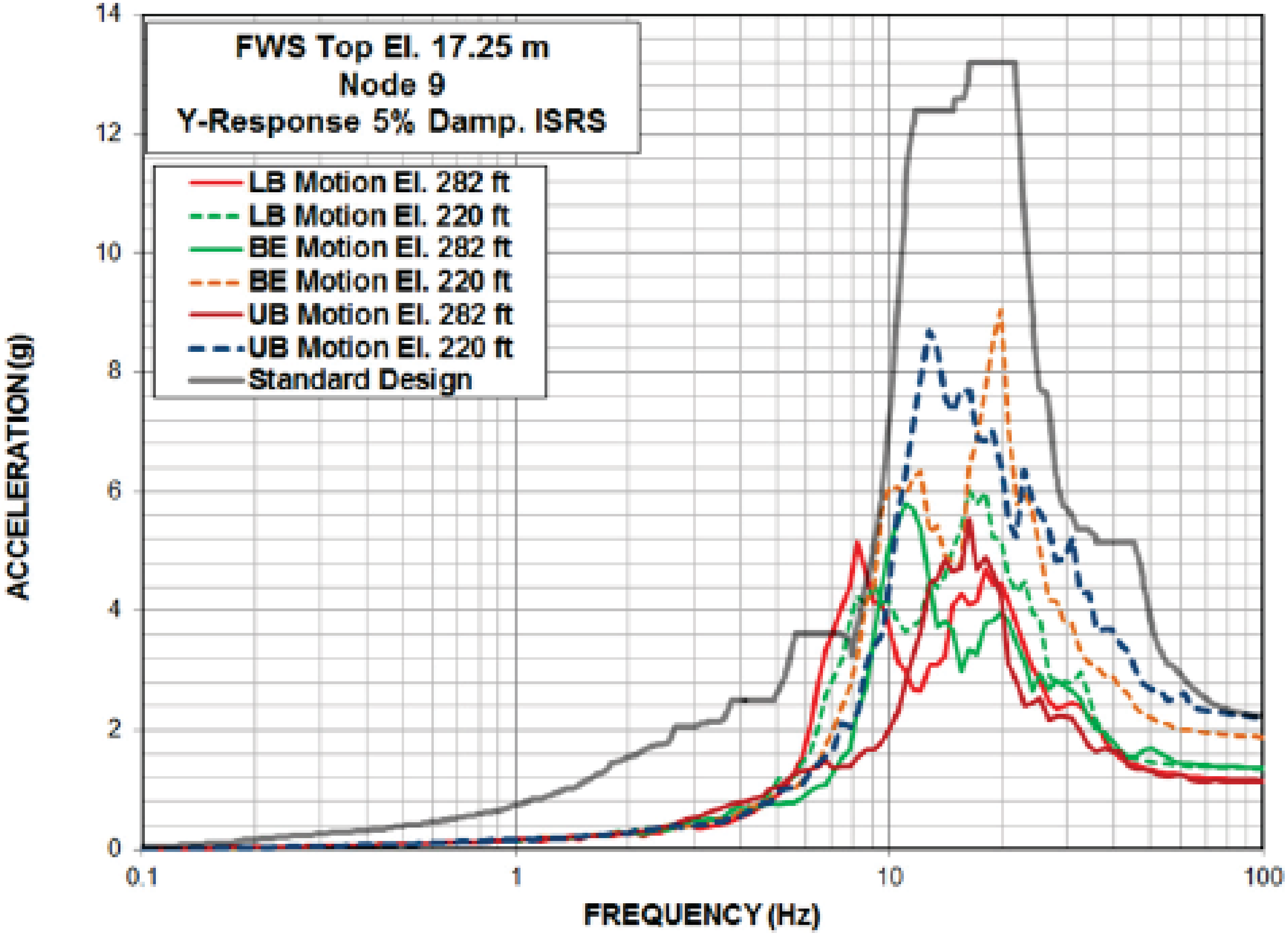
NAPS DEP 3.7-1

Figure 3A.17.14.3-201d **Comparison of ISRS for Response of FPE Basemat in NS (X) Direction**



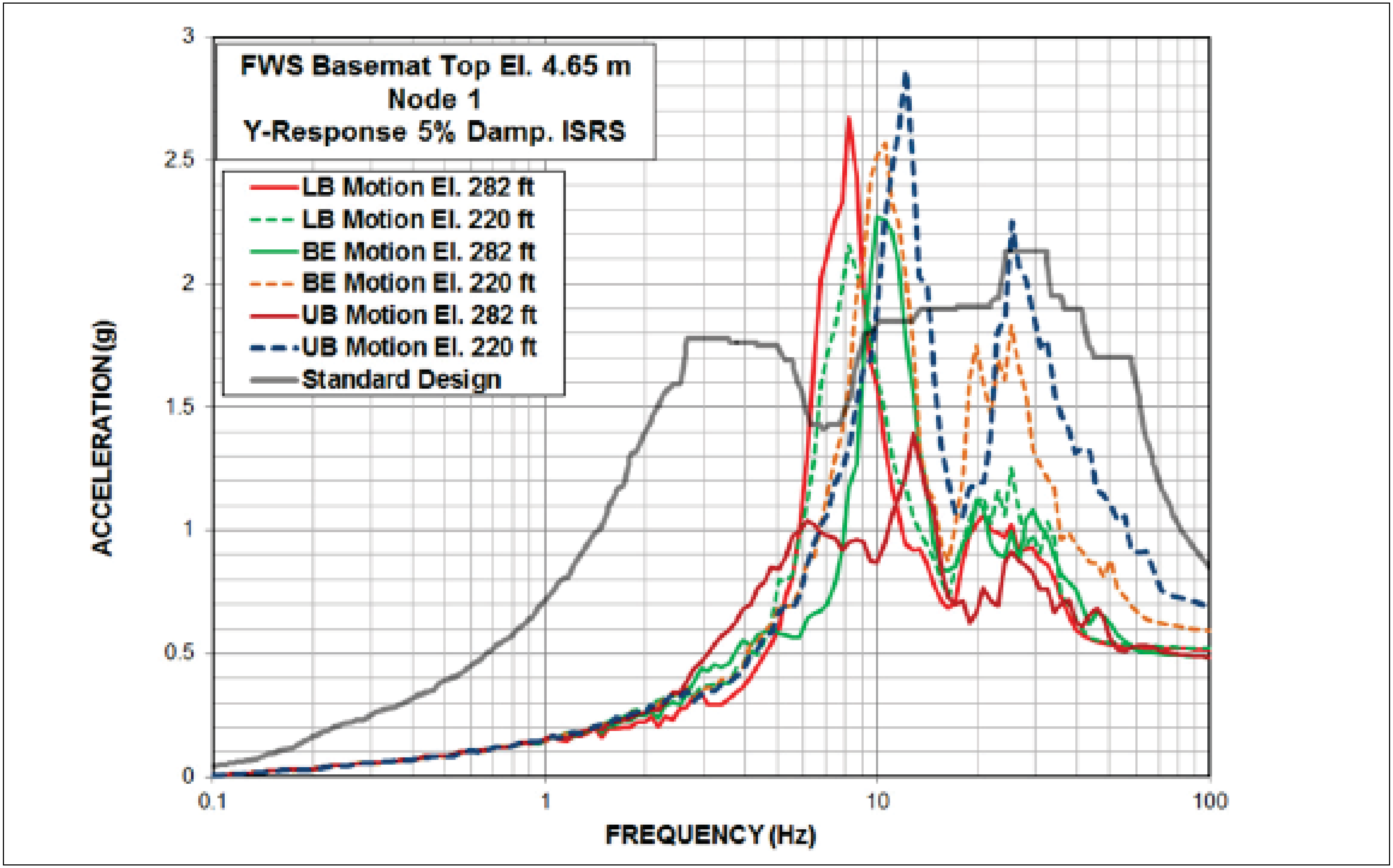
NAPS DEP 3.7-1

Figure 3A.17.14.3-202a **Comparison of ISRS for Response of FWS Wall Top in EW (Y) Direction**



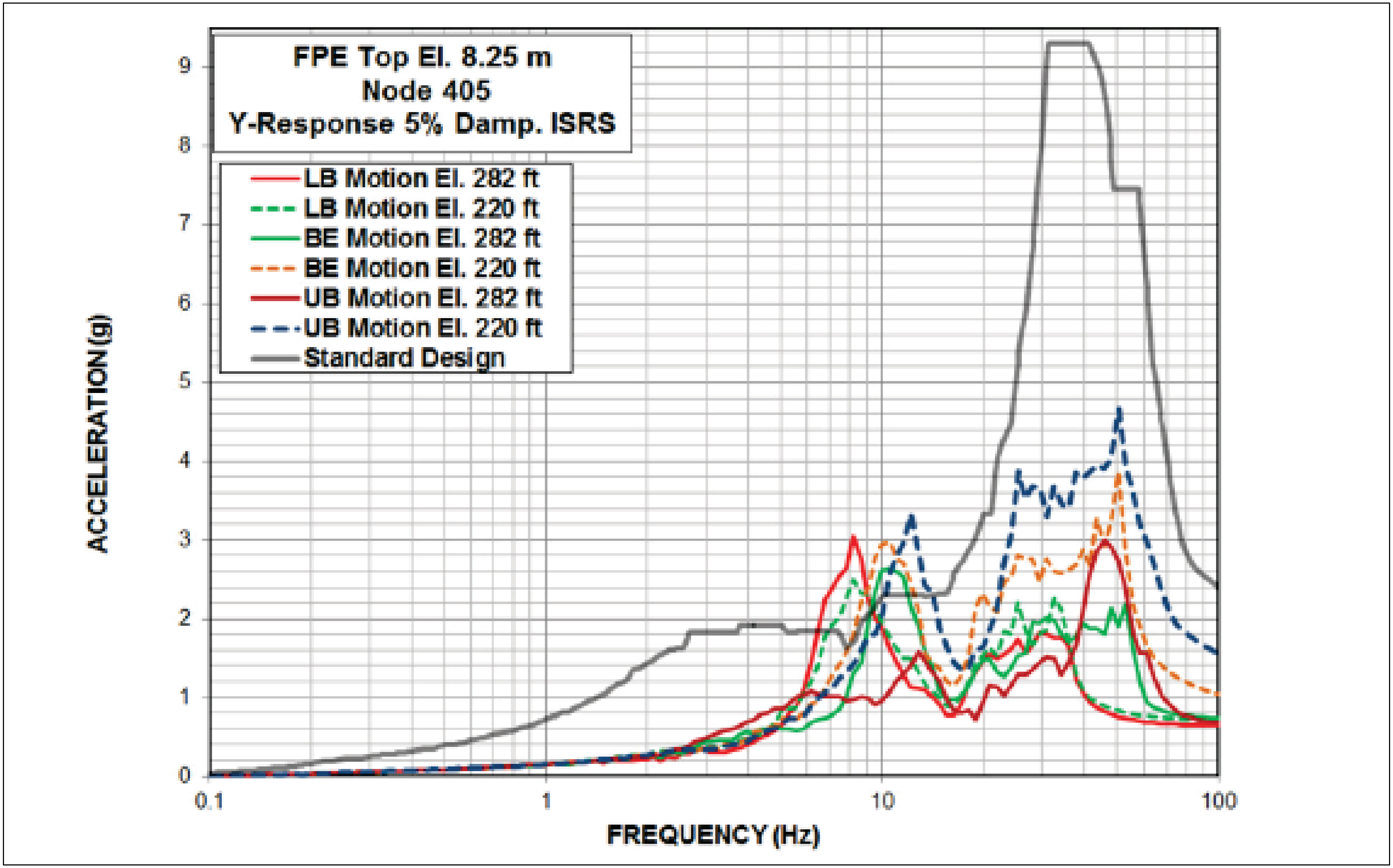
NAPS DEP 3.7-1

Figure 3A.17.14.3-202b **Comparison of ISRS for Response of FWS Basemat in EW (Y) Direction**



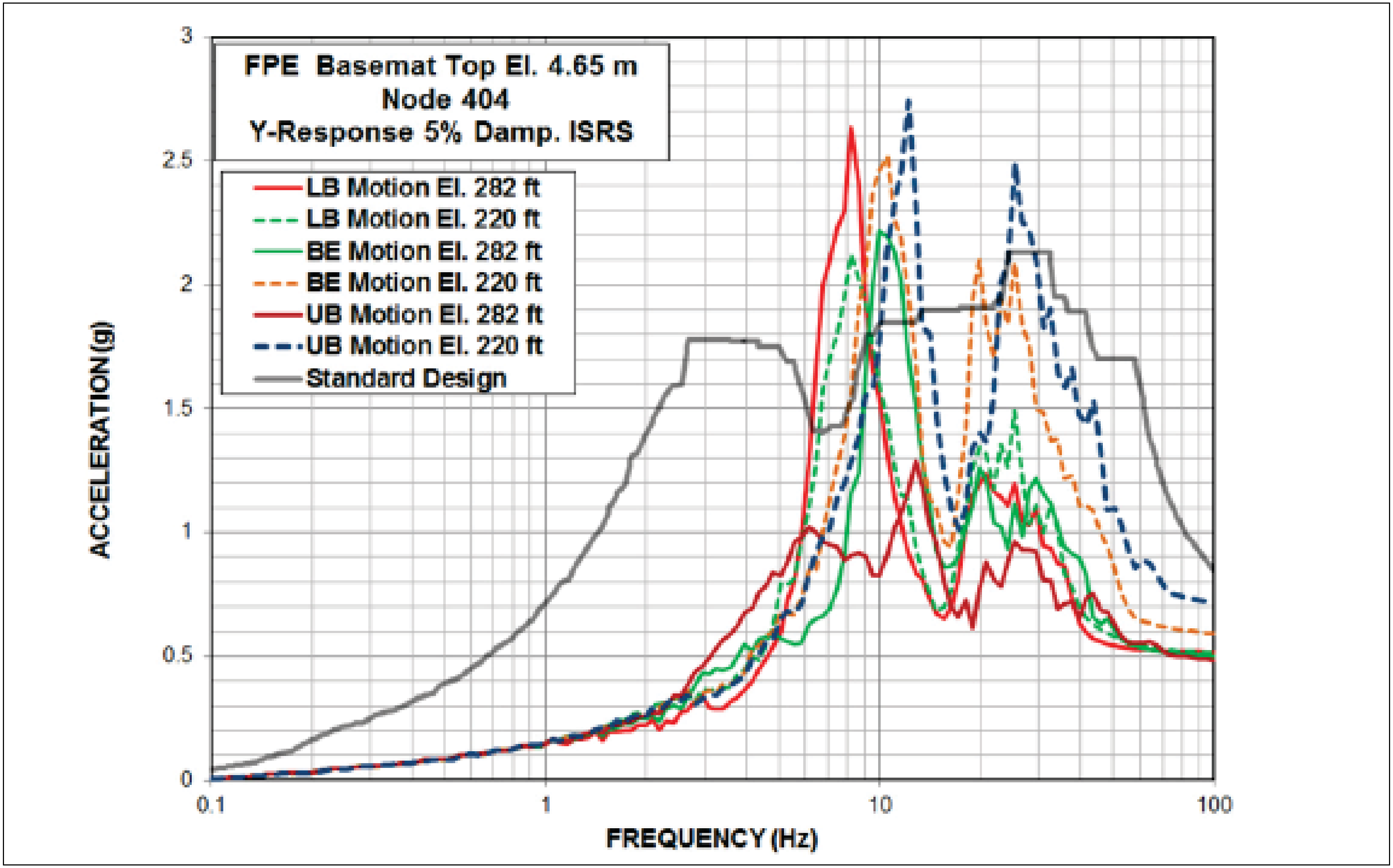
NAPS DEP 3.7-1

Figure 3A.17.14.3-202c **Comparison of ISRS for Response of FPE Top in EW (Y) Direction**



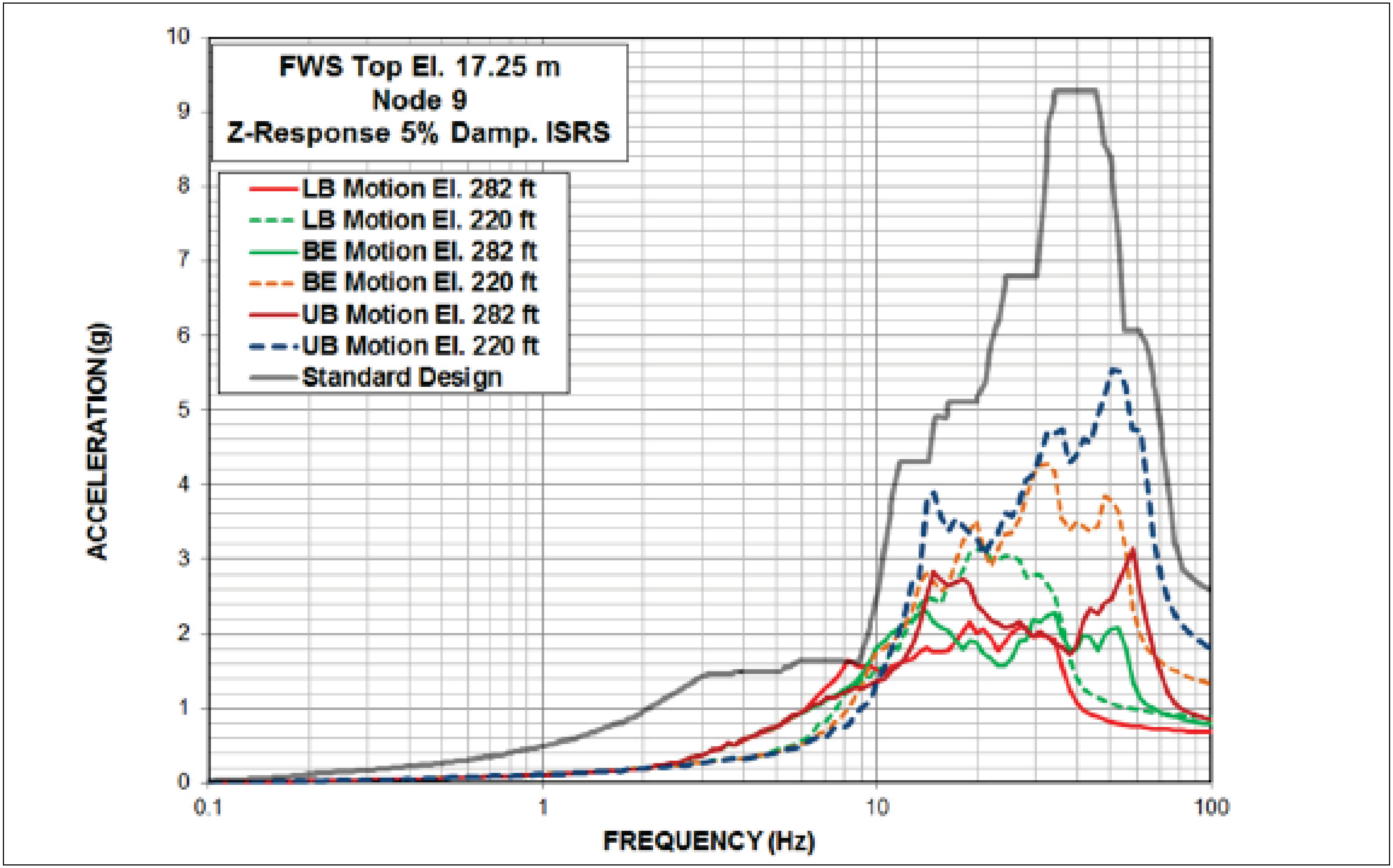
NAPS DEP 3.7-1

Figure 3A.17.14.3-202d Comparison of ISRS for Response of FPE Basemat in EW (Y) Direction



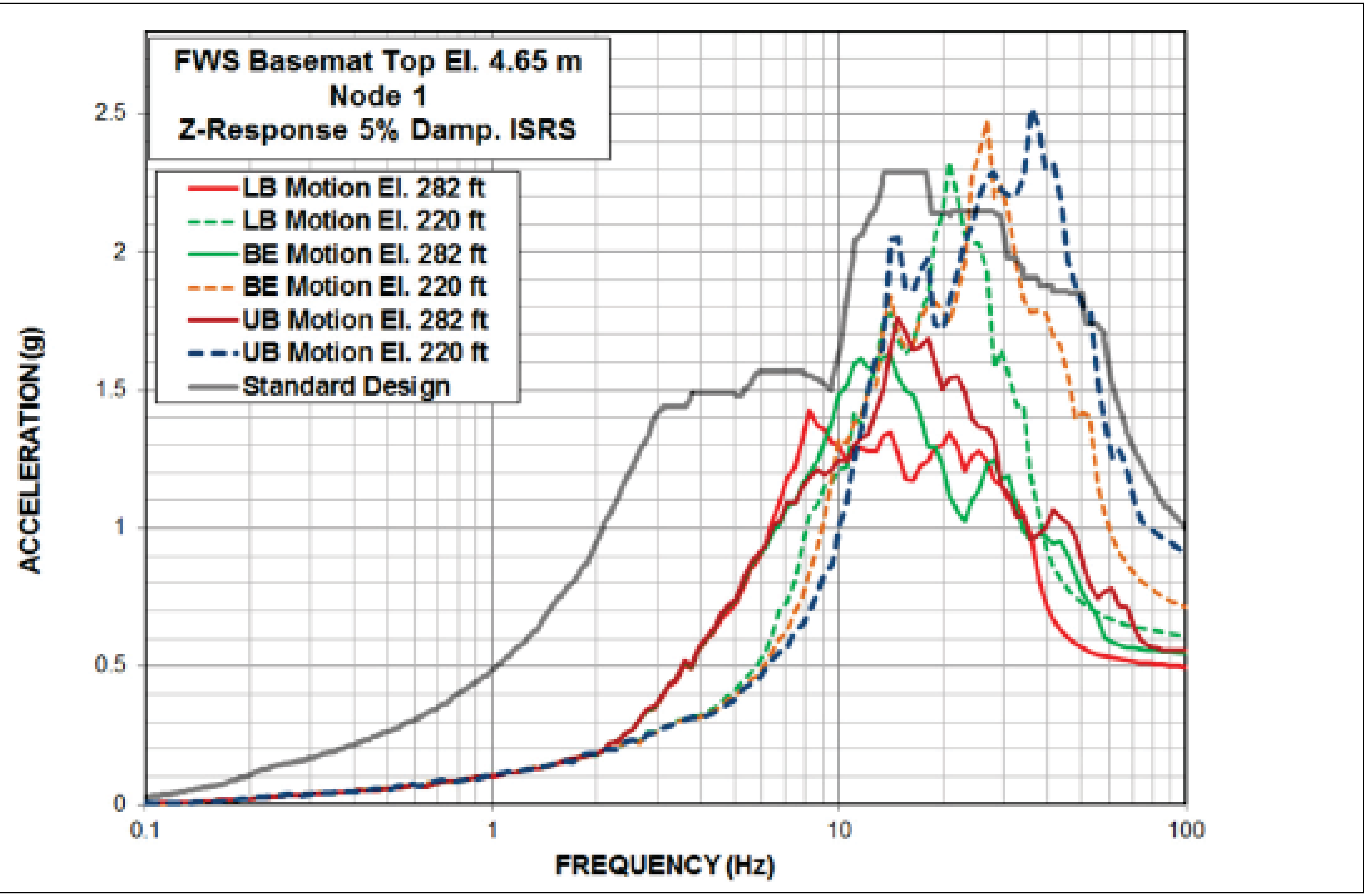
NAPS DEP 3.7-1

Figure 3A.17.14.3-203a **Comparison of ISRS for Response of FWS Wall Top in Vert. (Z) Direction**

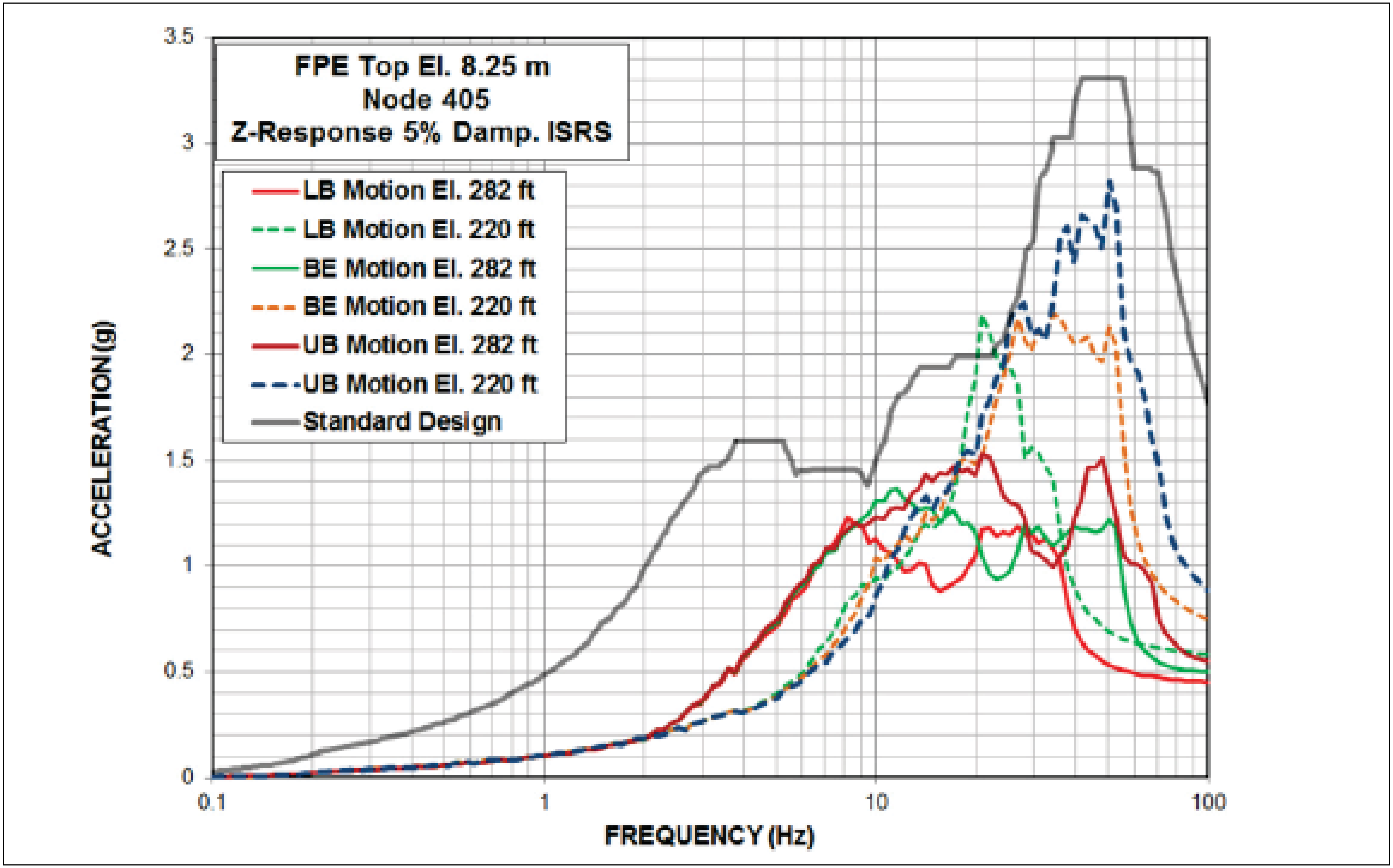


NAPS DEP 3.7-1

Figure 3A.17.14.3-203b **Comparison of ISRS for Response of FWS Basemat in Vert. (Z) Direction**

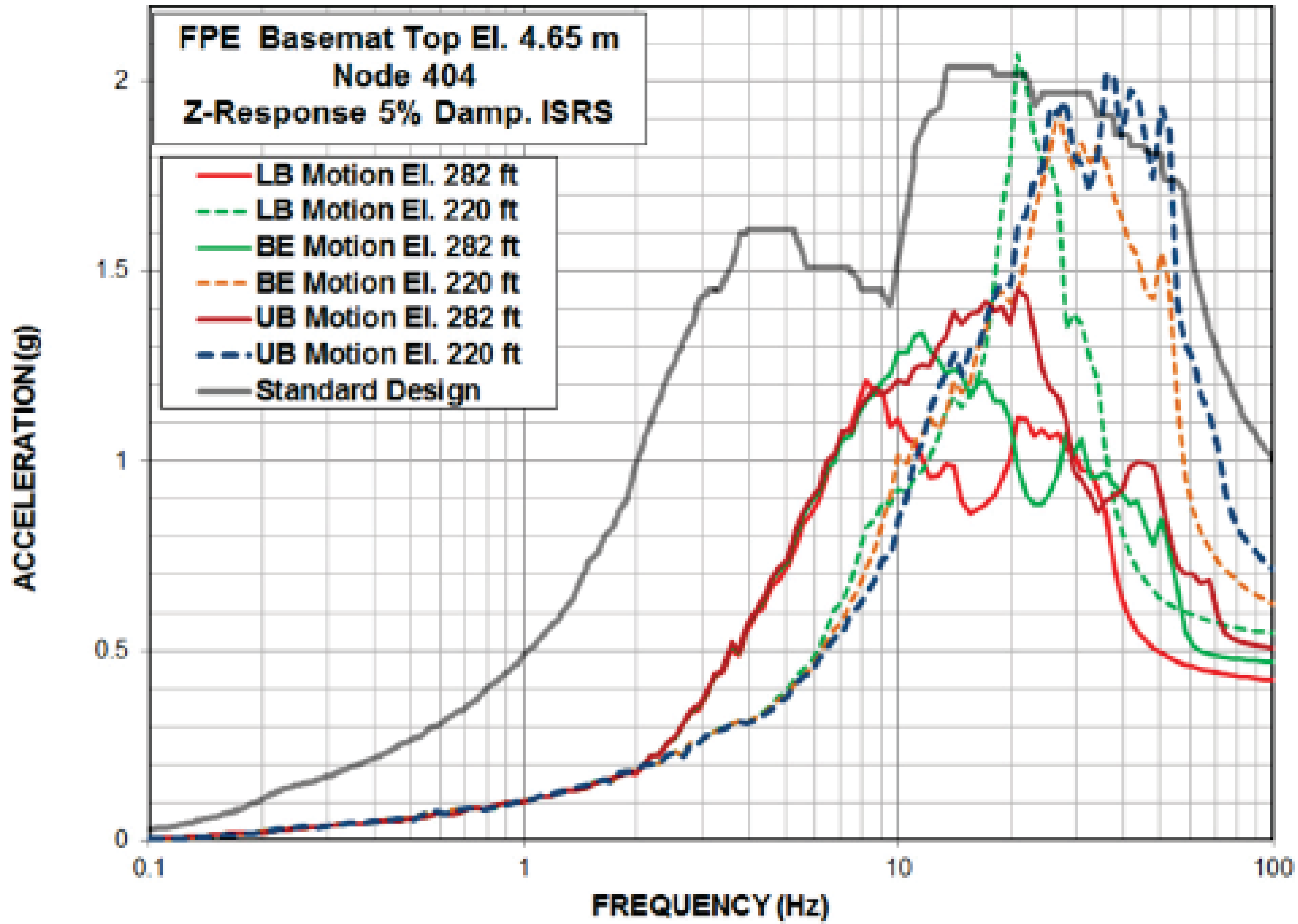


NAPS DEP 3.7-1 Figure 3A.17.14.3-203c **Comparison of ISRS for Response of FPE Top in Vert. (Z) Direction**



NAPS DEP 3.7-1

Figure 3A.17.14.3-203d Comparison of ISRS for Response of FPE Basemat in Vert. (Z) Direction



3A.17.14.4 Base Reactions and Contact Pressures

Results of the FWSC site-specific SSI analyses for contact spring forces are also used to calculate the time histories of the seismic driving forces used as input for the sliding stability evaluations and dynamic bearing pressure calculations. The time histories of the horizontal and vertical driving seismic forces and overturning base moments are calculated as described in Section 3A.17.12.5.

The time histories of the vertical spring forces obtained from the contact springs at the bottom of the FWSC basemat are also used to develop the base contact pressure plots that are used to calculate the minimum base contact area. These calculations are performed as described in Section 3A.17.12.5.

Figure 3A.17.14.4-201 presents the time histories of the eccentricities of vertical base reactions obtained from the SSI analyses of the FWSC UC_{SSE} model with full stiffness properties and SSE damping for the LB, BE, and UB profiles (analysis Cases 7 to 9 in Table 3A.15-203) that provide bounding results for maximum accelerations and member forces as discussed in Section 3A.17.14.2. The critical instances of time when the eccentricity of the vertical base reaction has maximum value are identified with red lines. Table 3A.17.14.4-201, which summarizes the calculations of maximum base reaction eccentricities the analysis of the BE profile (analysis Case 8 in Table 3A.15-203) yields the maximum value for the vertical base reaction eccentricity. Figure 3A.17.14.4-202 presents a contour plot of the SASSI2010 base contact pressure results for the analysis of the most critical UB profile at the critical instance of time t = 1.140 sec when the uplift of FWSC basemat is the largest. The positive values of the contact pressures in the figures represent tension of the base contact springs. The negative values represent compression of the base contact springs and are used to determine the portion of the basemat that is in contact with the underlying subgrade. Plots in Figure 3A.17.14.4-202 show that the contact area between the FWSC basemat and the underlying concrete fill remains greater than 85 percent. Therefore, the minimum base contact area is greater than 80 percent which, per the guidance of SRP 3.7.2, ensures that the possible uplift of the FWSC basemat has negligible effects on the FWSC seismic response and results of site-specific FWSC SSI analyses.

NAPS DEP 3.7-1

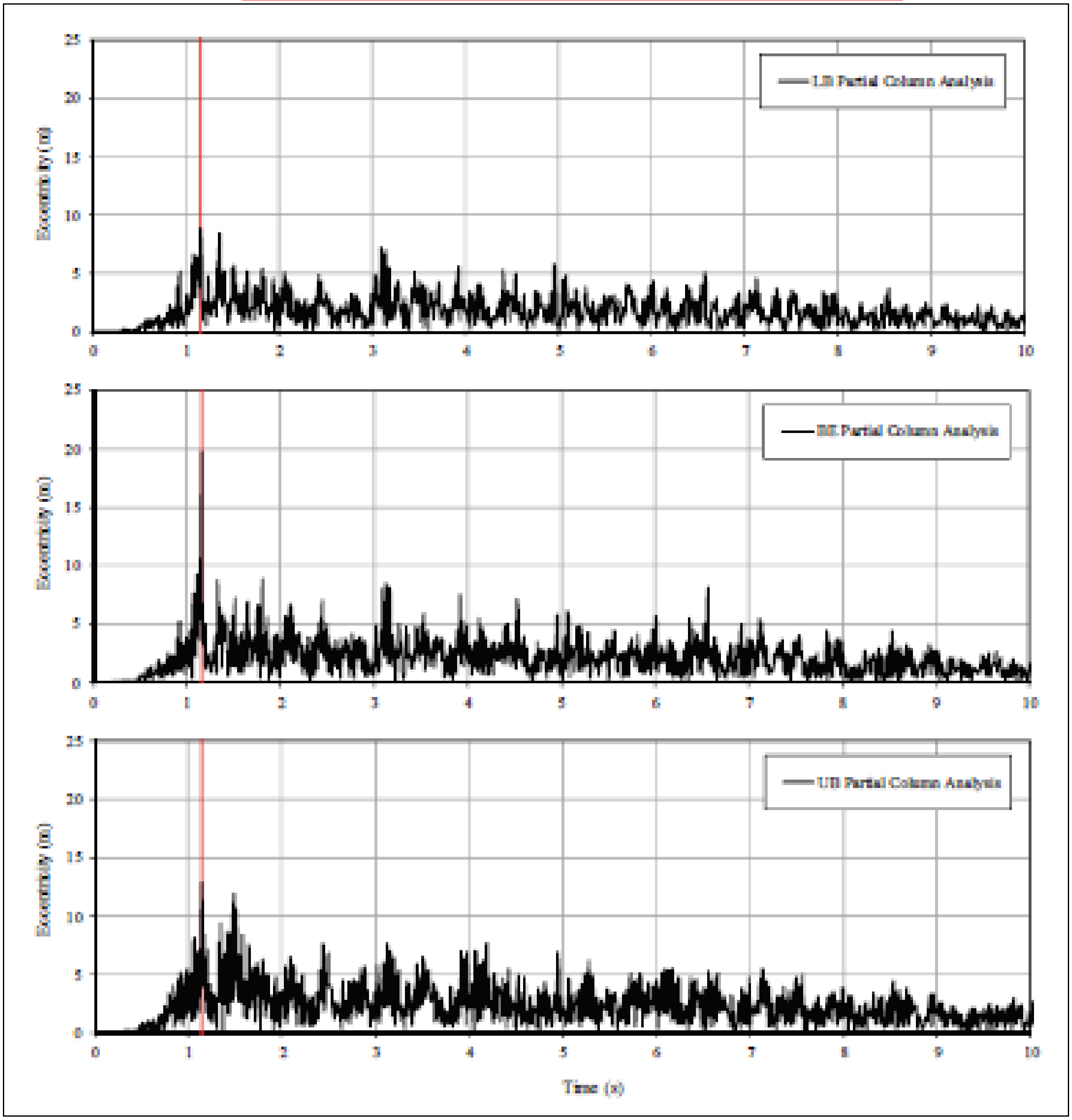
Table 3A.17.14.4-201 **Summary of Maximum Base Reaction Eccentricity Results - FWSC**

	UC_{SSE} Model Deep Input Motion at Elevation 220 ft NAVD88		
Analysis	**BE**	**UB**	**LB**
Max. Eccentricity (m)	19.7	12.9	8.8
at Time (s)	1.155	1.140	1.160

Note: The shaded values are the governing case.

NAPS DEP 3.7-1

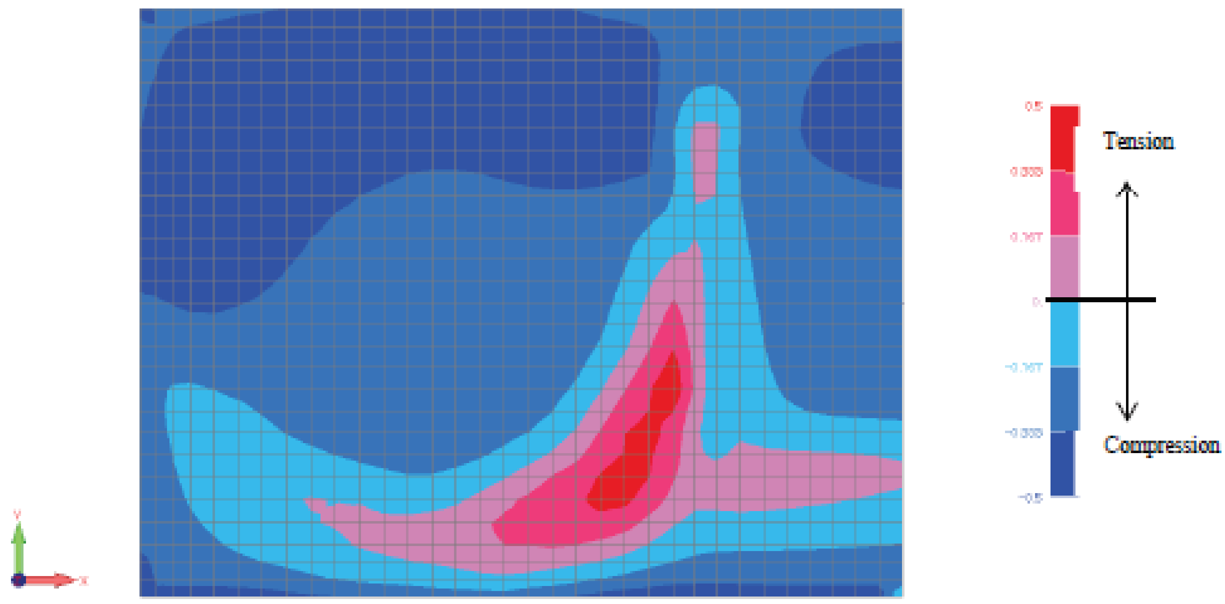
Figure 3A.17.14.4-201 **FWSC Base Reaction Eccentricity**



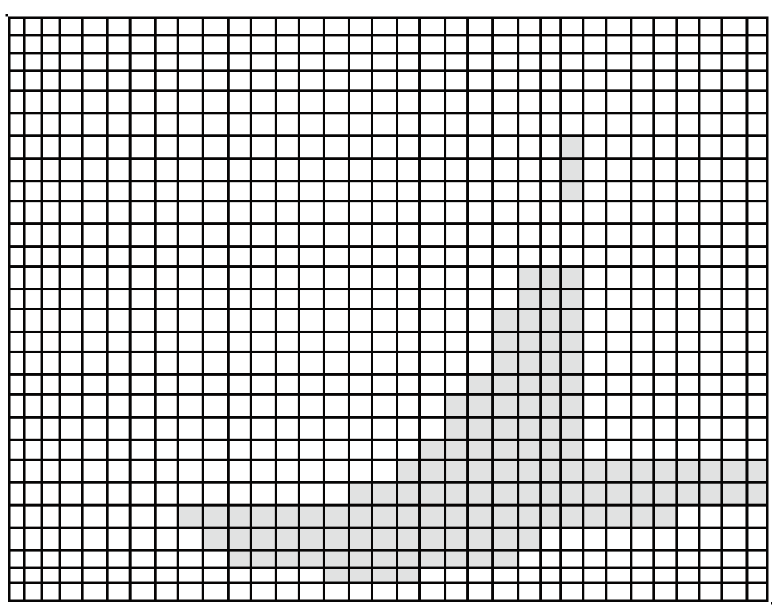
The critical instances of time when the eccentricity of the vertical base reaction has maximum value are identified with red lines.

NAPS DEP 3.7-1

Figure 3A.17.14.4-202 **FWSC Base Contact Area (UB Profile Analysis Case 9)**



(a) Base Contact Pressures



(b) Minimum Contact Area (85% at critical time t = 1.140 sec)

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3A.18 Unit 3 Site Envelope Seismic Responses

Site-specific enveloping seismic demands are established from the envelopes of the site-specific analyses results from SSI cases summarized in Tables 3A.15-201 through 3A.15-203 and SSSI analysis cases summarized in Table 3A.15-206. The site-specific enveloping seismic demands are compared to the enveloping seismic loads in DCD Section 3A.9 used for the standard design of Seismic Category I structures, as well as systems and components housed in the Seismic Category I structures.

3A.18.1 Unit 3 SSI Enveloping Maximum Structural Loads

Results from site-specific SSI analyses of the RB/FB, CB, and FWSC are enveloped and compared with the corresponding standard design envelopes in DCD Section 3A.9.1 to determine the exceedances relative to the seismic loads used for the standard design Seismic Category I structures.

Site-specific seismic load demands are developed for the site-specific structural evaluations of Seismic Category I structures following the methodology used to develop the standard design enveloping maximum structural loads. Horizontal load demands on the structures are developed from diagrams of the maximum enveloping shear forces and maximum enveloping torsional moments obtained as the envelope of the member force results from the different seismic response analyses. Vertical site-specific seismic load demands are developed from the diagrams of the maximum enveloping vertical floor mass accelerations. The maximum enveloping bending moments are also used for the structural evaluations to account for the effects of floor rocking on the wall axial forces.

The results for maximum enveloping vertical accelerations of the SDOF oscillator masses are used to develop the local out-of-plane load demands on flexible slabs and walls. The methodology used to calculate the site-specific out-of-plane loads on the flexible slabs and walls is identical to the methodology used for the standard design. Weighted average out-of-plane accelerations are calculated to represent the total site-specific seismic load demands on the flexible slabs and walls that include the contribution of both:

- The flexible modes of vibration represented by the maximum accelerations of the SDOF oscillators

- The rigid modes of vibration represented by the maximum accelerations of the floor lumped masses

3A.18.1.1 Unit 3 Reactor Building/Fuel Building SSI Enveloping Structural Load Demands

Tables 3A.18.1.1-201a through 3A.18.1.1-201f present the enveloping seismic shear forces and moments obtained as envelopes of the results from the site-specific SSI analyses of RB/FB for all six subsurface profiles and compare them with the corresponding standard design loads. The torsional moments in these tables are the calculated results from the SSI analyses and are combined with the accidental torsion in the structural evaluation. Tables 3A.18.1.1-202a through 3A.18.1.1-202e present the comparison of envelopes of site-specific maximum accelerations at floor lumped mass locations with the corresponding standard design enveloping accelerations. Comparisons of the diagrams of the site-specific horizontal and vertical load demands with the corresponding seismic loads used for the standard design are illustrated in Figures 3A.18.1.1-201a through 3A.18.1.1-202e. Figures 3A.18.1.1-201a through 3A.18.1.1-201e compare the shear force and torsion diagrams, which define the horizontal seismic load demands on the RB/FB structures. Figures 3A.18.1.1-202a through 3A.18.1.1-202e compare the maximum vertical accelerations and bending moment diagrams that define the vertical seismic load demands on the RB/FB structures.

Table 3A.18.1.1-203 presents the site-specific enveloping out-of-plane load demands on the RB/FB flexible slabs obtained as the envelope of the results of the SSI analyses of the RB/FB model with upper bound stiffness properties and OBE damping values (Cases 1 through 6 in Table 3A.15-201). Table 3A.18.1.1-204 presents the site-specific out-of-plane load demands on the RB/FB flexible walls enveloping the results obtained from Cases 1 through 6. Tables 3A.18.1.1-203 and 3A.18.1.1-204 compare the site-specific enveloping out-of-plane load demands on the RB/FB flexible slabs and walls with the magnitudes of the corresponding loads used for the standard design of RB/FB structures.

The weighted average out-of-plane accelerations presented in Tables 3A.18.1.1-203 and 3A.18.1.1-204 represent the site-specific seismic load demands on the flexible slabs and walls. These site-specific out-of-plane load demands are calculated using the SSI analyses results for the maximum accelerations of the floor lumped masses and the

maximum accelerations of SDOF oscillator masses presented in Tables 3A.17.12.2-201 and 3A.17.12.2-202.

The comparisons presented in Figures 3A.18.1.1-201a through 3A.18.1.1-202e and Tables 3A.18.1.1-201a through 3A.18.1.1-204 indicate that the enveloping site-specific load demands can exceed the corresponding loads used for the standard design of the RB/FB structures. The comparisons in Figures 3A.18.1.1-201a through 3A.18.1.1-201e and Tables 3A.18.1.1-201a through 3A.18.1.1-201f indicate that the Unit 3 site-specific horizontal load demands on the RB/FB walls are enveloped by the standard design with the exception of the NS shear load demands on the VW and RSW that exceed the corresponding standard design loads at elevations above 7.5 m. Comparisons in Figures 3A.18.1.1-202a through 3A.18.1.1-202e indicate larger exceedances in vertical loads at higher elevations above 8.0 m that can increase the axial stress demands and reduce the capacity of the reinforced concrete members. The most significant exceedances can be observed for the local out-of-plane loads on flexible walls and slabs at elevations of 17.5 m and above. The maximum exceedance of 58 percent is calculated for the out-of-plane vertical load on the main steam tunnel slab.

Site-specific evaluations of the RB/FB structures described in Appendix 3G are performed to address these exceedances in the site-specific load demands and calculate the available margins of RB/FB structures at the Unit 3 site. These site-specific evaluations use site-specific seismic loads that are developed based on the enveloping results obtained from the site-specific SSI analyses of the RB/FB and bound effects of structural stiffness variations as described in Section 3A.17.9.1.

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Table 3A.18.1.1-201a **Enveloping Maximum Structural Force and Moment Demands on RB/FB**

Element		Standard Design					NA3 Enveloping Demands					Difference/Exceedance				
Elev. (m)	Node No.	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS*)	Bending (MN-m) EW*)	Torsion (MN-m)	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS*)	Bending (MN-m) EW*)	Torsion (MN-m)	Shear NS	Shear EW	Bending NS*)	Bending EW*)	Torsion
52.40	110	151.9	158.2	1642	1808	1379	192.2	140.0	2724	2142	1284	27%	-12%	66%	18%	-7%
	109			4303	4465				5838	4488				36%	1%	
34.00	109	191.7	153.0	5585	5522	2405	173.2	113.9	8196	5821	1938	-10%	-26%	47%	5%	-19%
	108			6477	6317				8719	6389				35%	1%	
27.00	108	425.4	400.7	7685	7106	3333	396.0	259.4	9400	7162	2799	-7%	-35%	22%	1%	-16%
	107			8964	8596				9599	7958				7%	-7%	
22.50	107	483.7	464.0	9905	9193	6093	436.4	291.8	11216	8328	4678	-10%	-37%	13%	-9%	-23%
	106			11464	11297				11424	9227				0%	-18%	
17.5	106	532.9	555.4	12386	11935	5068	438.4	343.5	12105	9408	4023	-18%	-38%	-2%	-21%	-21%
	105			13778	13867				12349	10195				-10%	-26%	
13.57	105	569.2	599.9	14298	14377	5245	450.7	363.7	12839	10255	4211	-21%	-39%	-10%	-29%	-20%
	104			16593	16740				13651	11216				-18%	-33%	
9.06	104	610.1	654.3	16966	17191	5985	454.6	383.4	13904	11338	4694	-25%	-41%	-18%	-34%	-22%
	103			19378	19672				15231	12506				-21%	-36%	
4.65	103	839.8	872.2	19064	20192	11425	454.7	360.1	9392	6302	5248	-46%	-59%	-51%	-69%	-54%
	102			23163	24272				10952	7759				-53%	-68%	
-1.00	102	871.4	938.5	23673	24948	11523	240.0	226.6	6545	4819	2718	-72%	-76%	-72%	-81%	-76%
	101			27655	29263				7303	5358				-74%	-82%	
-6.40	101	933.6	1029.7	28126	30038	11690	237.7	200.4	4748	3351	2079	-75%	-81%	-83%	-89%	-82%
-11.5	2			32235	35275				5053	3356				-84%	-90%	

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-201b **Enveloping Maximum Structural Force and Moment Demands on RCCV**

Element		Standard Design					NA3 Enveloping Demands					Difference/Exceedance				
Elev. (m)	Node No.	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS*)	Bending (MN-m) EW*)	Torsion (MN-m)	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS*)	Bending (MN-m) EW*)	Torsion (MN-m)	Shear NS	Shear EW	Bending NS*)	Bending EW*)	Torsion
34.00	209	137.0	183.2	195	581	36	130.9	133.2	230	510	29	-4%	-27%	18%	-12%	-19%
	208			1057	1496				1029	1160				-3%	-22%	
27.00	208	164.9	248.5	1708	2532	1814	141.1	151.9	2162	2303	1489	-14%	-39%	27%	-9%	-18%
	207			2959	4368				2938	3071				-1%	-30%	
17.50	206	230.2	290.2	3315	4715	1982	184.1	158.4	3259	3667	1591	-20%	-45%	-2%	-22%	-20%
	205			4147	5761				3691	3904				-11%	-32%	
13.57	205	263.4	326.2	4327	5949	2186	207.9	173.4	3817	4203	1762	-21%	-47%	-12%	-29%	-19%
	204			5404	7264				4389	4491				-19%	-38%	
9.06	204	304.2	365.8	5628	7519	2616	225.4	201.2	4481	4853	2062	-26%	-45%	-20%	-35%	-21%
	203			6785	8909				5190	5203				-24%	-42%	
4.65	203	227.3	289.4	6992	9171	2870	109.2	125.7	5523	5470	1439	-52%	-57%	-21%	-40%	-50%
	202			7958	10581				5740	5824				-28%	-45%	
-1.00	202	272.4	330.6	8076	10738	2926	67.6	68.1	6008	6066	690	-75%	-79%	-26%	-44%	-76%
	201			9417	12523				5924	6035				-37%	-52%	
-6.40	201	261.7	303.5	9534	12651	1962	70.7	55.1	6053	6141	349	-73%	-82%	-37%	-51%	-82%
-11.50	2			10836	14200				5961	6127				-45%	-57%	

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-201c **Enveloping Maximum Structural Force and Moment Demands on Vent Wall**

Element		Standard Design					NA3 Enveloping Demands					Difference/Exceedance				
Elev. (m)	Node No.	Shear (MN)		Bending (MN-m)		Torsion (MN-m)	Shear (MN)		Bending (MN-m)		Torsion (MN-m)	Shear		Bending		Torsion
		NS	EW	NS*)	EW*)		NS	EW	NS*)	EW*)		NS	EW	NS*)	EW*)	
17.50	701	35.0	37.0	78	85	116	47.9	32.4	74	56	107	37%	-12%	-5%	-34%	-8%
	702			114	136				139	107				22%	-21%	
14.50	702	36.4	39.3	119	148	118	47.1	32.4	139	113	108	29%	-18%	17%	-24%	-8%
	703			226	260				279	204				23%	-22%	
11.50	703	37.0	41.8	229	269	120	45.8	35.1	280	207	111	24%	-16%	22%	-23%	-8%
	704			340	390				411	301				21%	-23%	
8.50	704	37.8	44.7	341	396	122	44.7	36.5	411	302	112	18%	-18%	21%	-24%	-8%
	705			379	438				458	338				21%	-23%	
7.4625	705	40.7	40.5	359	438	101	39.1	29.4	440	352	92	-4%	-27%	23%	-20%	-9%
4.65	706, 303			456	525				513	427				13%	-19%	

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-201d **Enveloping Maximum Structural Force and Moment Demands on Pedestal**

Element		Standard Design					NA3 Enveloping Demands					Difference/Exceedance				
		Shear (MN)		Bending (MN-m)			Shear (MN)		Bending (MN-m)			Shear		Bending		
Elev. (m)	Node No.	NS	EW	NS*)	EW*)	Torsion (MN-m)	NS	EW	NS*)	EW*)	Torsion (MN-m)	NS	EW	NS*)	EW*)	Torsion
4.65	303	32.8	44.8	581	621	142	20.5	16.9	667	496	71	-38%	-62%	15%	-20%	-50%
	377			599	667				651	502				9%	-25%	
2.42	377	48.1	66.3	732	817	172	30.9	25.4	793	614	86	-36%	-62%	8%	-25%	-50%
	302			778	922				754	631				-3%	-32%	
-1.00	302	65.6	81.4	839	959	146	22.1	15.7	691	571	34	-66%	-81%	-18%	-40%	-77%
	376			928	1050				658	555				-29%	-47%	
-2.75	376	66.0	81.7	928	1050	146	21.8	16.1	658	555	34	-67%	-80%	-29%	-47%	-77%
	301			1116	1330				594	524				-47%	-61%	
-6.40	301	104.4	121.2	1149	1346	118	29.8	22.4	555	518	21	-71%	-82%	-52%	-62%	-82%
-11.50	2			1655	1963				553	514				-67%	-74%	

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-201e **Enveloping Maximum Structural Force and Moment Demands on Reactor Shield Wall**

Element		Standard Design					NA3 Enveloping Demands					Difference/Exceedance				
		Shear (MN)		Bending (MN-m)		Torsion	Shear (MN)		Bending (MN-m)		Torsion	Shear		Bending		
Elev. (m)	Node No.	NS	EW	NS*)	EW*)	(MN-m)	NS	EW	NS*)	EW*)	(MN-m)	NS	EW	NS*)	EW*)	Torsion
24.18	707	3.0	2.7	2.1	1.7	0.4	4.2	3.0	3	2	0.5	40%	11%	43%	18%	25%
	708			13.2	12.4				19	14				44%	13%	
20.20	708	14.6	12.3	18.4	16.8	1.4	20.8	10.4	26	20	1.7	42%	-15%	41%	19%	21%
	709			79.0	68.4				113	59				43%	-14%	
15.775	709	17.3	14.4	81.9	71.0	1.9	24.4	11.9	117	61	2.4	41%	-17%	43%	-14%	26%
	710			158.4	133.6				224	114				41%	-15%	
11.35	710	19.9	16.6	159.1	136.4	2.4	27.1	12.7	228	116	3.0	36%	-23%	43%	-15%	25%
	711			236.2	198.7				332	166				41%	-16%	
7.4625	711	41.1	35.6	197.0	183.6	23.4	22.4	16.8	99	78	21.4	-45%	-53%	-50%	-58%	-9%
	712			292.4	251.3				141	119				-52%	-53%	
4.65	712	14.3	19.5	125.1	133.0	30.3	8.9	7.4	148	108	15.2	-38%	-62%	18%	-19%	-50%
	713			133.0	150.9				140	111				5%	-26%	
2.4615	713	1.5	1.3	3.6	3.2	0.2	1.4	1.1	4	3	0.1	-7%	-15%	11%	-6%	-50%
1.96	714			2.9	2.7				3	2				3%	-26%	
1.96	714	0.9	0.7	2.7	2.4	0.1	0.9	0.7	3	2	0.1	0%	0%	11%	-17%	0%
-0.8	715			0.5	0.5				1	1				100%	100%	

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-201f **Enveloping Maximum Load Demands on RPV**

Component	Node No.	Elem. No.	Standard Design Axial (MN)	Standard Design Shear (MN) NS	Standard Design Shear (MN) EW	Standard Design Moment (MN-m) NS*)	Standard Design Moment (MN-m) EW*)	NA3 Enveloping Demands Axial (MN)	NA3 Enveloping Demands Shear (MN) NS	NA3 Enveloping Demands Shear (MN) EW	NA3 Enveloping Demands Moment (MN-m) NS*)	NA3 Enveloping Demands Moment (MN-m) EW*)	Difference Axial	Difference Shear NS	Difference Shear EW	Difference Moment NS*)	Difference Moment EW*)
Shroud Bottom	845	844	8.58	7.2	7.0	16.2	14.3	9.10	18.6	6.7	29.9	10.3	6%	158%	-4%	85%	-28%
	846					21.3	17.3				44.5	14.8				109%	-14%
Fuel Top	847	845	0.05	1.2	1.1	0.0	0.0	0.05	0.8	0.8	0.0	0.0	0%	-33%	-27%	-	-
	848					0.1	0.1				0.1	0.1				0%	0%
Fuel Middle	848	846	0.44	1.0	1.0	0.1	0.1	0.46	1.0	0.8	0.1	0.1	5%	0%	-20%	0%	0%
	849					0.8	0.8				0.8	0.6				0%	-25%
Fuel Middle	849	847	1.15	0.6	0.6	0.8	0.8	1.20	0.6	0.5	0.8	0.6	4%	0%	-17%	0%	-25%
	850					1.2	1.3				1.2	0.9				0%	-31%
Fuel Middle	850	848	1.86	0.2	0.2	1.2	1.3	1.93	0.4	0.3	1.2	0.9	4%	100%	50%	0%	-31%
	851					1.2	1.3				1.1	0.9				-8%	-31%
Fuel Middle	851	849	2.56	0.6	0.6	1.2	1.3	2.63	0.7	0.5	1.1	0.9	3%	17%	-17%	-8%	-31%
	852					0.8	0.8				0.7	0.7				-13%	-13%
Fuel Middle	852	850	3.24	1.0	1.0	0.8	0.8	3.31	0.9	0.8	0.7	0.7	2%	-10%	-20%	-13%	-13%
	853					0.1	0.1				0.1	0.1				0%	0%
Fuel Bottom	853	851	3.56	1.1	1.1	0.1	0.1	3.64	1.0	0.9	0.1	0.1	2%	-9%	-18%	0%	0%
	854					0.0	0.0				0.0	0.0				-	-
RPV Support	815	871	25.3	18.6	17.9	144	136	23.3	29.8	16.8	183	113	-8%	60%	-6%	27%	-17%
	711					141	137				177	109				26%	-20%

Note: Shaded values in the table show an exceedance from standard design.

*) NS and EW represent moments for bending in NS or EW direction, respectively.

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Table 3A.18.1.1-202a **Enveloping Maximum Accelerations for RB/FB**

Elev. (m)	Node No.	Acceleration (g)						Difference		
		Standard Design			NA3 Enveloping					
		NS	EW	Vert.	NS	EW	Vert.	NS	EW	Vert.
52.40	110	1.68	1.78	1.27	2.13	1.55	1.56	27%	-13%	23%
34.00	109	1.18	1.15	0.83	1.02	0.81	1.20	-14%	-30%	45%
27.00	108	0.99	1.02	0.73	0.96	0.69	1.02	-3%	-32%	40%
22.50	107	0.98	0.91	0.73	0.83	0.73	0.92	-15%	-20%	26%
17.50	106	0.98	0.84	0.73	0.80	0.65	0.80	-18%	-23%	10%
13.57	105	0.97	0.77	0.74	0.79	0.62	0.72	-19%	-19%	-3%
9.06	104	0.84	0.73	0.73	0.76	0.54	0.62	-10%	-26%	-15%
4.65	103	0.73	0.68	0.78	0.76	0.56	0.56	4%	-18%	-28%
-1.00	102	0.68	0.63	0.76	0.62	0.51	0.57	-9%	-19%	-25%
-6.40	101	0.61	0.62	0.68	0.50	0.43	0.50	-18%	-31%	-26%
-11.50	2	0.60	0.55	0.63	0.43	0.37	0.47	-28%	-33%	-25%
-15.50	1	0.51	0.51	0.51	0.44	0.37	0.46	-14%	-27%	-10%

Note: Values presented are the maximum accelerations at floor lumped mass locations.
Shaded values in the table show an exceedance from standard design.

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Table 3A.18.1.1-202b **Enveloping Maximum Accelerations for RCCV**

Elev. (m)	Node No.	Acceleration (g)						Difference		
		Standard Design			NA3 Enveloping					
		NS	EW	Vert.	NS	EW	Vert.	NS	EW	Vert.
34.00	209	1.18	1.15	0.90	1.02	0.81	1.20	-14%	-30%	33%
27.00	208	0.98	1.02	0.88	0.96	0.69	1.12	-2%	-32%	27%
17.50	206	0.98	0.85	0.73	0.80	0.66	0.91	-18%	-22%	25%
13.57	205	0.97	0.78	0.78	0.79	0.63	0.82	-19%	-19%	5%
9.06	204	0.84	0.74	0.65	0.76	0.54	0.72	-10%	-27%	11%
4.65	203	0.73	0.69	0.70	0.76	0.55	0.65	4%	-20%	-7%
-1.00	202	0.68	0.63	0.59	0.59	0.52	0.58	-13%	-17%	-2%
-6.40	201	0.61	0.60	0.59	0.50	0.44	0.50	-18%	-27%	-15%

Note: Values presented are the maximum accelerations at floor lumped mass locations.
Shaded values in the table show an exceedance from standard design.

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Table 3A.18.1.1-202c Enveloping Maximum Accelerations for Vent Wall and Pedestal

Elev. (m)	Node No.	Acceleration (g)						Difference		
		Standard Design			NA3 Enveloping					
		NS	EW	Vert.	NS	EW	Vert.	NS	EW	Vert.
17.50	701	0.98	0.85	1.10	0.80	0.66	0.82	-18%	-22%	-25%
14.50	702	1.40	1.06	1.04	0.77	0.61	0.77	-45%	-42%	-26%
11.50	703	1.42	1.14	0.92	0.77	0.55	0.71	-46%	-52%	-23%
8.50	704	0.98	0.86	0.77	0.74	0.53	0.68	-24%	-38%	-12%
7.4625	705	0.85	0.77	0.70	0.75	0.52	0.67	-12%	-32%	-4%
4.65	706, 303	0.73	0.69	0.67	0.76	0.55	0.65	4%	-20%	-3%
-1.00	302	0.68	0.63	0.59	0.59	0.52	0.59	-13%	-17%	0%
-6.40	301	0.61	0.60	0.50	0.50	0.44	0.49	-18%	-27%	-2%

Note: Values presented are the maximum accelerations at mass center.
Shaded values in the table show an exceedance from standard design.

NAPS DEP 3.7-1

Table 3A.18.1.1-202d **Enveloping Maximum Accelerations for Reactor Shield Wall**

Elev. (m)	Node No.	Acceleration (g)						Difference		
		Standard Design			NA3 Enveloping					
		NS	EW	Vert.	NS	EW	Vert.	NS	EW	Vert.
24.18	707	2.51	2.38	0.97	3.61	2.51	1.19	44%	5%	23%
20.20	708	2.13	1.90	0.94	2.81	1.82	1.14	32%	-4%	21%
15.775	709	1.74	1.39	0.84	1.77	1.23	0.99	2%	-12%	18%
11.35	710	1.29	0.95	0.76	1.01	0.69	0.78	-22%	-27%	3%
7.4625	711	0.85	0.77	0.70	0.75	0.52	0.67	-12%	-32%	-4%
4.65	712	0.73	0.69	0.67	0.76	0.55	0.65	4%	-20%	-3%
2.4615	713	0.74	0.66	0.64	0.69	0.55	0.64	-7%	-17%	0%
1.96	714	0.75	0.66	0.64	0.70	0.57	0.64	-7%	-14%	0%
-0.80	715	0.86	0.72	0.65	0.83	0.65	0.64	-3%	-10%	-2%

Note: Values presented are the maximum accelerations at mass center.
Shaded values in the table show an exceedance from standard design.

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Table 3A.18.1.1-202e **Enveloping Maximum Accelerations for Fuel**

Elev. (m)	Node No.	Standard Design			NA3 Enveloping Demands			Difference		
		NS (g)	EW (g)	Vert. (g)	NS (g)	EW (g)	Vert. (g)	NS	EW	Vert.
7.896	847	1.09	1.00	1.41	2.06	0.92	1.49	89%	-8%	6%
7.8071	848	1.06	0.98	1.41	1.91	0.86	1.49	80%	-12%	6%
7.111	849	0.91	0.92	1.41	1.00	0.49	1.47	10%	-47%	4%
6.401	850	0.98	0.91	1.39	0.62	0.64	1.45	-37%	-30%	4%
5.691	851	0.91	0.98	1.36	0.85	0.74	1.41	-7%	-24%	4%
4.981	852	1.20	1.12	1.33	1.61	1.15	1.36	34%	3%	2%
4.2713	853	1.65	1.42	1.28	2.52	1.74	1.31	53%	23%	2%
4.1784	854	1.73	1.52	1.27	2.63	1.81	1.30	52%	19%	2%

Note: Values presented are the maximum accelerations at mass center.
Shaded values in the table show an exceedance from standard design.

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Table 3A.18.1.1-203 **Enveloping Maximum Out-of-Plane Loads on RB/FB Flexible Slabs**

Elev. (m)	Location	Slab Equivalent Out-of-Plane Acceleration Load (g)								Difference
		Partial Column			Full Column			NA3 Envelope	Standard Design[1]	
		BE	UB	LB	BE	UB	LB			
52.40	RB Roof	1.07	1.23	0.87	1.35	*1.51*	1.07	1.51	1.64	-8%
34.00	RB-RCCV	0.83	0.98	0.71	1.01	*1.23*	0.84	1.23	0.90	37%
	RCCV	0.87	1.05	0.74	1.01	*1.30*	0.84	1.30	0.93	40%
27.00	Top Slab	0.89	1.09	0.77	1.14	*1.37*	0.91	1.37	0.98	40%
	RB-RCCV	0.70	0.81	0.59	0.86	*1.06*	0.69	1.06	0.77	38%
	M/S tunnel roof	0.73	0.88	0.62	0.90	*1.13*	0.69	1.13	0.82	38%
22.50	FB Roof	0.79	0.86	0.74	1.24	*1.31*	1.17	1.31	1.47	-11%
17.50	M/S tunnel slab	1.14	1.47	0.88	1.38	*1.74*	0.99	1.74	1.10	58%
	RB-RCCV	0.64	0.76	0.55	0.77	*0.94*	0.62	0.94	0.78	21%
	DF	1.07	1.17	0.93	1.38	*1.53*	1.14	1.53	1.84	-17%
13.57	RB-RCCV	0.63	0.78	0.54	0.72	*0.89*	0.59	0.89	0.84	6%
9.06	RB-RCCV	0.59	0.69	0.53	0.67	*0.79*	0.61	0.79	0.82	-4%
4.65	FB	0.91	1.10	0.72	0.92	*1.11*	0.60	1.11	1.03	8%
	RB-RCCV	0.70	0.81	0.64	0.81	*0.87*	0.72	0.87	0.95	-8%
	RCCV-Pedestal	0.63	0.71	0.55	0.70	*0.82*	0.61	0.82	0.80	2%
-1.00	FB	0.61	*0.71*	0.51	0.60	*0.71*	0.52	0.71	0.88	-19%
	RB-RCCV	0.59	0.70	0.52	0.64	*0.73*	0.55	0.73	0.85	-14%
	RCCV-Pedestal	0.57	0.65	0.46	0.61	*0.70*	0.55	0.70	0.71	-1%
-6.40	RCCV-Pedestal	0.48	0.55	0.41	0.54	*0.60*	0.51	0.60	0.63	-5%
	RB-RCCV	0.46	0.56	0.38	0.50	*0.57*	0.46	0.57	0.71	-20%

Note:
1) ESBWR standard design values.
Shaded values in the table show an exceedance from standard design.
Values shown in Italics are the governing cases.

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Table 3A.18.1.1-204 **Enveloping Maximum Out-of-Plane Loads on RB/FB Flexible Walls**

		Wall Equivalent Out-of-Plane Acceleration Load (g)								
		Partial Column			Full Column					
Elev. (m)	Location	BE	UB	LB	BE	UB	LB	NA3 Envelop.	Stand. Design[1)]	Difference
42.00	R1 and R7 walls	1.56	1.80	1.23	1.88	*2.10*	1.47	2.10	1.48	42%
	RB and RF walls	1.02	1.14	0.84	1.07	*1.27*	1.02	1.27	1.52	-16%
13.75	F3 wall	1.16	1.39	0.98	1.15	*1.48*	0.93	1.48	1.19	24%
	FA and FF walls	1.25	*1.55*	0.98	1.08	1.21	0.80	1.55	1.09	42%

Note:
1) ESBWR standard design values.
Shaded values in the table show an exceedance from standard design.
Values shown in Italics are the governing cases.

NAPS DEP 3.7-1

Figure 3A.18.1.1-201a **Site-Specific Horizontal Seismic Load Demands on RB/FB**

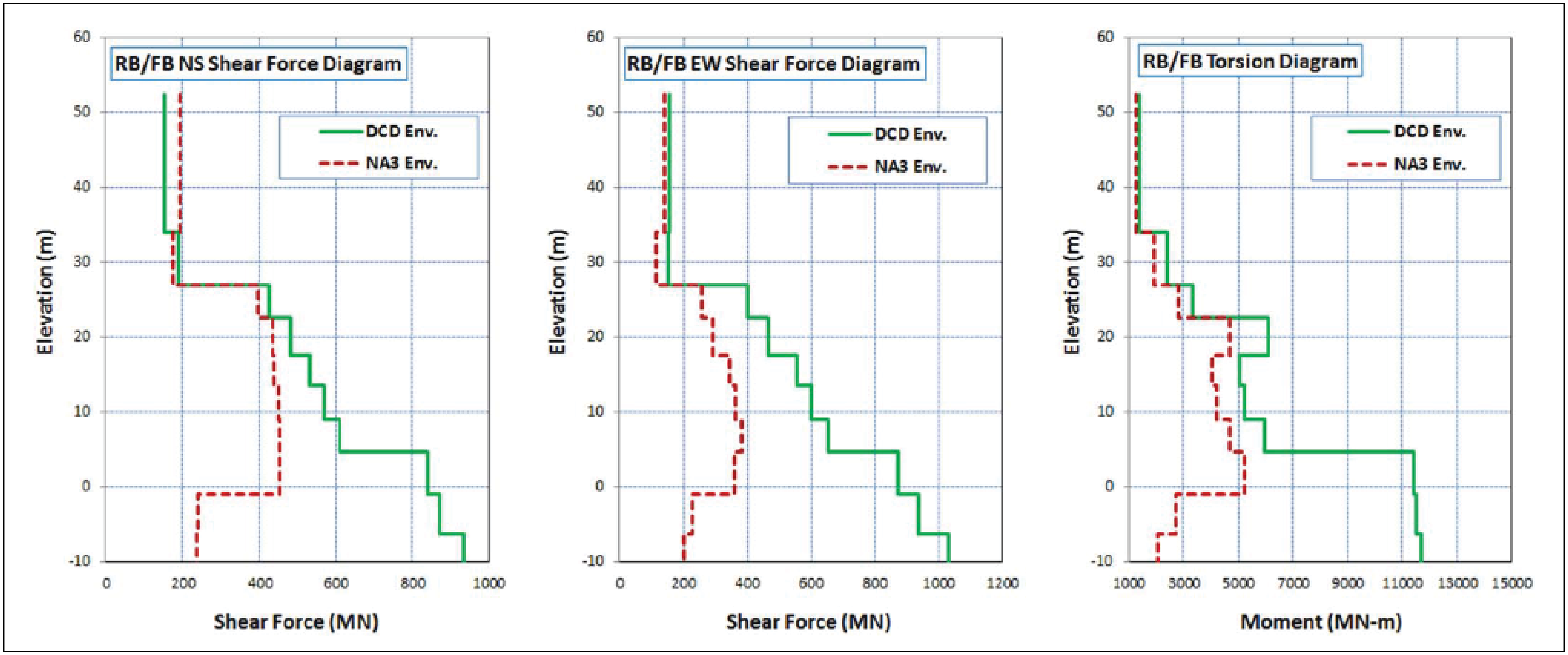

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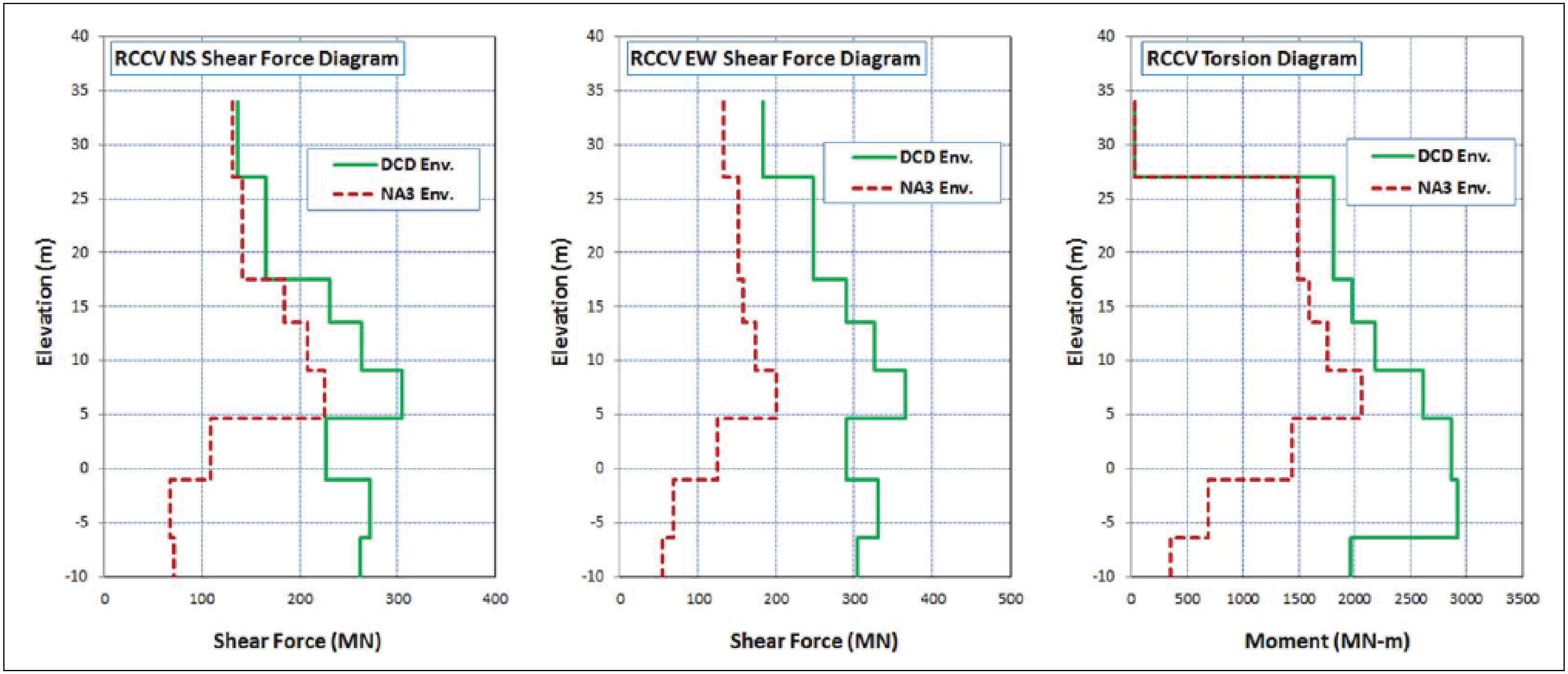
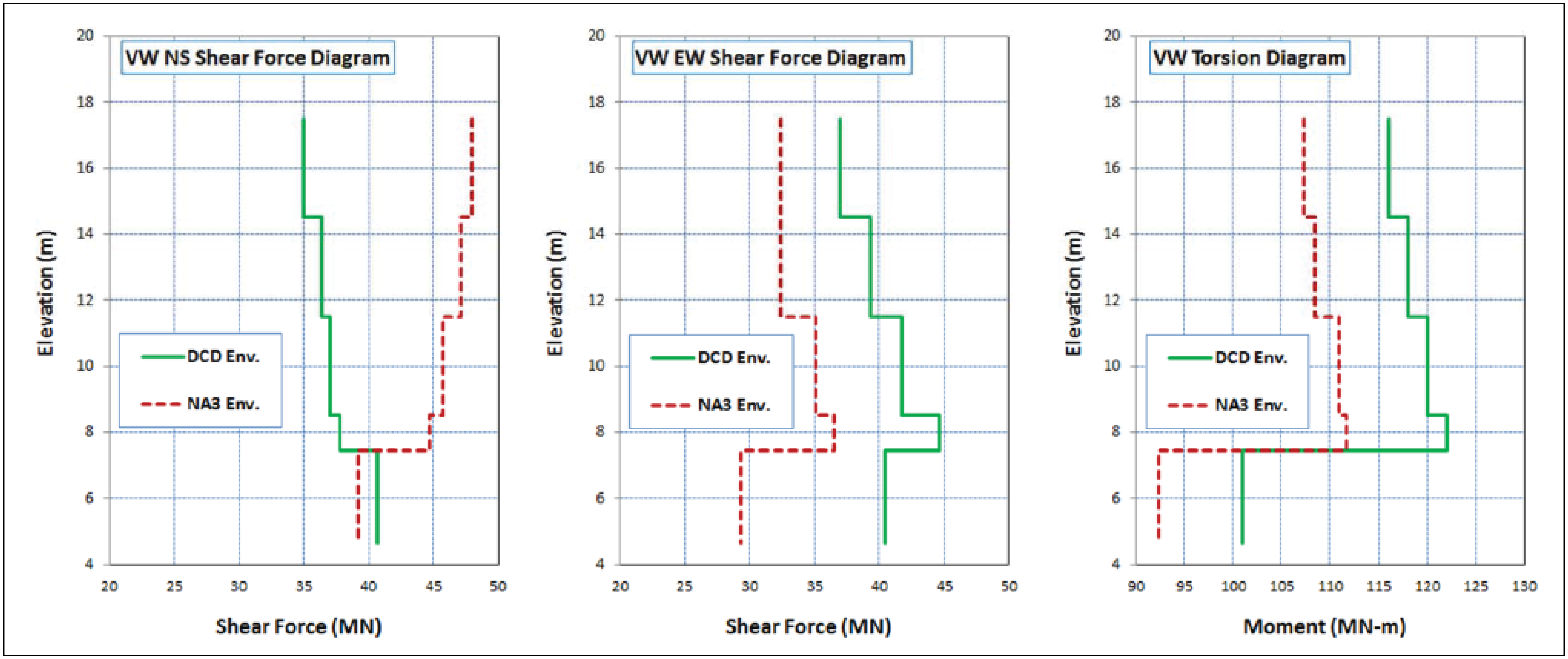
Figure 3A.18.1.1-201b Site-Specific Horizontal Seismic Load Demands on RCCV



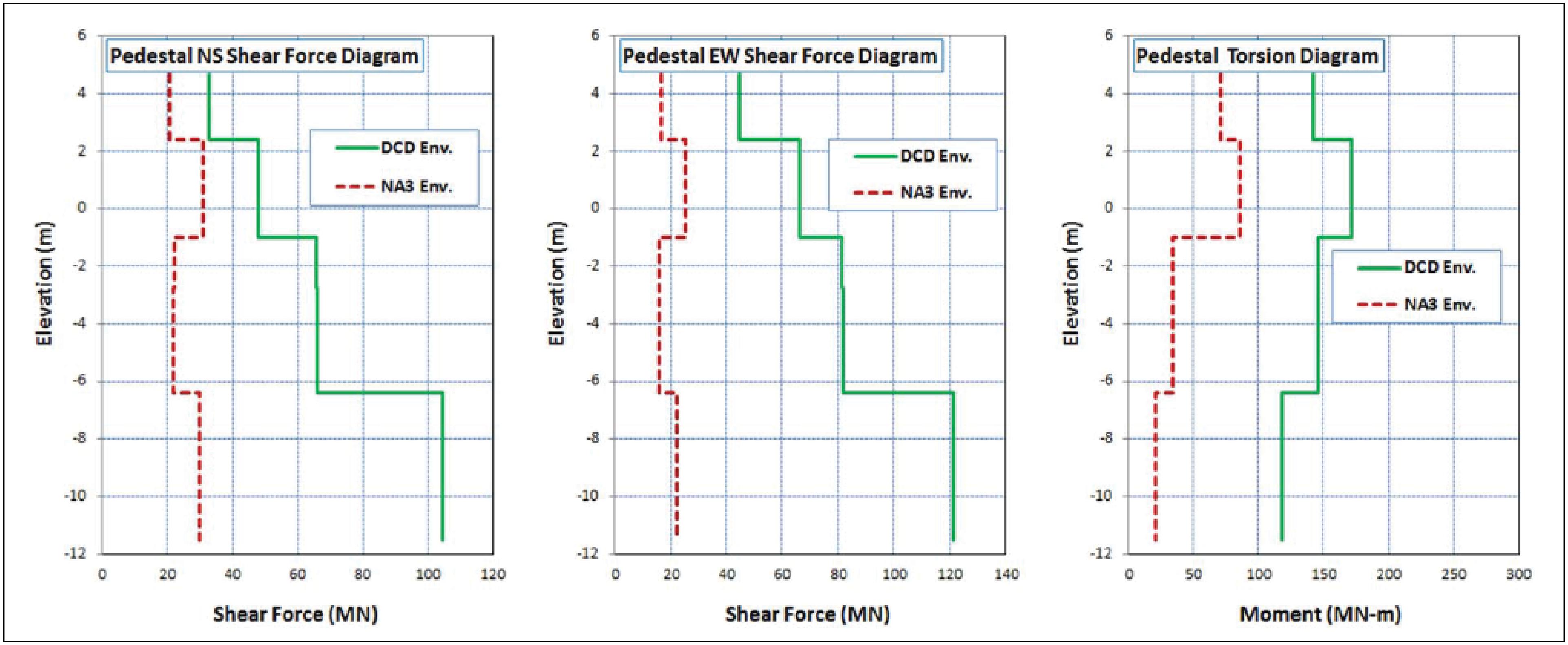
NAPS DEP 3.7-1

Figure 3A.18.1.1-201c Site-Specific Horizontal Seismic Load Demands on Vent Wall



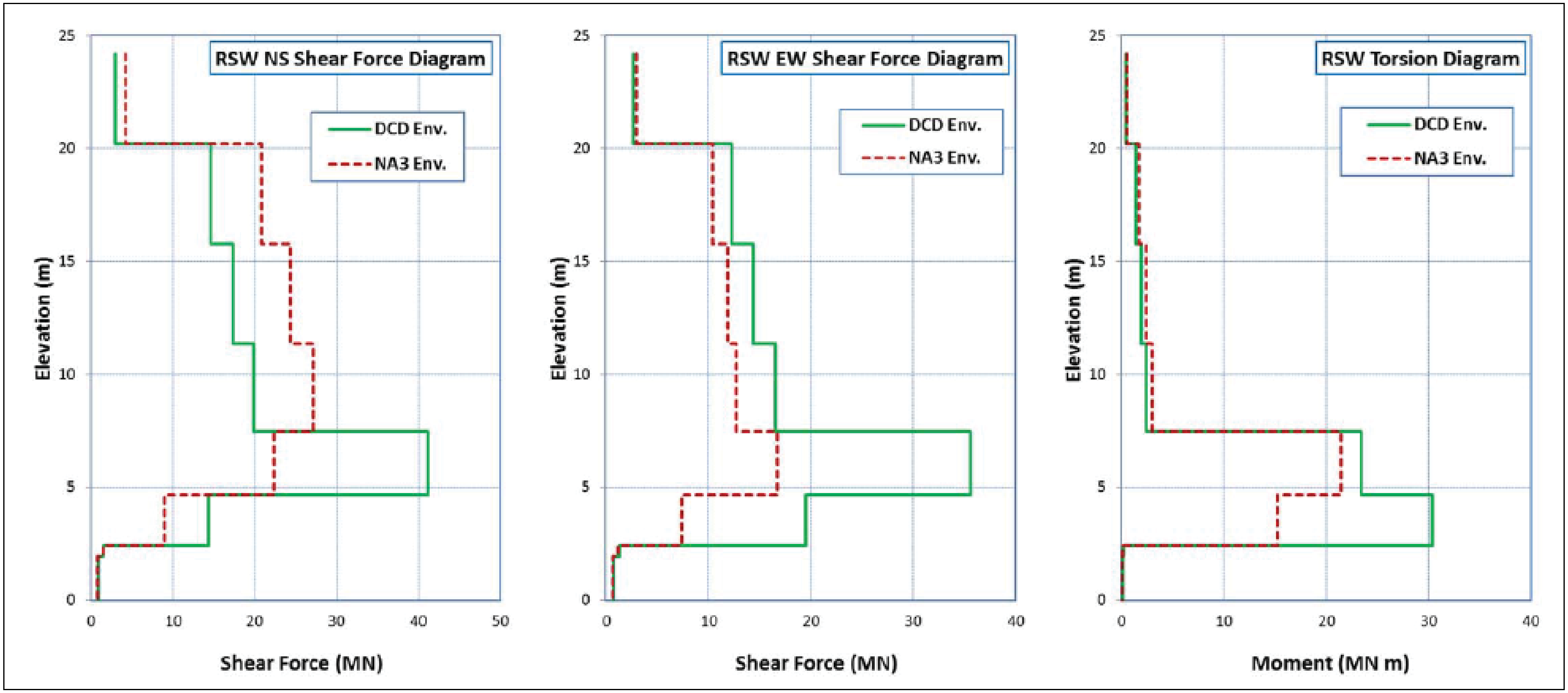
NAPS DEP 3.7-1

Figure 3A.18.1.1-201d **Site-Specific Horizontal Seismic Load Demands on Pedestal**



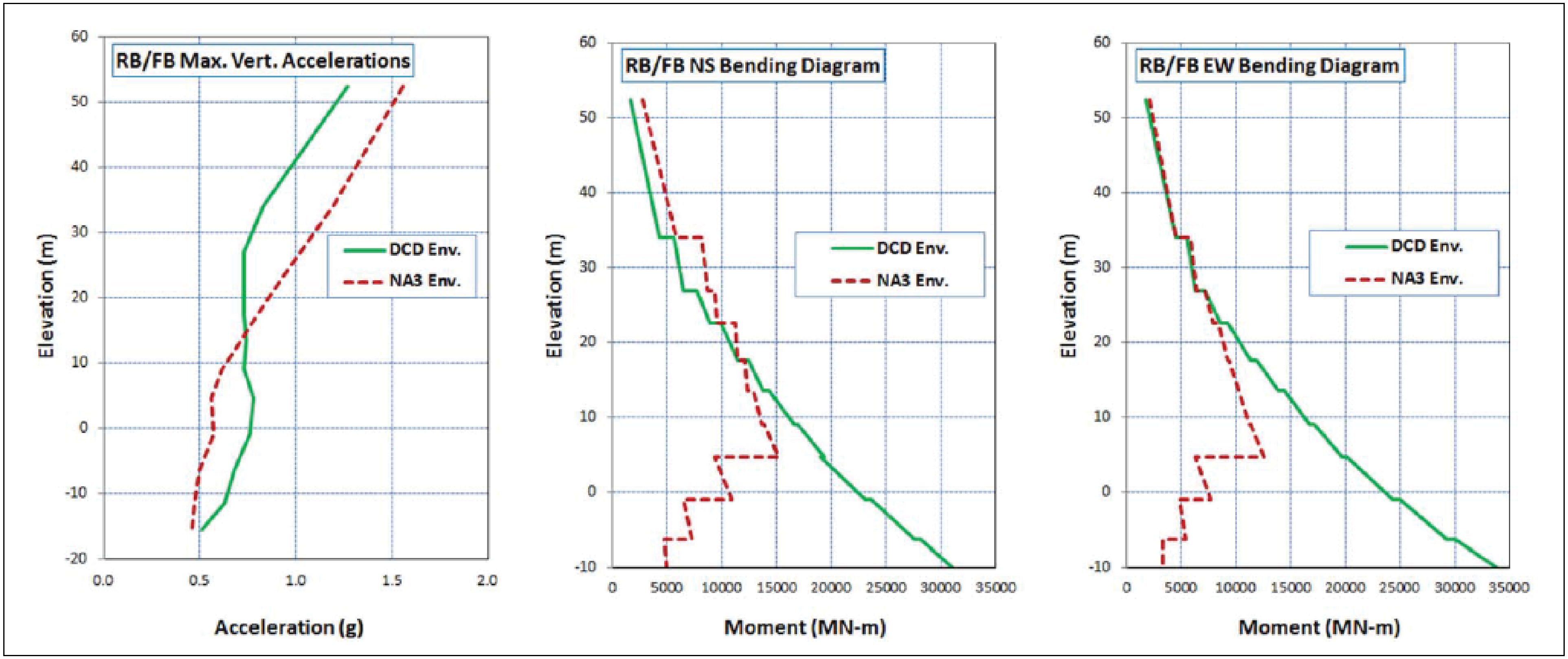
NAPS DEP 3.7-1

Figure 3A.18.1.1-201e **Site-Specific Horizontal Seismic Load Demands on Reactor Shield Wall**



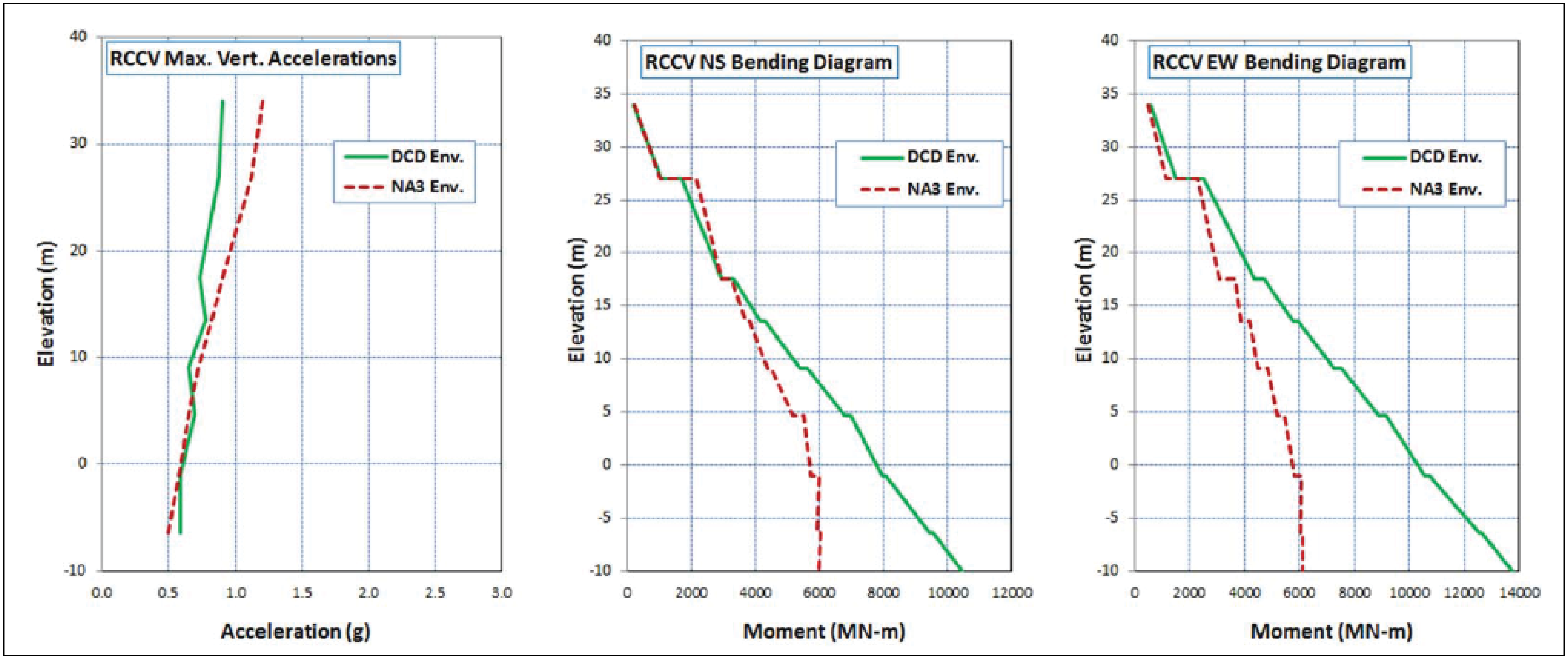
NAPS DEP 3.7-1

Figure 3A.18.1.1-202a **Site-Specific Vertical Seismic Load Demands on RB/FB**



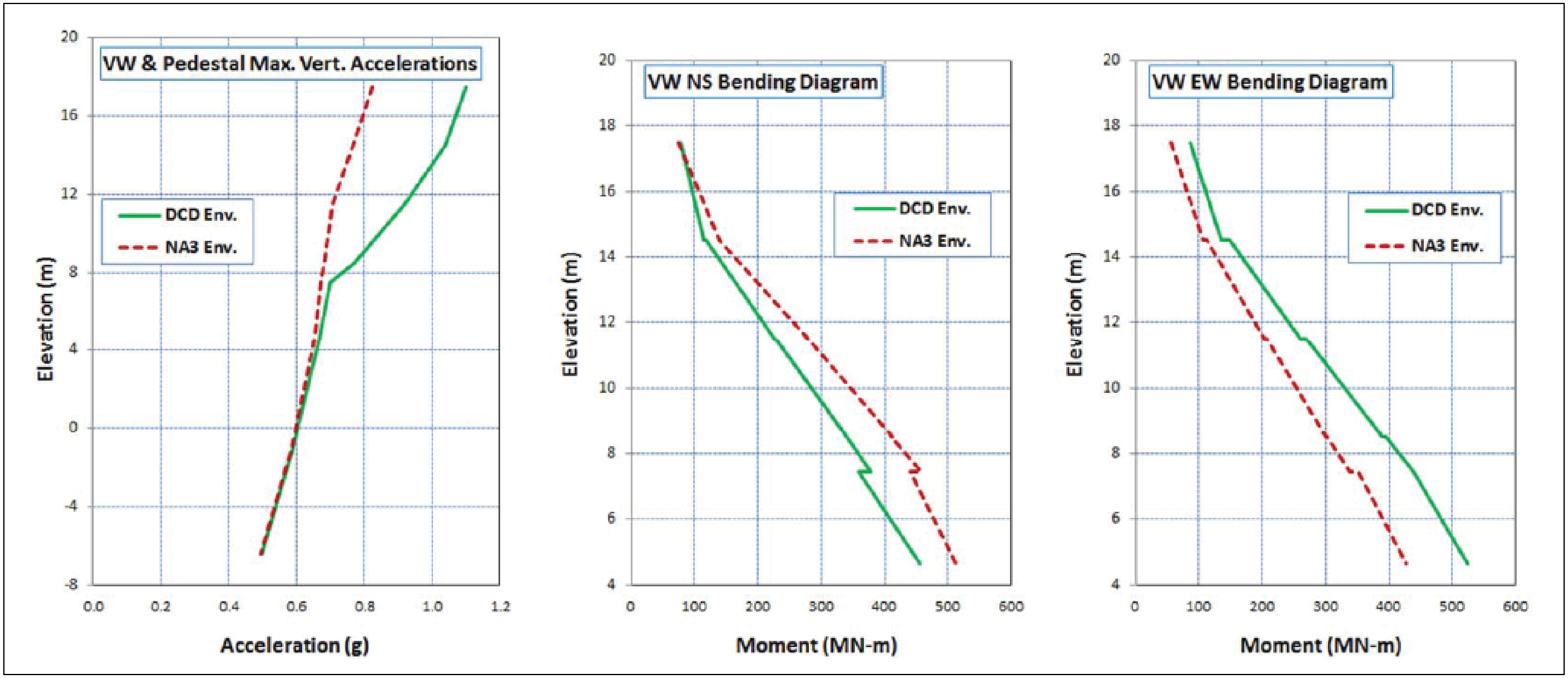
NAPS DEP 3.7-1

Figure 3A.18.1.1-202b Site-Specific Vertical Seismic Load Demands on RCCV



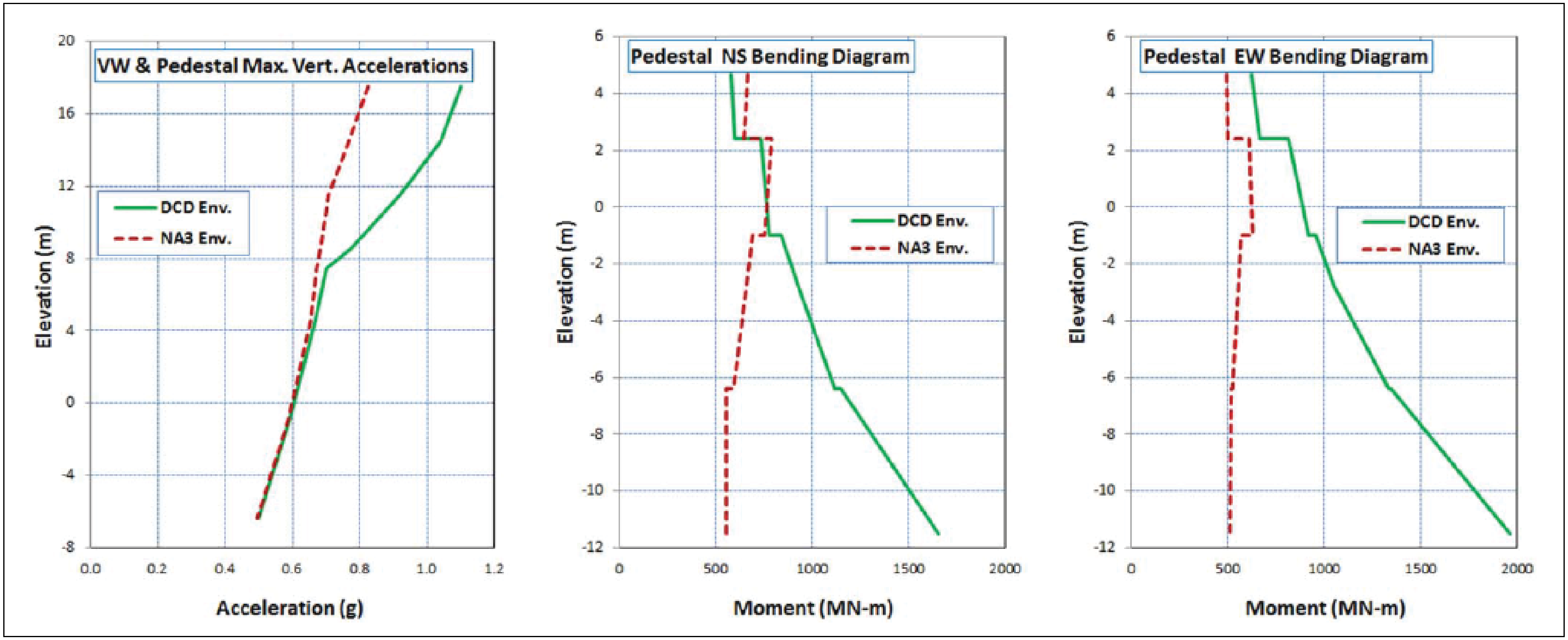
NAPS DEP 3.7-1

Figure 3A.18.1.1-202c **Site-Specific Vertical Seismic Load Demands on Vent Wall**



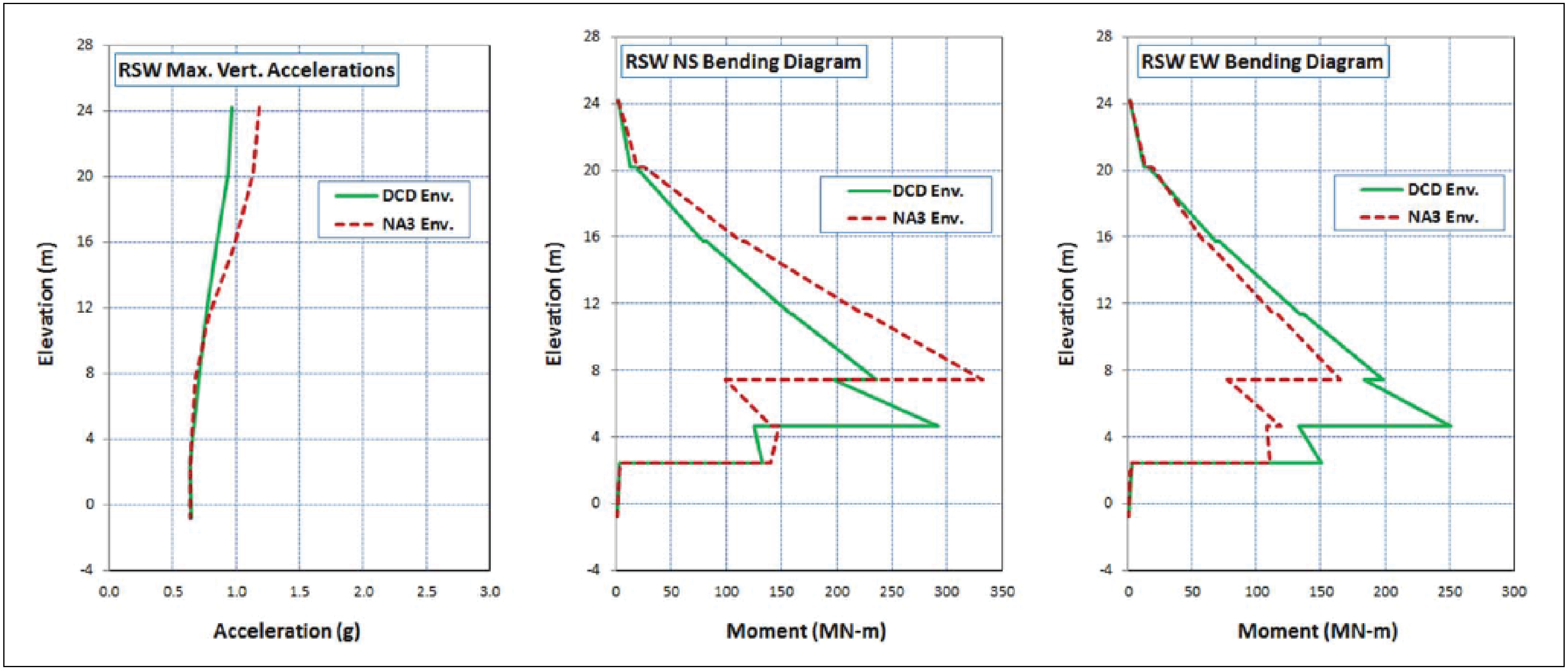
NAPS DEP 3.7-1

Figure 3A.18.1.1-202d **Site-Specific Vertical Seismic Load Demands on Pedestal**



NAPS DEP 3.7-1

Figure 3A.18.1.1-202e **Site-Specific Vertical Seismic Load Demands on Reactor Shield Wall**



3A.18.1.2 **Unit 3 CB SSI Enveloping Structural Load Demands**

Table 3A.18.1.2-201 presents the seismic shear forces and moments obtained as the envelope of the results from the site-specific SSI analyses of the CB model with full stiffness properties and SSE damping for all six subsurface profiles and compares them with the corresponding standard design loads. Table 3A.18.1.2-202 compares the envelopes of site-specific maximum accelerations at floor lumped mass locations with the corresponding standard design enveloping accelerations. Site-specific horizontal and vertical load demands are compared to the corresponding seismic loads used for standard design in Figures 3A.18.1.2-201 and 3A.18.1.2-202. Figure 3A.18.1.2-201 compares the shear force and torsion diagrams that define the horizontal seismic load demands on the CB structures. Figure 3A.18.1.2-202 compares the maximum vertical accelerations and bending moment diagrams that define the vertical seismic load demands on the CB structures.

Table 3A.18.1.2-203 presents the site-specific enveloping out-of-plane load demands on the CB flexible slabs obtained as the envelope of the results of the SSI analysis of the CB model with the upper bound stiffness properties and SSE damping values (Cases 7 through 12 in Table 3A.15-202). The site-specific enveloping out-of-plane load demands on the CB flexible slabs in Table 3A.18.1.2-203 are compared with the magnitudes of the corresponding loads used for the standard design of CB structures. Site-specific out-of-plane load demands on flexible slabs are calculated using the SSI analysis results of the analyses of the CB model with full stiffness properties and SSE damping for the maximum accelerations of the floor lumped masses and the maximum accelerations of SDOF oscillator masses presented in Table 3A.17.13.2-201.

Comparisons presented in Figures 3A.18.1.2-201 and 3A.18.1.2-202 and Tables 3A.18.1.2-201 through 3A.18.1.2-203 indicate that the enveloping site-specific load demands exceed the corresponding loads used for the standard design of the CB structures. The comparisons in Figure 3A.18.1.2-201 and Table 3A.18.1.2-201 indicate that the Unit 3 site-specific horizontal load demands on the CB walls can exceed the seismic loads used for standard design by as much as 50 percent. Comparisons in Figure 3A.18.1.2-202 and Table 3A.18.1.2-202 show the exceedances in vertical loads to be smaller (< 30 percent). Exceedances

of the local out-of-plane loads on flexible slabs are no more than 20 percent.

Site-specific evaluations of the CB structures are performed to address these exceedances in the site-specific load demands and calculate the available margins of CB structures at the Unit 3 site. These site-specific evaluations use site-specific seismic loads that are developed based on the results obtained from the site-specific SSI analysis of the CB and bound the effects of structural variations as described in Section 3A.17.9.2.

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Table 3A.18.1.2-201 **Enveloping Maximum Structural Force and Moment Demands CB**

Element		Standard Design					NA3 Enveloping Demands**)					Difference/Exceedance				
		Shear (MN)		Bending (MN-m)		Torsion	Shear (MN)		Bending (MN-m)		Torsion	Shear		Bending		
Elev. (m)	Node No.	NS	EW	NS*)	EW*)	(MN-m)	NS	EW	NS*)	EW*)	(MN-m)	NS	EW	NS	EW	Torsion
13.80	6	33.1	29.1	160	124	23.1	42.7	40.2	104	83	27.5	29%	38%	-35%	-33%	19%
	5			250	197				276	226				10%	15%	
9.06	5	53.4	54.8	360	275	44.9	77.9	70.1	360	293	57.7	46%	28%	0%	6%	29%
	4			573	443				685	562				20%	27%	
4.65	4	75.6	80.1	723	540	56.9	101.0	91.2	382	204	51.2	34%	14%	-47%	-62%	-10%
	3			1136	988				1053	736				-7%	-25%	
-2.00	3	124.4	99.4	1232	1036	59.9	41.0	44.8	567	511	26.9	-67%	-55%	-54%	-51%	-55%
-7.40	2			1570	1525				771	693				-51%	-55%	

Note: The shaded values in the table show exceedance from standard design.

*) NS and EW represent moments for bending the NS and EW direction, respectively.

**) Envelope of results from the site-specific SSI analyses of CB UC_{SSE} model with full stiffness and SSE damping (Analysis Cases 7 to 12 in Table 3A.15-202)

NAPS DEP 3.7-1

Table 3A.18.1.2-202 **Enveloping Maximum Accelerations CB**

		Acceleration (g)								
		Standard Design			NA3 Enveloping*)			Difference		
Elev. (m)	Node No.	NS	EW	Vert.	NS	EW	Vert.	NS	EW	Vert.
13.80	6	1.26	1.11	1.00	1.63	1.55	1.05	29%	40%	5%
9.06	5	0.88	0.90	0.86	1.26	1.12	0.95	43%	25%	11%
4.65	4	0.86	0.82	0.74	0.88	0.98	0.82	2%	19%	10%
-2.00	3	0.79	0.71	0.56	0.63	0.67	0.60	-20%	-6%	6%
-7.40	2	0.54	0.54	0.51	0.48	0.43	0.64	-12%	-21%	25%
-10.40	1	0.54	0.53	0.51	0.48	0.43	0.63	-12%	-19%	24%

Note: *) Envelope of results from the site-specific SSI analyses of CB UC_{SSE} model with full stiffness and SSE damping (Analysis Cases 7 to 12 in Table 3A.15-202)
The presented values are the maximum accelerations at floor lumped mass locations.
The shaded values in the table show exceedance from standard design.

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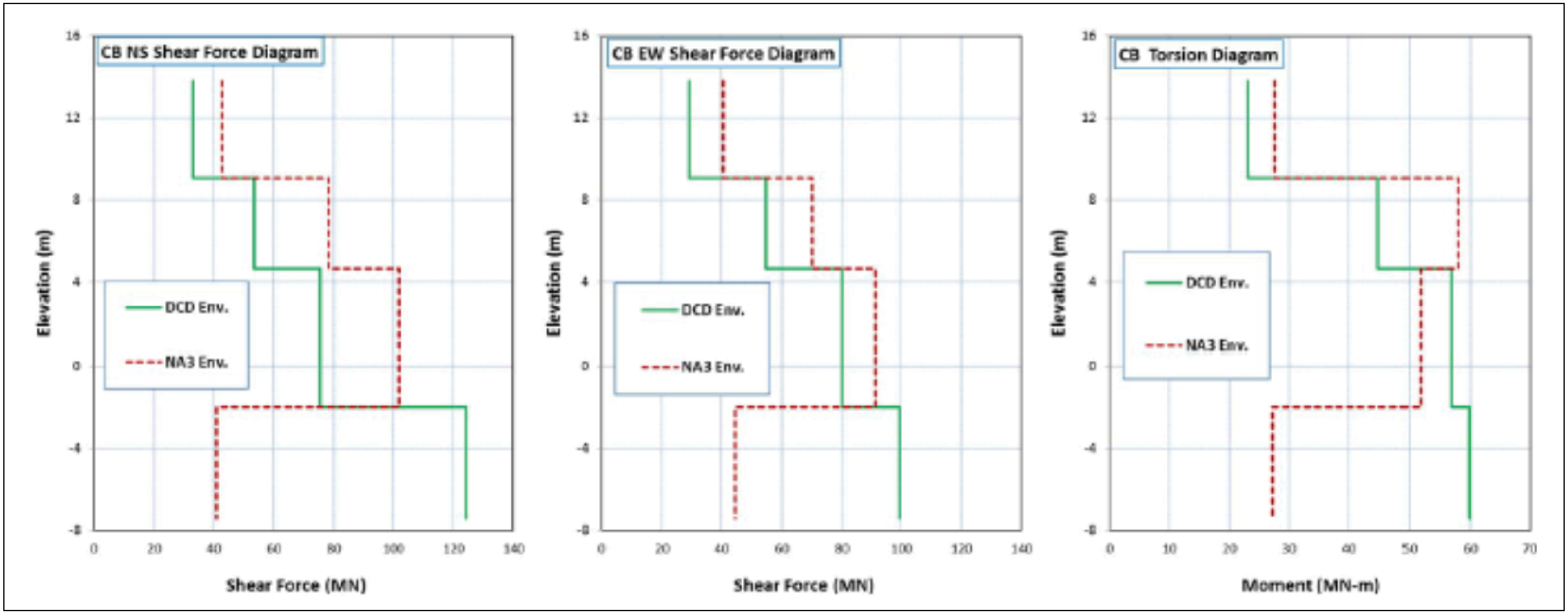
Table 3A.18.1.2-203 **Enveloping Maximum Out-of-Plane Loads on CB Flexible Slabs**

		Slab Equivalent Out-of-Plane Acceleration Load (g)								
		Partial Column			Full Column					
El. (m)	Location	BE	UB	LB	BE	UB	LB	NA3 Envelope*)	Standard Design	Difference
13.80	Roof	1.34	1.52	1.12	0.93	1.13	0.83	1.52	1.39	10%
9.06	CA-CD	1.08	1.21	0.92	0.80	0.98	0.66	1.21	1.08	12%
4.65	CA-CD	0.82	0.91	0.70	0.66	0.75	0.58	0.91	0.87	5%
-2.00	CA-CD	0.62	0.66	0.59	0.52	0.62	0.42	0.66	0.66	0%

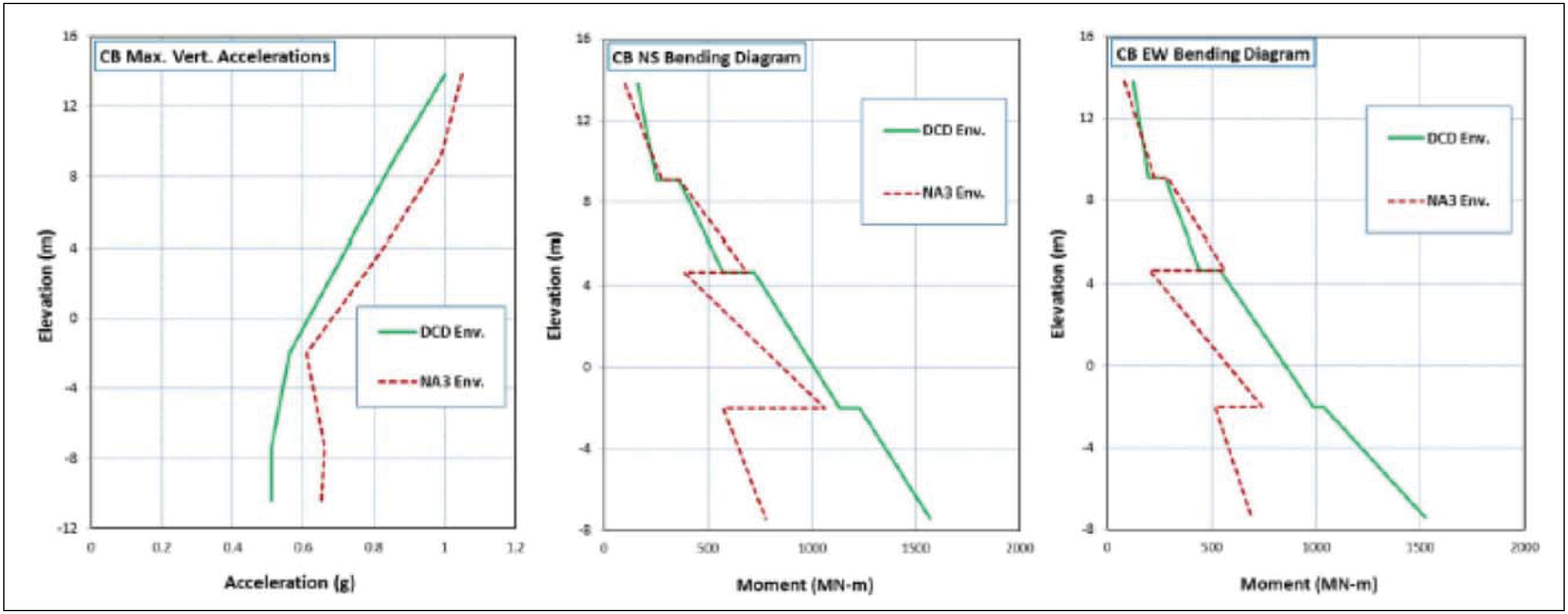
Note: *)Envelope of results from the site-specific SSI analyses of CB UC_{SSE} model with full stiffness and SSE damping (Analysis Cases 7 to 12 in Table 3A.15-202)
The presented values are the maximum accelerations at floor lumped mass locations.
The shaded values in the table show exceedance from standard design.

NAPS DEP 3.7-1 Figure 3A.18.1.2-201 **CB Site-Specific Horizontal Seismic Load Demands**



NAPS DEP 3.7-1

Figure 3A.18.1.2-202 **CB Site-Specific Vertical Seismic Load Demands**



3A.18.1.3 Unit 3 FWSC SSI Enveloping Structural Load Demands

The site-specific enveloping seismic responses presented in this section form the basis for seismic design and evaluation of the FWSC at the Unit 3 site. The site-specific SSSI evaluation shows that responses obtained from the SSI analyses of the stand-alone FWSC models do not envelope the SSSI effects of the CB on the FWSC seismic response. Therefore, results of the site-specific SSI analyses of the FWSC stand-alone models presented in Section 3A.17 are used, together with the corresponding results of the site-specific SSSI analyses of the FWSC-CB combined model, to develop the seismic load demands for the site-specific evaluation of the Unit 3 FWSC structures. Site-specific enveloping seismic load demands on the FWSC structures are developed as an envelope of the results for maximum member forces and vertical accelerations obtained from the SSI and SSSI analyses of the FWSC stand-alone and the FWSC-CB combined models with full (uncracked concrete) stiffness properties and SSE damping using deep input control motion applied at the bottom of the concrete fill at Elevation 220 ft NAVD88. The comparisons presented in Section 3A.17 of the maximum responses obtained from the site–specific SSI analyses of the FWSC model with full stiffness properties and OBE damping show that analyses with deep input motions yield maximum response results that envelope the results of analyses with surface input motion at Elevation 282 ft NAVD88. Comparisons of the maximum responses obtained from the site-specific SSSI analyses of the FWSC-CB combined model with full stiffness properties and OBE damping results in similar conclusions.

Table 3A.18.1.3-201 presents the FWSC enveloping maximum member forces and moments obtained as the envelope of the results from the site-specific SSI analysis (Cases 7 through 9 in Table 3A.15-203) and the FWSC-CB SSSI analysis (Cases FC7 through FC9 in Table 3A.15-206) performed on structural models with full (uncracked concrete) stiffness properties and SSE damping. This table compares these site-specific enveloping maximum member forces and moments are compared with the corresponding standard design values.

Table 3A.18.1.3-202 presents the enveloping maximum accelerations at FWSC lumped mass locations obtained as envelope of the results from the site-specific SSI analysis Cases 7 through 9 in Table 3A.15-203 and the FWSC-CB SSSI analysis Cases FC7 through FC9 in

Table 3A.15-206 performed on structural models with full (uncracked concrete) stiffness properties and SSE damping. This table compares these site-specific enveloping maximum accelerations to the corresponding standard design enveloping accelerations.

Table 3A.18.1.3-203 presents the envelope of results for maximum accelerations of the FWSC SDOF oscillators obtained from the site-specific FWSC SSI analyses of the FWSC model with full stiffness properties and SSE damping (analysis Cases 7 to 9 in Table 3A.15-203). These results are enveloped with the corresponding results from the FWSC-CB SSSI analysis (Cases FC7 through FC9 in Table 3A.15-206) to obtain enveloping maximum accelerations used for the calculation of the site-specific out-of-plane seismic loads on the FWS roof and the site-specific hydrodynamic seismic loads from the water contained in the FWS tank. Table 3A.18.1.3-203 also presents a comparison of the site-specific enveloping maximum accelerations of the FWSC SDOF oscillators with the corresponding standard design values.

Table 3A.18.1.3-204 presents the calculations of the equivalent average acceleration representing the out-of-plane seismic load demand on the FWS roof. In Table 3A.18.1.3-204, the site-specific out-of-plane seismic load demands on the FWS roof are compared with the corresponding loads used for the standard design of the FWSC structures.

Figures 3A.18.1.3-201 and 3A.18.1.3-202 present diagrams of the site-specific horizontal load demands on the FWS and FPE structures, respectively. Vertical load demands on these two FWSC structures are shown in Figures 3A.18.1.3-203 and 3A.18.1.3-204. Figures 3A.18.1.3-201 through 3A.18.1.3-204 compare the site-specific load demands with the corresponding loads used for the standard design.

Comparisons presented in Figures 3A.18.1.3-201 through 3A.18.1.3-204 and Tables 3A.18.1.3-201 through 3A.18.1.3-204 show that, in general, the standard design envelopes the site-specific load demands on the FWSC structures at the Unit 3 site with a few small exceedances that have a local effect. The largest exceedances in the torsional load demands on the FWS structure are approximately 2.5 times the torsion considered in the standard design. As shown in 3A.18.1.3-204, the out-of-plane load on the FWS roof can exceed the corresponding standard design load by 32 percent. Comparisons presented in Figure 3A.18.1.3-201 and Table 3A.18.1.3-202 indicate small

(< 10 percent) exceedances in the site-specific shear load demands on the FWS structure.

Site-specific evaluations of the FWSC structures are performed to address these exceedances in the site-specific load demands and to calculate the available margins at the Unit 3 site. These site-specific evaluations use site-specific seismic loads that are developed based on the FWSC enveloping forces, moments and accelerations and which bound the effects of structural stiffness variations as described in Section 3A.17.9.3.

NAPS DEP 3.7-1

Table 3A.18.1.3-201 **Enveloping Maximum Member Forces and Moments FWSC**

Structure	Element			Standard Design					NA3 Enveloping					Difference				
	Elev. (m)	No.	Node No.	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS	Bending (MN-m) EW	Torsion (MN-m)	Shear (MN) NS	Shear (MN) EW	Bending (MN-m) NS	Bending (MN-m) EW	Torsion (MN-m)	Shear NS	Shear EW	Bending NS	Bending EW	Torsion
FWS	19.70	9	10	4.6	5.1	4	7	0.7	4.7	5.2	5	3	2.4	3%	2%	7%	-50%	248%
			9			14	19				16	15				13%	-20%	
	17.25	8	9	11.1	12.1	22	27	2.2	11.4	12.8	25	21	7.4	2%	5%	14%	-20%	235%
			8			39	47				44	43				14%	-8%	
	15.53	7	8	15.5	16.5	45	57	3.6	15.8	17.8	54	48	12.2	2%	8%	18%	-17%	239%
			7			71	84				80	80				13%	-6%	
	13.81	6	7	19.3	20.1	76	92	4.9	19.7	22.1	88	85	16.7	2%	10%	16%	-8%	240%
			6			108	125				120	123				12%	-1%	
	12.10	5	6	22.8	23.8	111	128	5.8	22.4	25.1	126	127	20.0	-2%	6%	13%	-1%	244%
			5			134	153				149	155				11%	1%	
	11.00	4	5	24.6	25.3	136	157	6.4	24.3	27.2	153	158	22.3	-1%	8%	13%	1%	249%
			4			163	184				178	188				9%	2%	
	9.90	3	4	26.1	26.6	166	188	6.9	25.8	29.0	182	191	24.4	-1%	9%	10%	2%	254%
			3			194	217				207	222				6%	3%	
	8.81	2	3	43.3	45.5	197	221	7.5	40.5	44.5	212	226	27.1	-7%	-2%	8%	2%	262%
			2			279	295				290	318				4%	8%	
	6.73	1	2	45.3	48.0	281	299	8.1	41.6	46.1	294	321	29.6	-8%	-4%	5%	8%	265%
	4.65		8002			366	375				379	417				3%	11%	
FPE	8.25	405	402	8.1	7.4	2	10	15.1	4.1	7.1	2	7	6.2	-50%	-4%	-25%	-32%	-59%
	4.65	404	401			28	27				14	27				-51%	0%	

Shaded values are exceedances of site-specific loads with respect to standard design.

NAPS DEP 3.7-1

Table 3A.18.1.3-202 **Enveloping Maximum Accelerations FWSC**

Elev. (m)	Node No.	Location	Standard Design NS (g)	Standard Design EW (g)	Standard Design Vert. (g)	NA3 Enveloping NS (g)	NA3 Enveloping EW (g)	NA3 Enveloping Vert. (g)	Difference NS	Difference EW	Difference Vert.
19.70	10	FWS	2.20	2.40	1.69	2.21	2.42	1.43	1%	1%	-15%
17.25	9	FWS	2.00	2.10	1.64	2.00	2.26	1.43	0%	7%	-13%
15.53	8	FWS	1.80	1.80	1.58	1.80	2.04	1.40	0%	13%	-11%
13.81	7	FWS	1.60	1.60	1.58	1.58	1.78	1.35	-1%	11%	-15%
12.10	6	FWS	1.40	1.60	1.43	1.35	1.50	1.27	-3%	-6%	-11%
11.00	5	FWS	1.30	1.50	1.23	1.20	1.32	1.21	-8%	-12%	-1%
9.90	4	FWS	1.20	1.40	1.13	1.05	1.13	1.15	-13%	-19%	1%
8.81	3	FWS	1.10	1.40	1.05	0.94	1.01	1.07	-15%	-28%	2%
6.73	2	FWS	0.80	0.90	1.00	0.70	0.81	0.92	-13%	-9%	-8%
4.65	8002	Basemat Top	0.70	0.80	0.78	0.63	0.66	0.75	-10%	-17%	-4%
2.15	8001	Basemat Bottom	0.70	0.80	0.78	0.66	0.58	0.90	-5%	-28%	16%
8.25	405	FPE	1.70	1.60	1.12	0.81	1.46	0.77	-52%	-9%	-31%
6.45	402	FPE	1.20	1.20	1.09	0.72	1.10	0.72	-40%	-8%	-34%

Shaded values are exceedances of site-specific loads with respect to standard design.

NAPS DEP 3.7-1

Table 3A.18.1.3-203 **Site-Specific Enveloping Maximum Accelerations of FWSC SDOF Oscillators**

SDOF Oscillator Elev. (m)	Node No.	Description	Direction	Acceleration (g) SSI Analyses Envelope	SSSI Analyses Envelope*)	NA3 Enveloping	Standard Design	Difference
19.70	11	FWS Roof	Vert. (Z)	3.36	3.98	3.98	3.26	22%
12.10	60	FWS Water Sloshing Mode	NS (X)	0.10	0.10	0.10	0.30	-66%
			EW (Y)	0.09	0.09	0.09	0.40	-78%
8.81	30	FWS Water Impulsive Mode	NS (X)	0.87	0.94	0.94	1.10	-15%
			EW (Y)	1.01	1.00	1.01	1.40	-28%

Shaded values are exceedances of site-specific loads with respect to standard design.
*) Obtained from the FWSC-CB SSSI Analyses

NAPS DEP 3.7-1

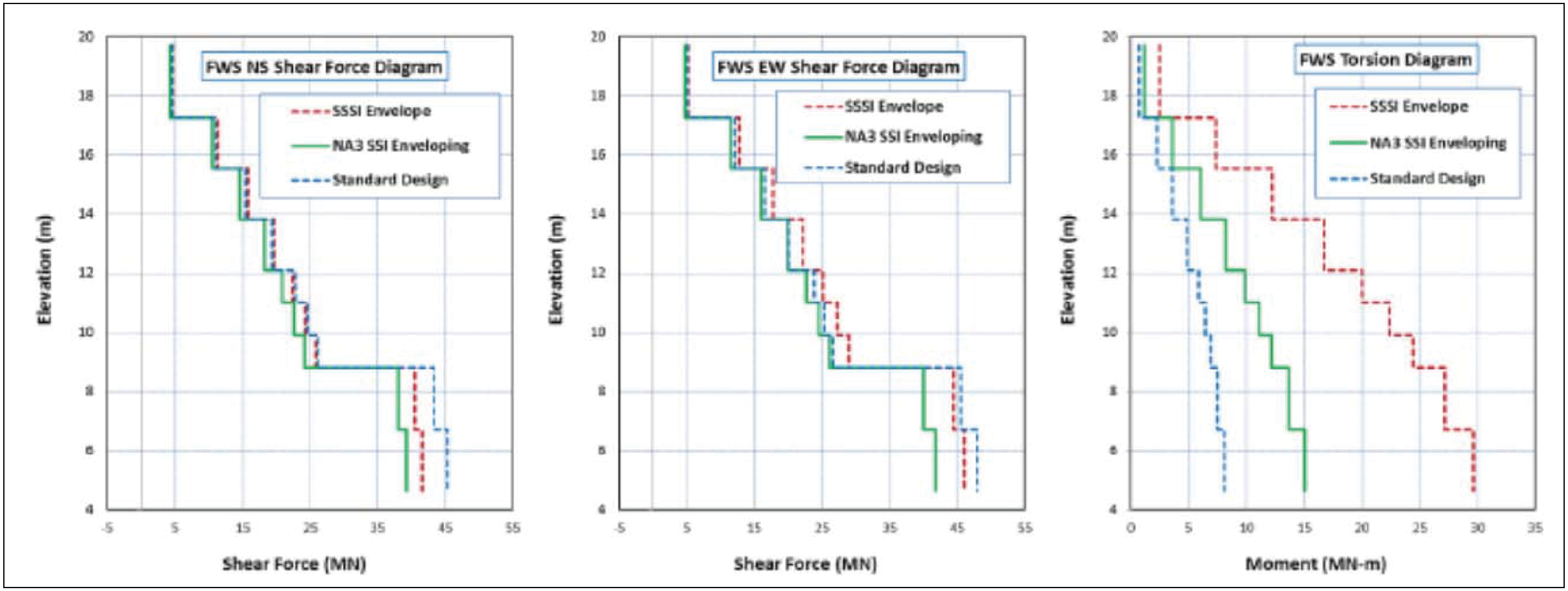
Table 3A.18.1.3-204 **Site-Specific Out-of-Plane Load on FWS Roof**

Slab		Flexible Mode (SDOF Oscillator)				Rigid Mode (LMSM)				Eq. Ave. Acc. (g)			
				Acceleration (g)				Acceleration (g)		NA3 Site-Specific			
Elev. (m)	Location	Weight (kN)	Mass Node	SSSI Envelope*)	SSI Envelope	Mass Node	Weight (kN)	SSSI Envelope*)	SSI Envelope	SSSI Envelope	SSI Envelope	Stand. Design	Exceed.
19.70	FWS Roof	1339	11	3.98	3.36	10	2480	1.40	1.43	2.30	2.11	1.74	32%

Shaded values are the governing case.

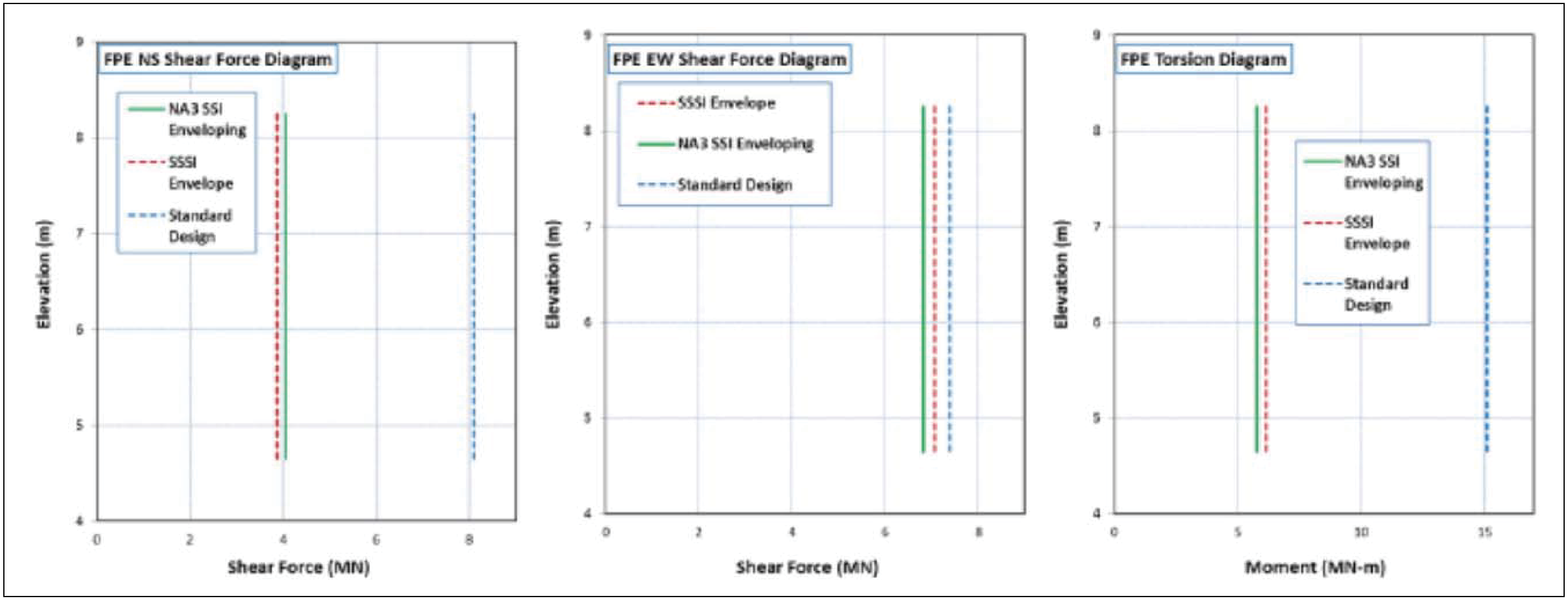
*) Obtained from the FWSC-CB SSSI Analyses

NAPS DEP 3.7-1 Figure 3A.18.1.3-201 **Comparison of Horizontal Seismic Load Demands on FWS Structure**



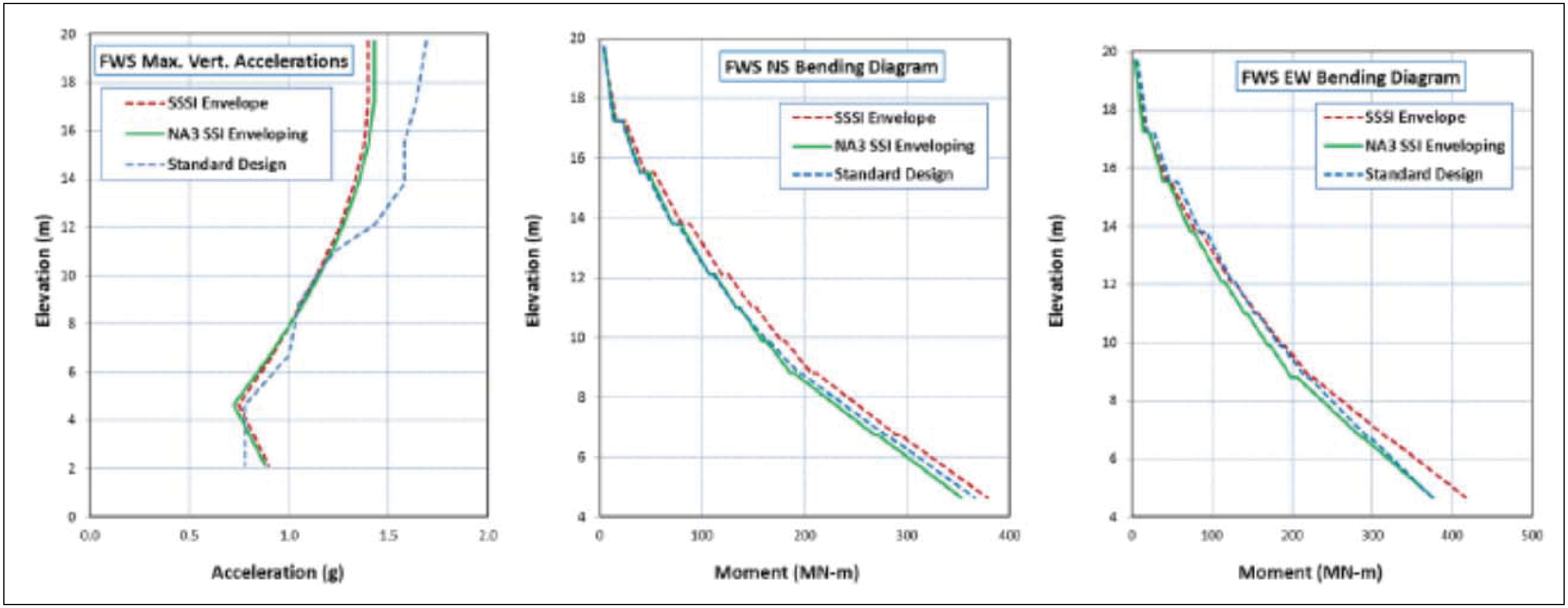
NAPS DEP 3.7-1

Figure 3A.18.1.3-202 **Comparison of Horizontal Seismic Load Demands on FPE Structure**



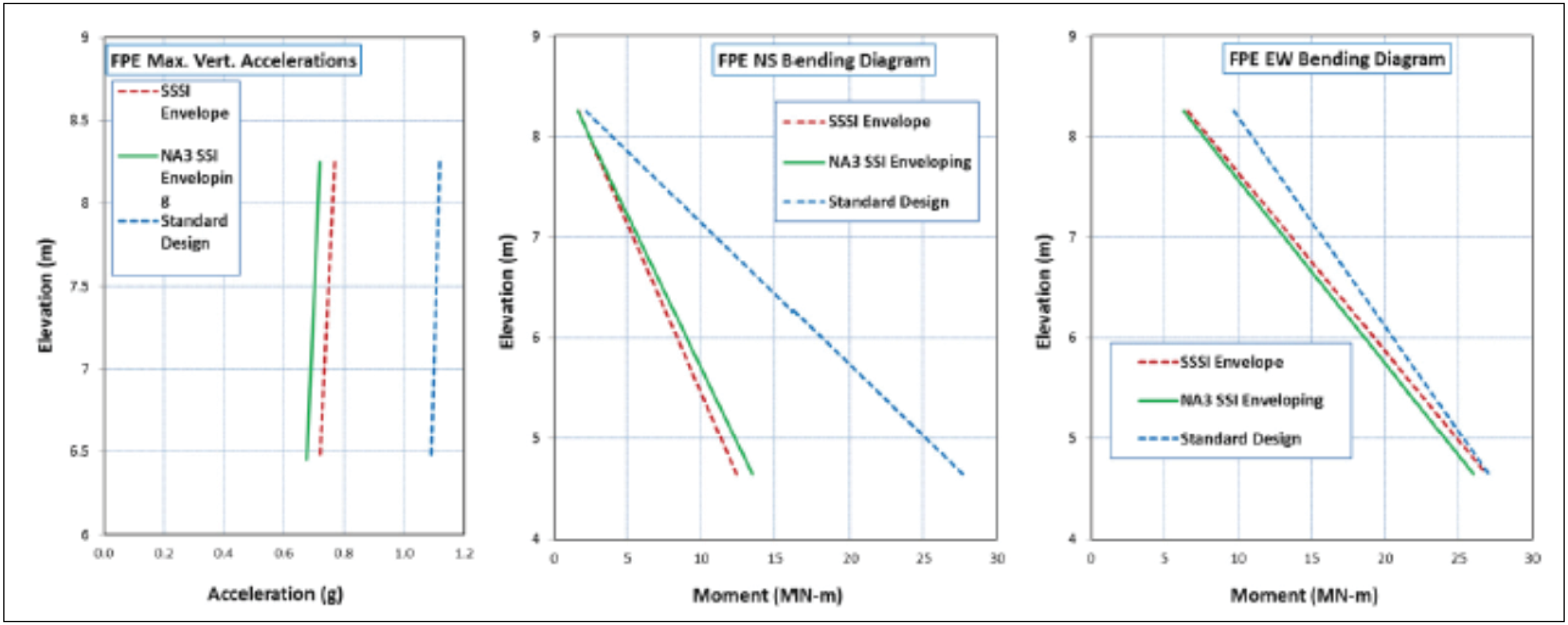
NAPS DEP 3.7-1

Figure 3A.18.1.3-203 **Comparison of Vertical Seismic Load Demands on FWS Structure**



NAPS DEP 3.7-1

Figure 3A.18.1.3-204 **Comparison of Vertical Seismic Load Demands on FPE Structure**



3A.18.2 Unit 3 SSI Enveloping Floor Response Spectra

For the RB/FB, CB, and FWSC, site-specific enveloping ISRS for critical damping ratios 2, 3, 4, 5, 7, 10, and 20 percent are determined for all locations in the RB/FB that are peak broadened by +15 percent and valley filled.

These ISRS represent the envelope of ISRS results from the site-specific SSI analyses of RB/FB model with UB stiffness properties and OBE damping (analysis Cases 1 through 6 in Table 3A.15-201) and serve as the basis for site-specific design and qualification of RB/FB equipment and components. To address possible exceedances related to the effects of structural stiffness variation, these spectra are enhanced as described in Section 3A.17.9.1.

These ISRS represent the envelope of ISRS results from the site-specific SSI analyses of the CB model with upper bound stiffness properties and OBE damping (analysis Cases 1 through 6 in Table 3A.15-202) and serve as basis for site-specific design and qualification of CB equipment and components. To address possible exceedances related to the concrete cracking effects of these spectra are enhanced as described in Section 3A.17.9.2.

For the FWSC, these ISRS represent the envelope of ISRS results from:

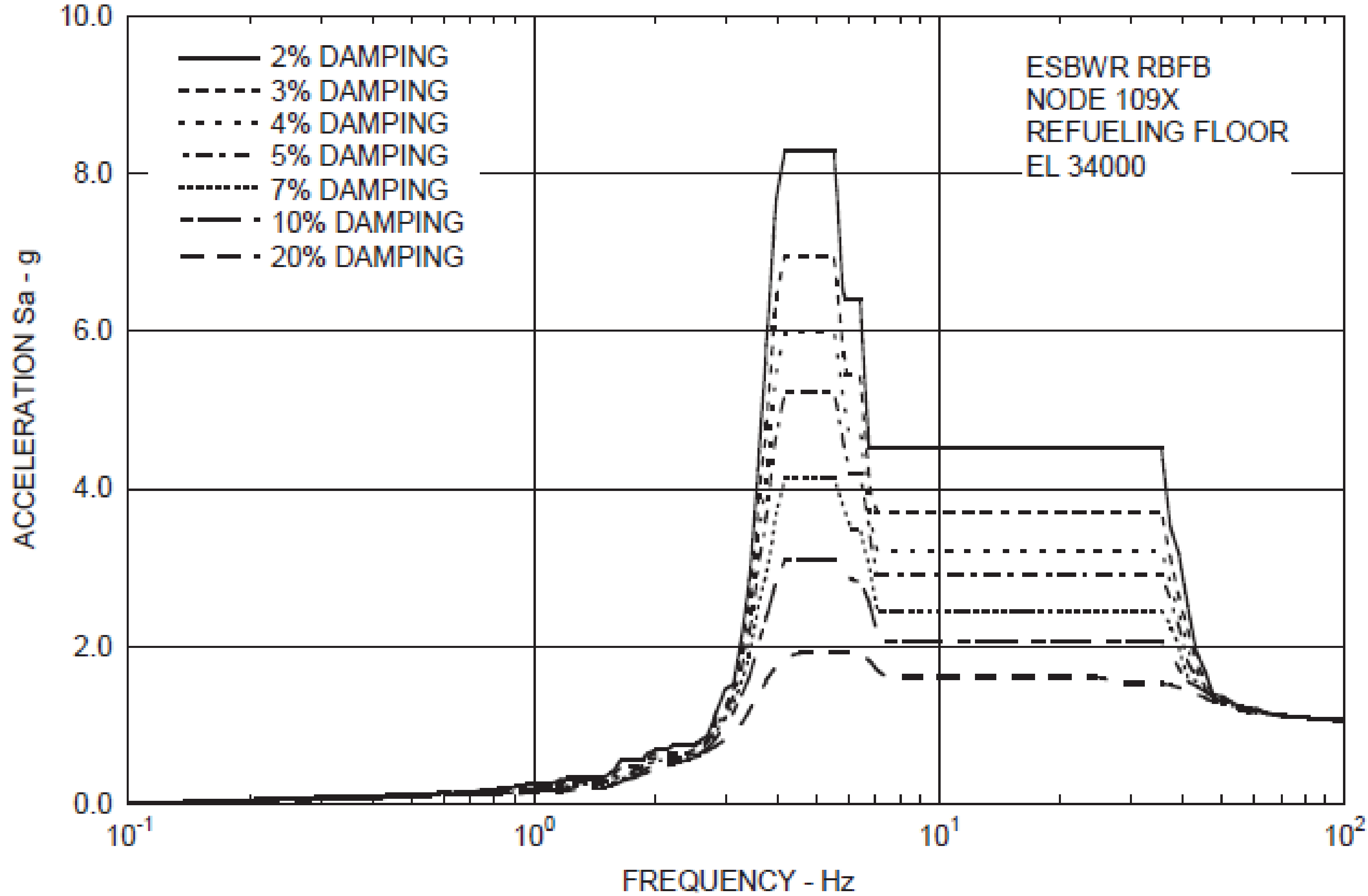
- Site-specific SSI analyses of the FWSC stand-alone model with full (uncracked concrete) stiffness properties and OBE damping (Cases 1 through 6 in Table 3A.15-203)
- Site-specific SSSI analyses of the FWSC-CB combined model with full stiffness properties and OBE damping (Cases FC1 through FC6 in Table 3A.15-206)

To address possible exceedances related to the concrete cracking effects, these spectra are enhanced as described in Section 3A.17.9.3. The site-specific design ISRS envelopes serve as the basis for site-specific design and qualification of FWSC equipment and components.

For key locations in the RB/FB, CB, and FWSC, Figures 3A.18.2-201a through 3A.18.2-203l present the site-specific design ISRS in NS(x), EW(y), and vertical (z) directions.

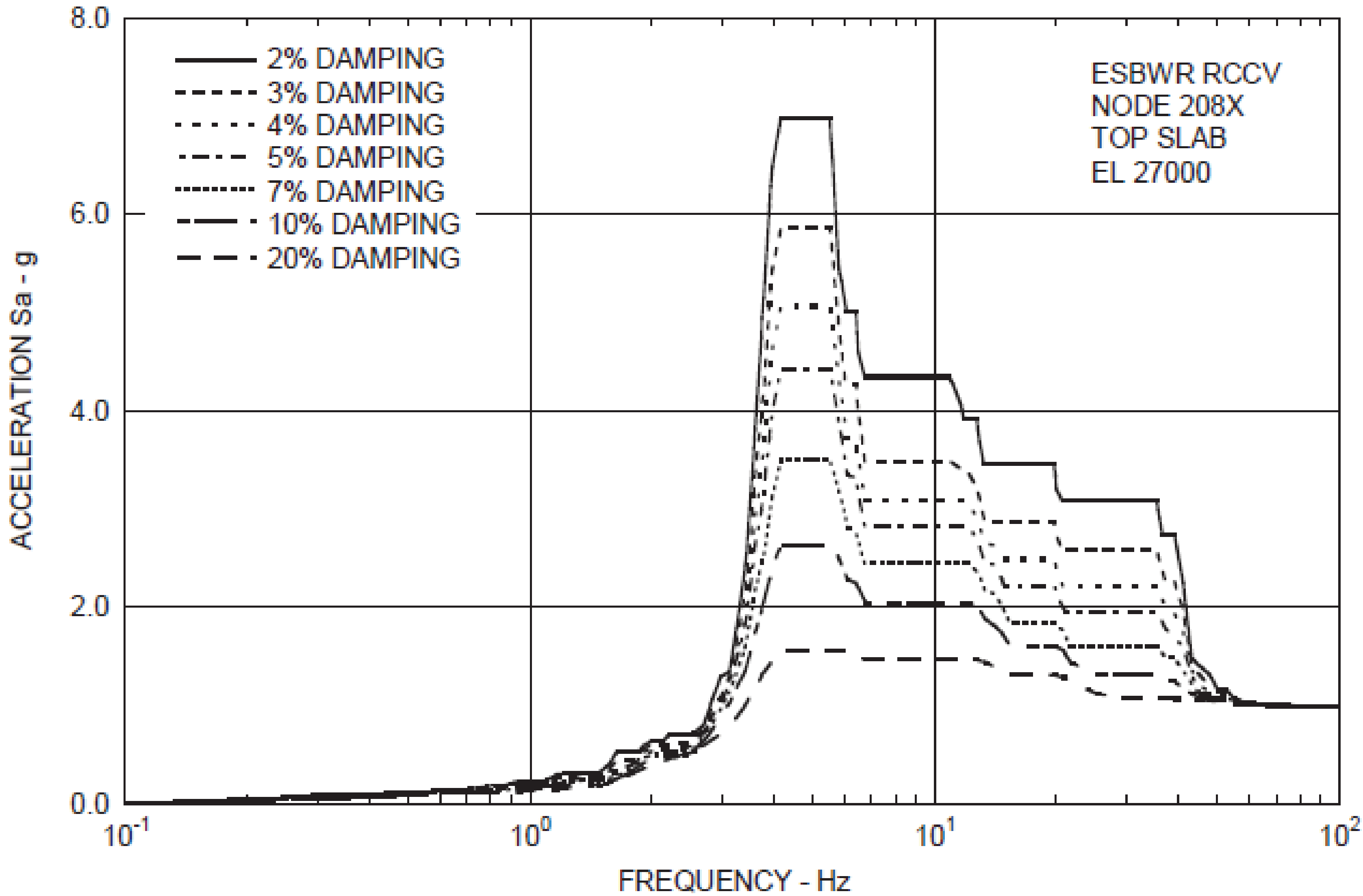
NAPS DEP 3.7-1

Figure 3A.18.2-201a **Site-Specific In-Structure Response Spectra - RB/FB Node 109 X**



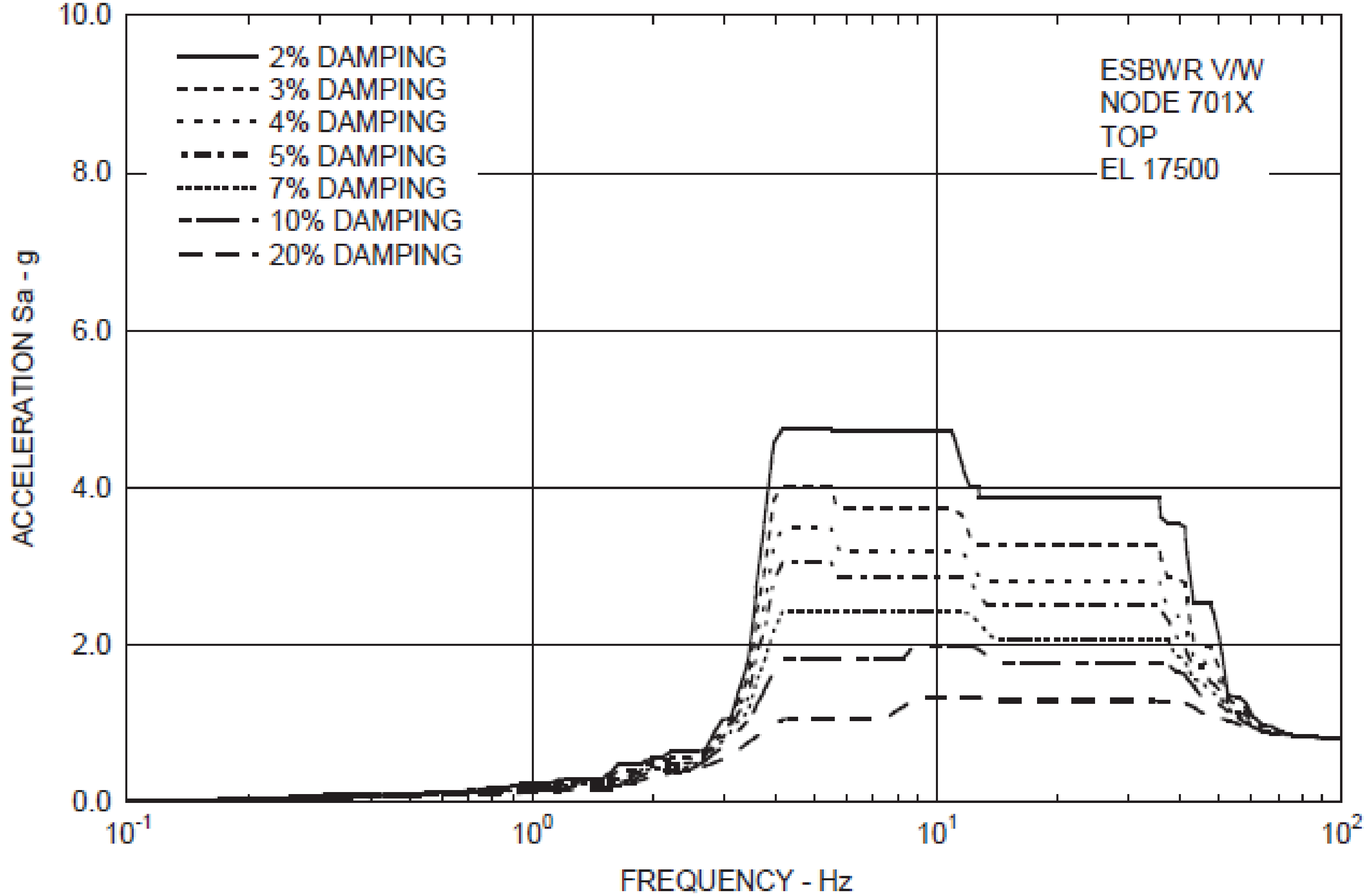
NAPS DEP 3.7-1

Figure 3A.18.2-201b **Site-Specific In-Structure Response Spectra - RCCV Node 208 X**



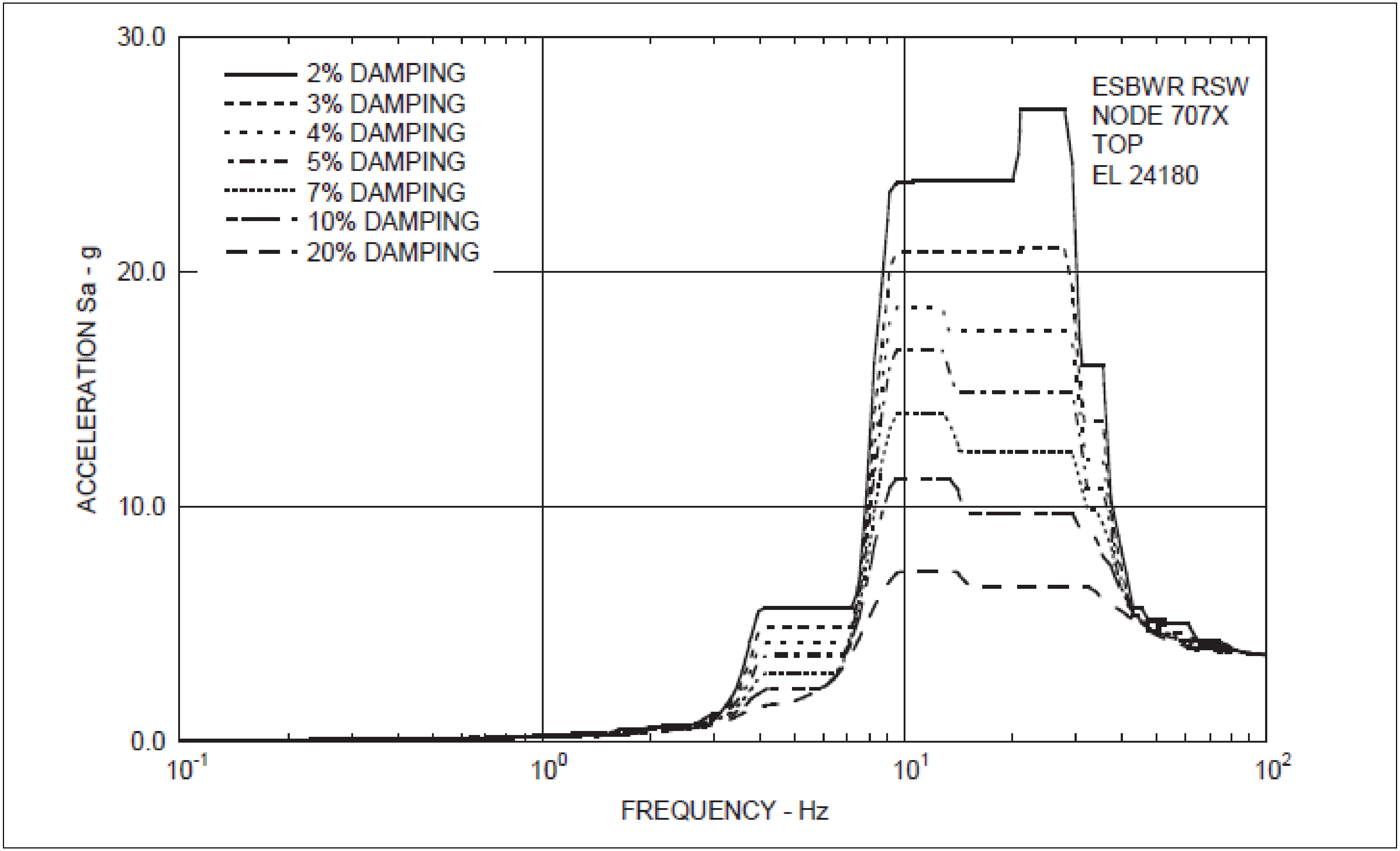
NAPS DEP 3.7-1

Figure 3A.18.2-201c **Site-Specific In-Structure Response Spectra - Vent Wall Node 701 X**



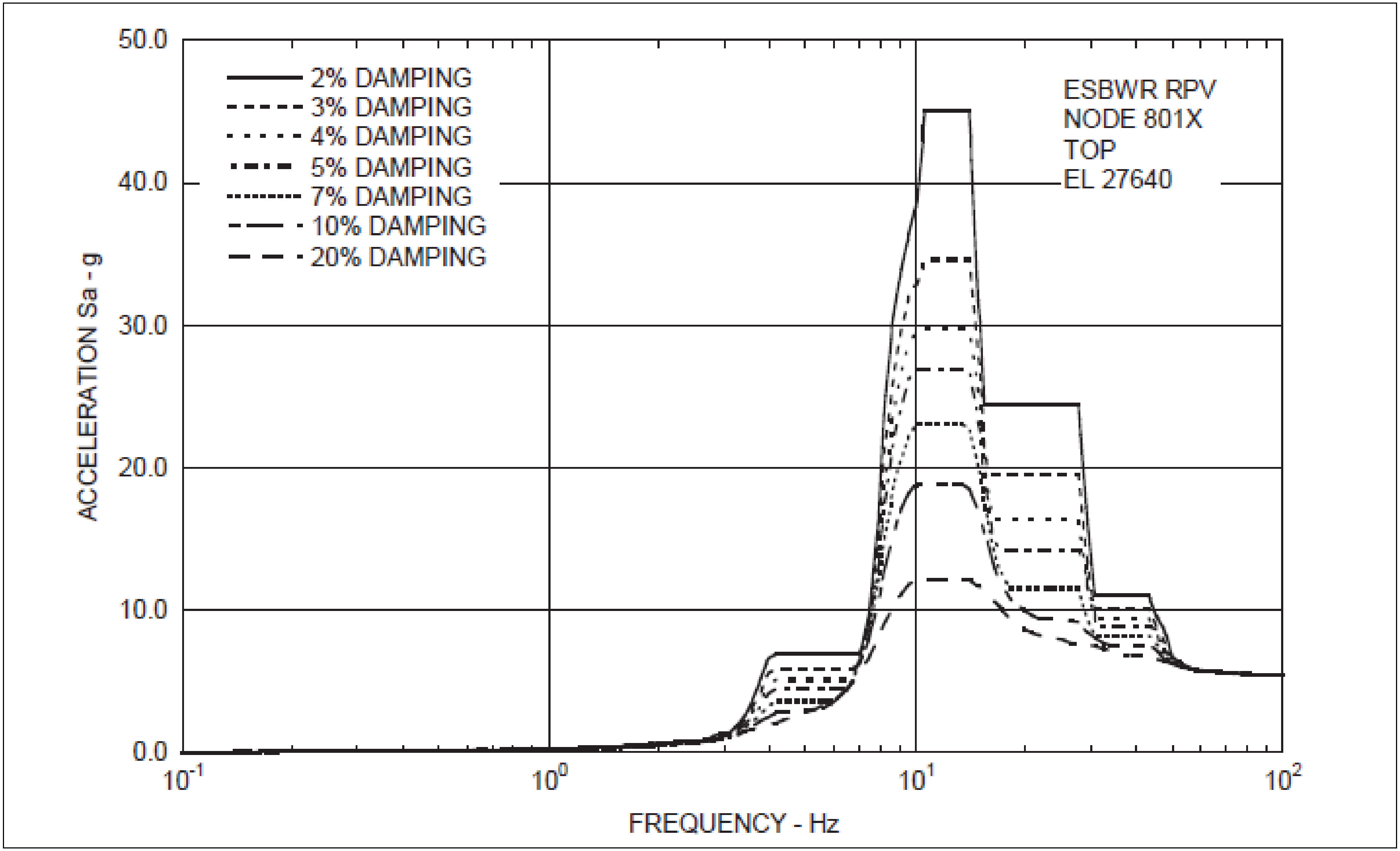
NAPS DEP 3.7-1

Figure 3A.18.2-201d **Site-Specific In-Structure Response Spectra - RSW Node 707 X**



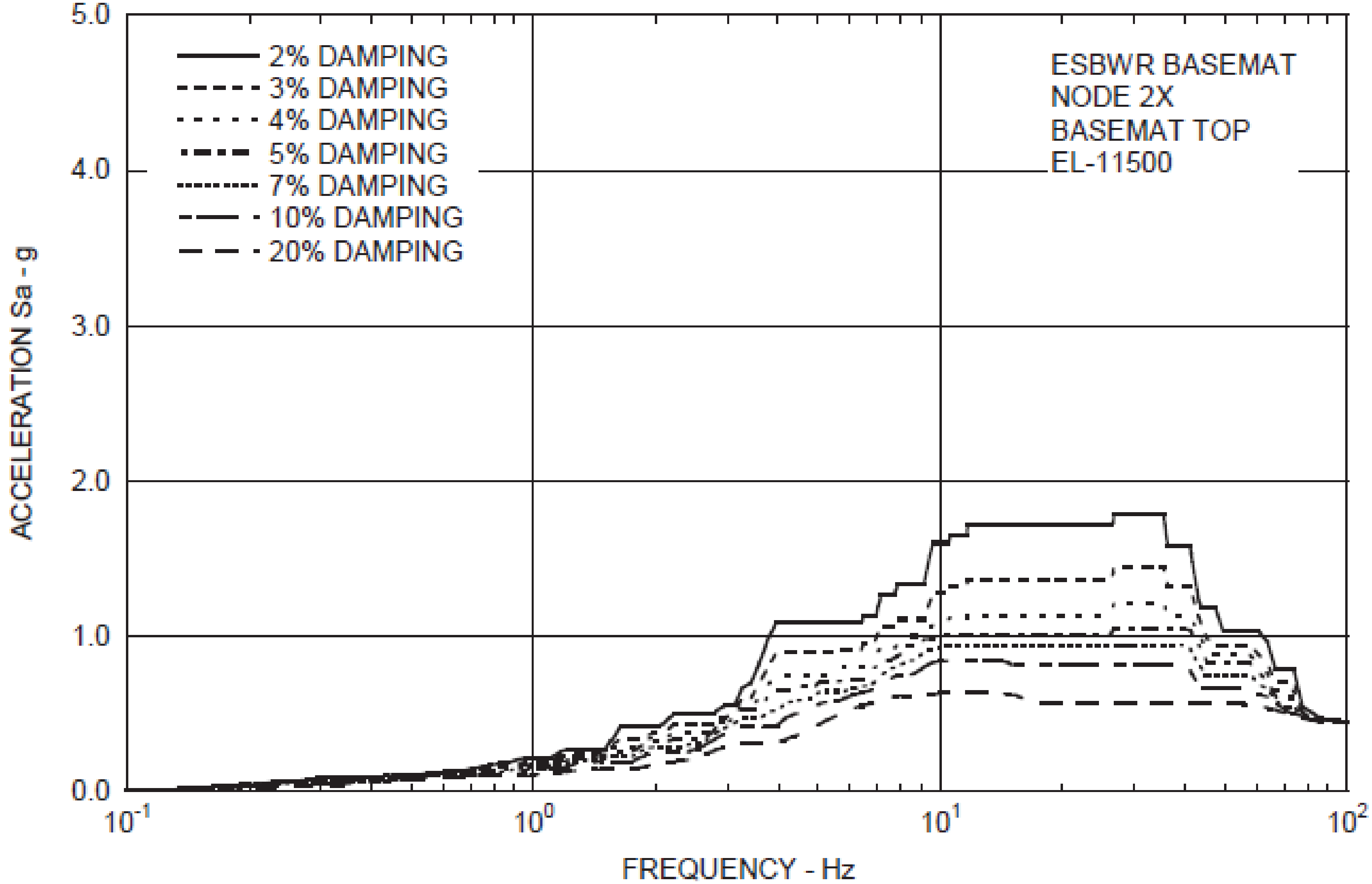
NAPS DEP 3.7-1

Figure 3A.18.2-201e **Site-Specific In-Structure Response Spectra - RPV Node 801 X**



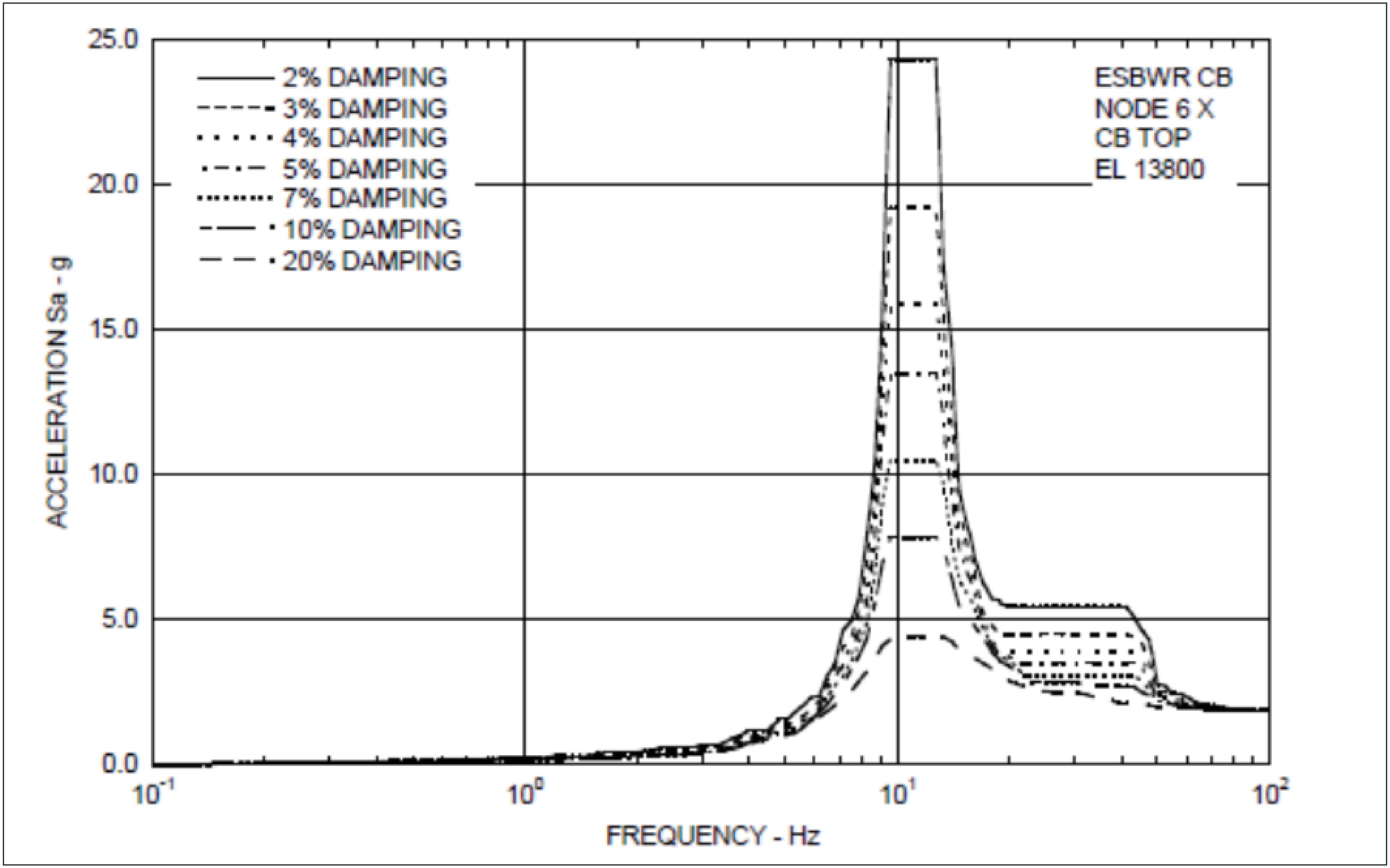
NAPS DEP 3.7-1

Figure 3A.18.2-201f Site-Specific In-Structure Response Spectra - RB/FB Node 2X



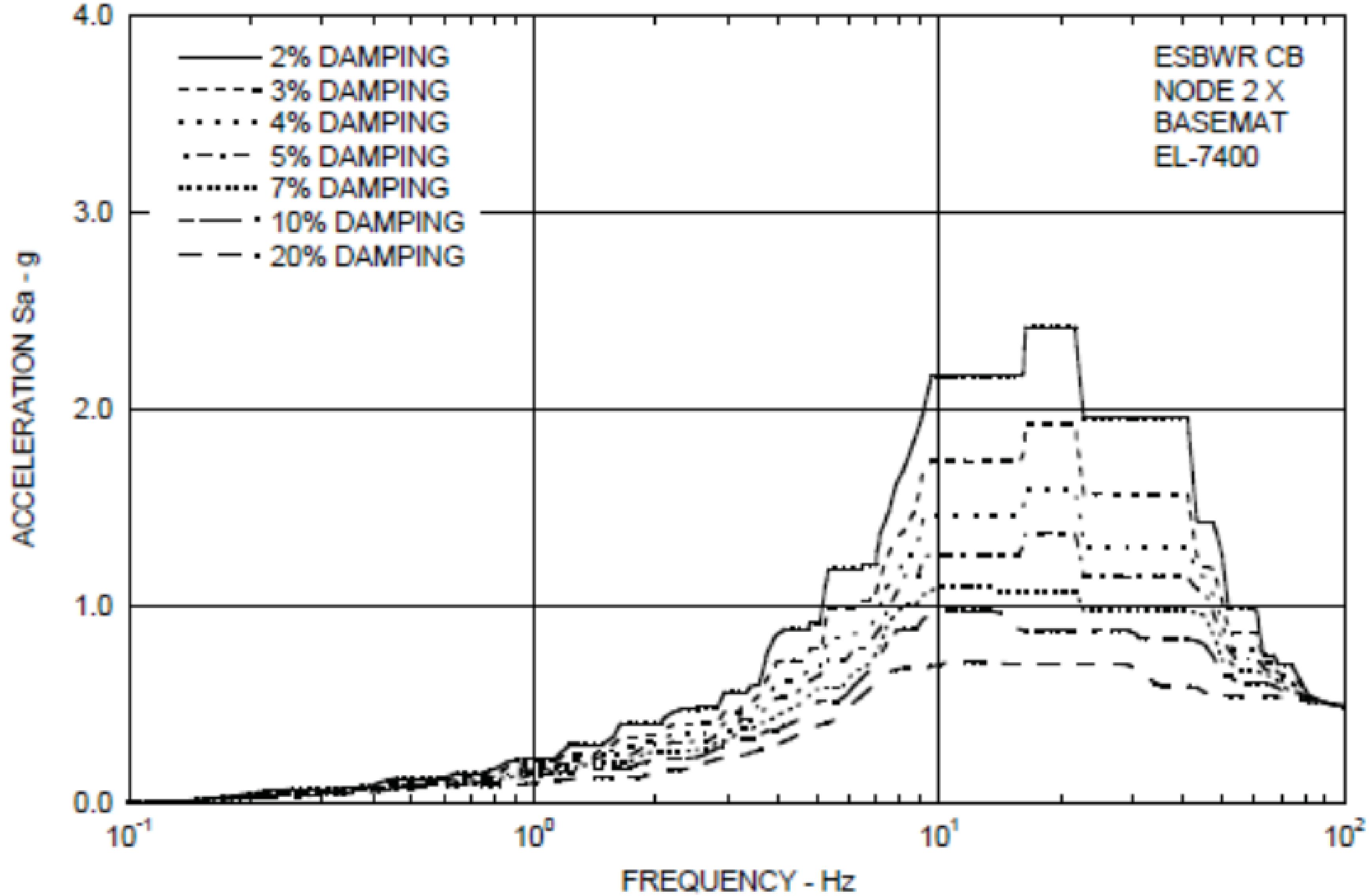
NAPS DEP 3.7-1

Figure 3A.18.2-201g Site-Specific In-Structure Response Spectra - CB Node 6 X



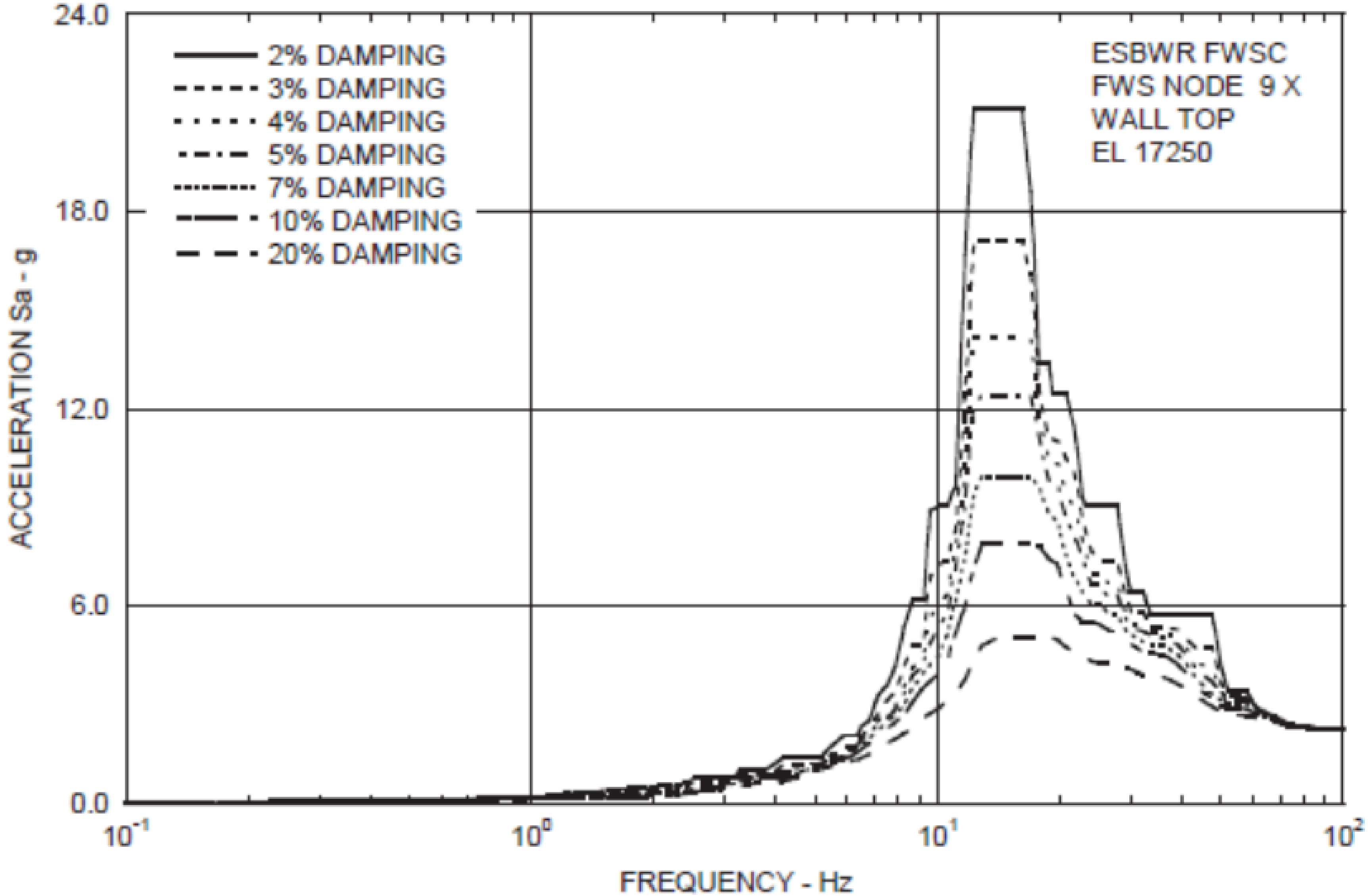
NAPS DEP 3.7-1

Figure 3A.18.2-201h **Site-Specific In-Structure Response Spectra - CB Node 2 X**



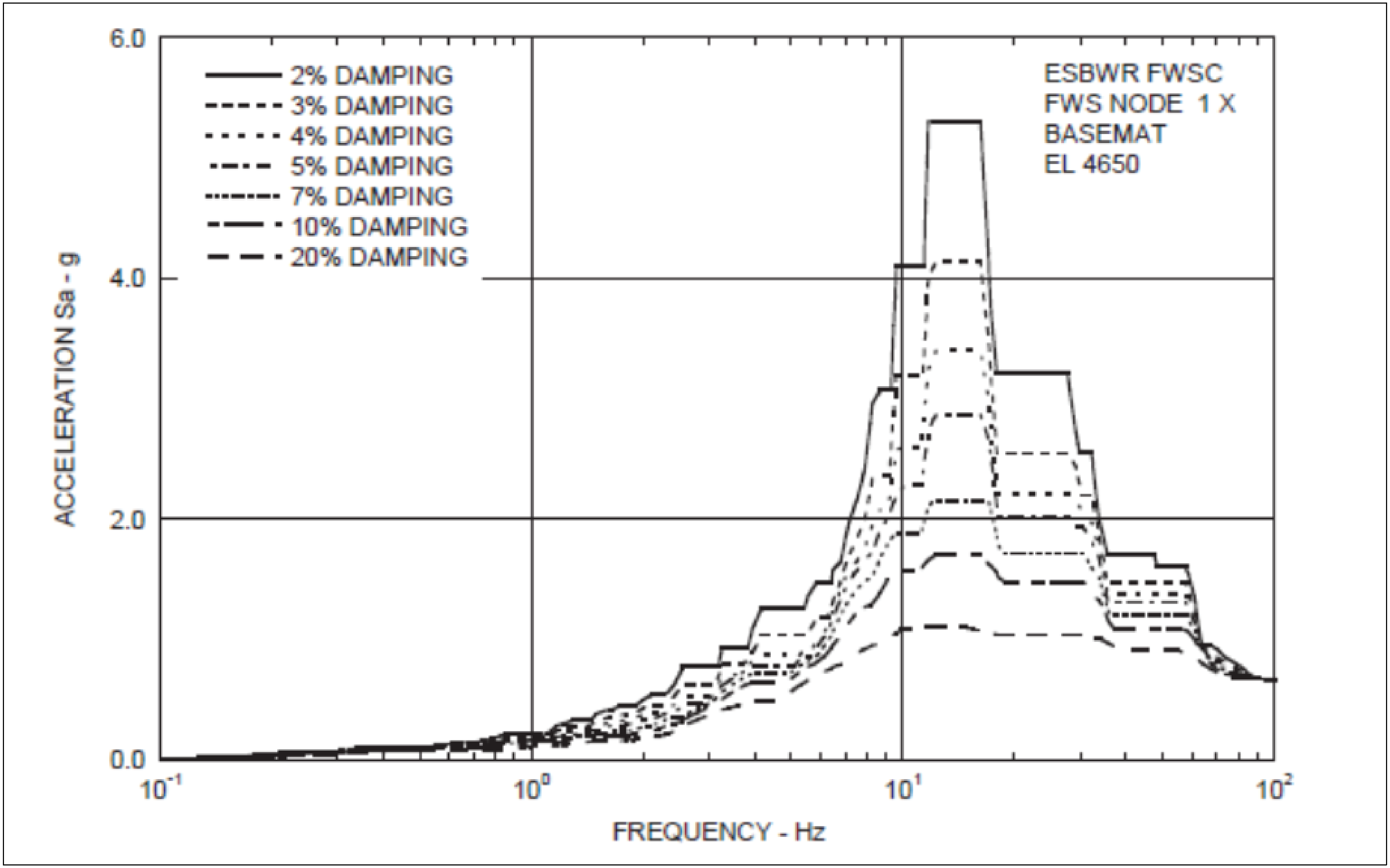
NAPS DEP 3.7-1

Figure 3A.18.2-201i **Site-Specific In-Structure Response Spectra - FWS Node 9 X**



NAPS DEP 3.7-1

Figure 3A.18.2-201j **Site-Specific In-Structure Response Spectra - FWS Node 1 X**



NAPS DEP 3.7-1

Figure 3A.18.2-201k **Site-Specific In-Structure Response Spectra - FPE Node 405 X**

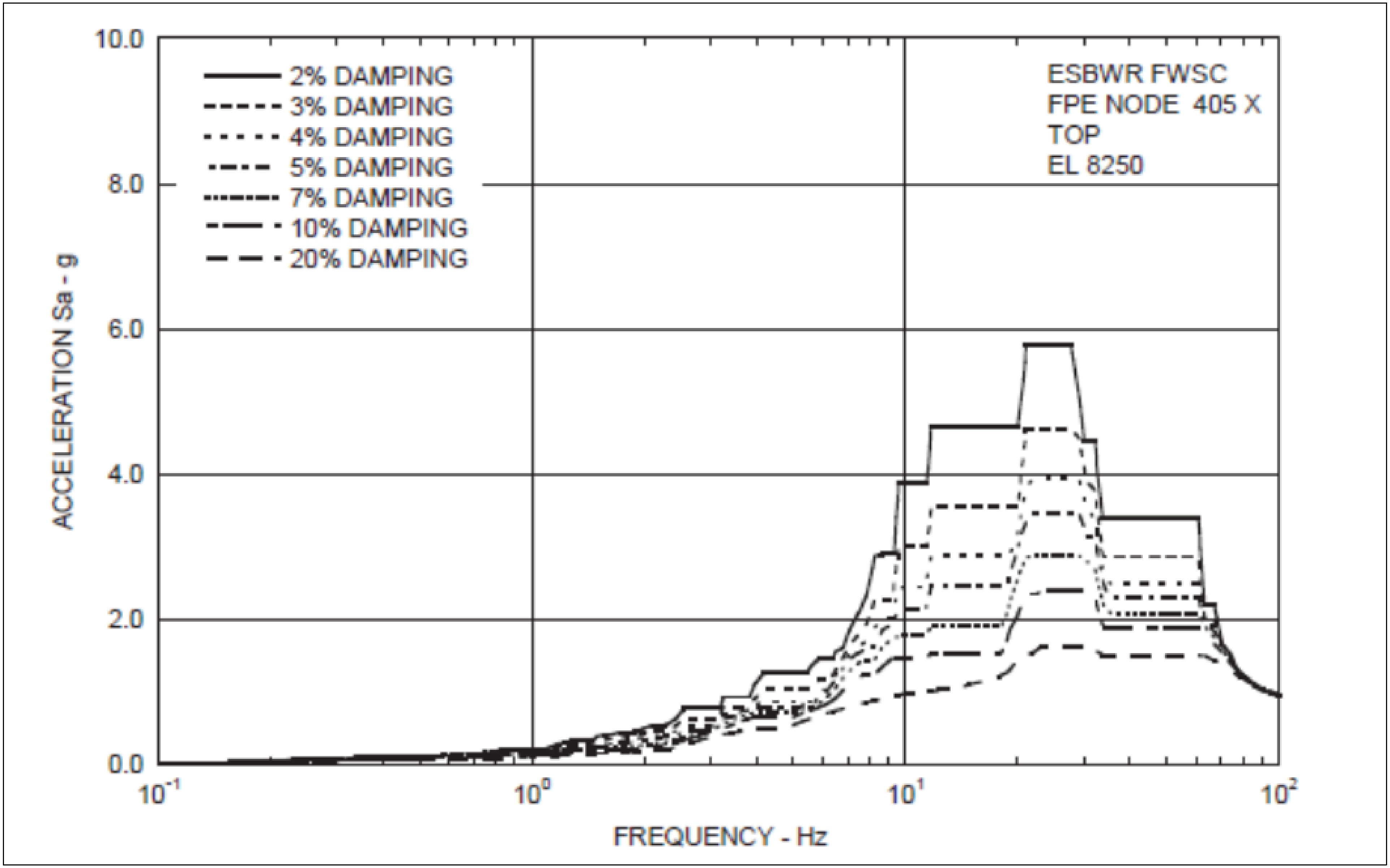


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Figure 3A.18.2-201I **Site-Specific In-Structure Response Spectra - FPE Node 404 X**

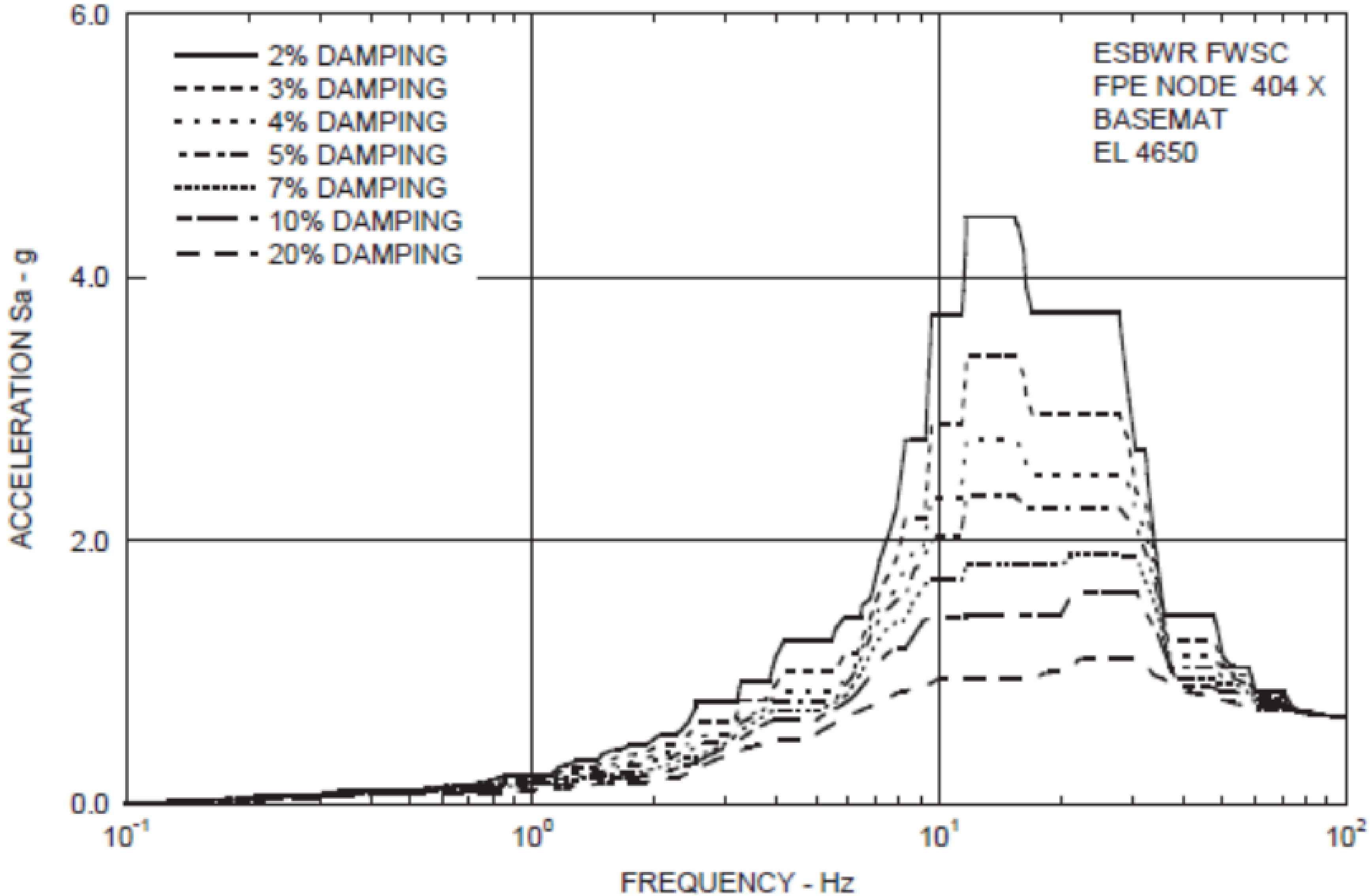

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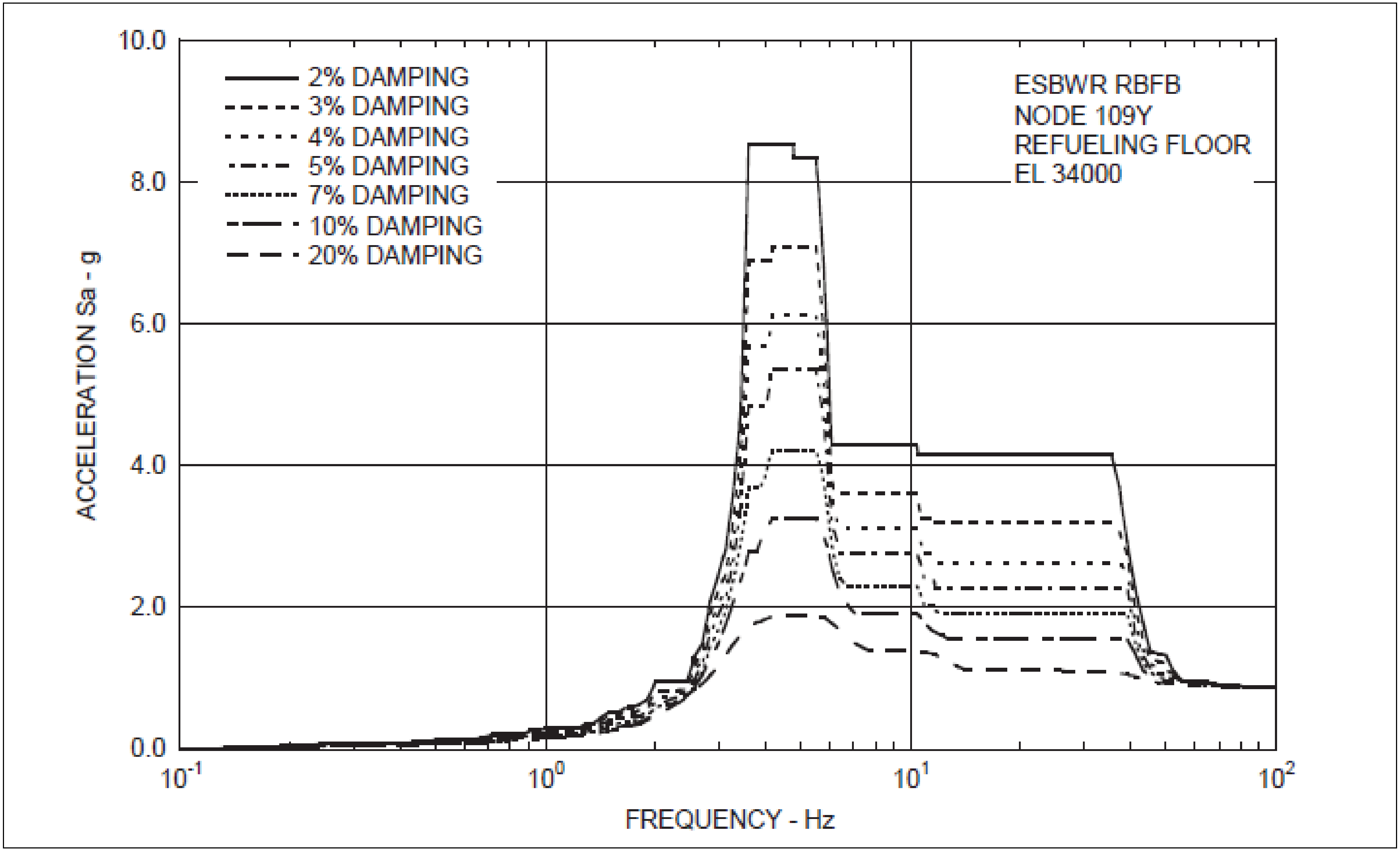
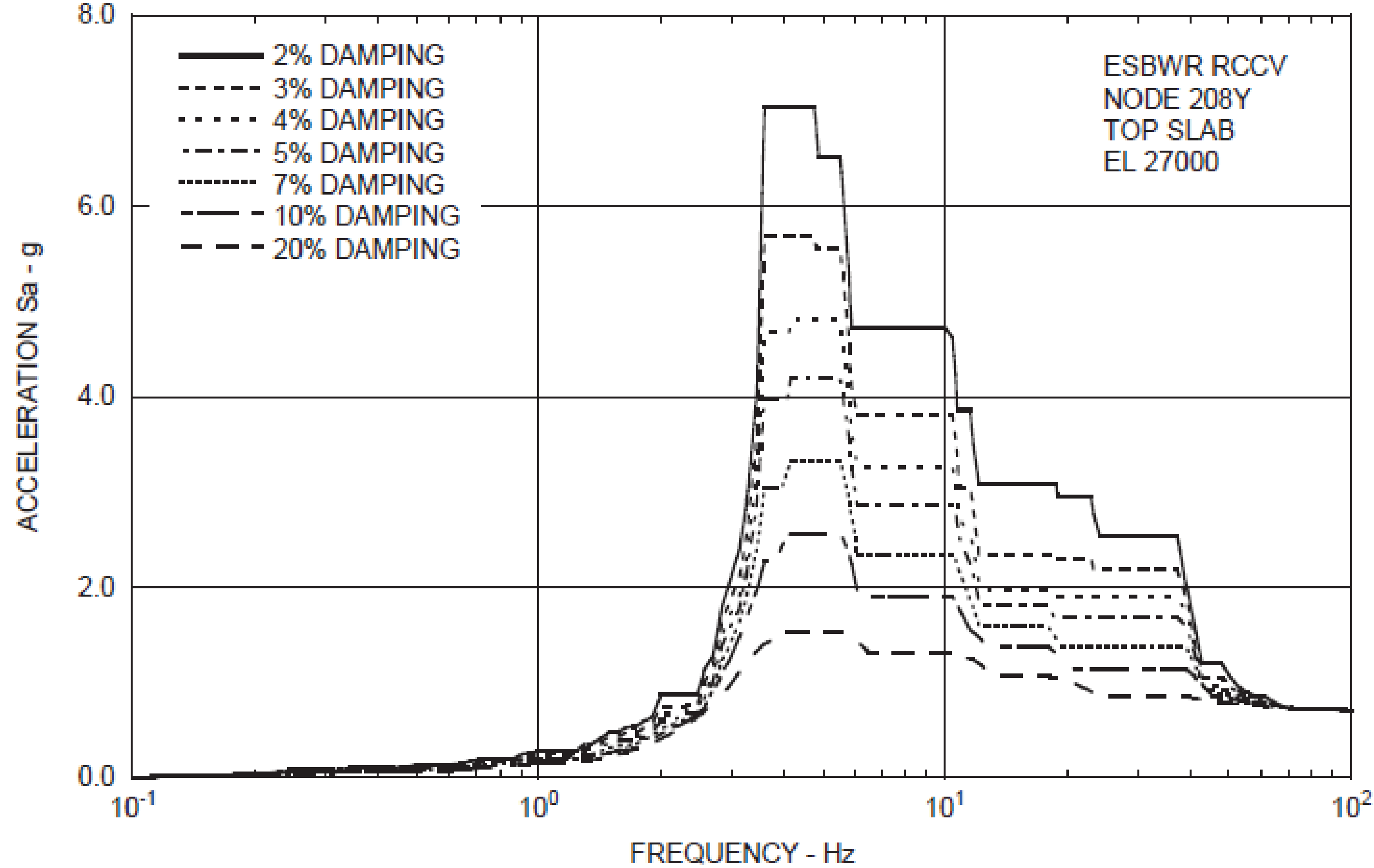
Figure 3A.18.2-202a Site-Specific In-Structure Response Spectra - RB/FB Node 109 Y



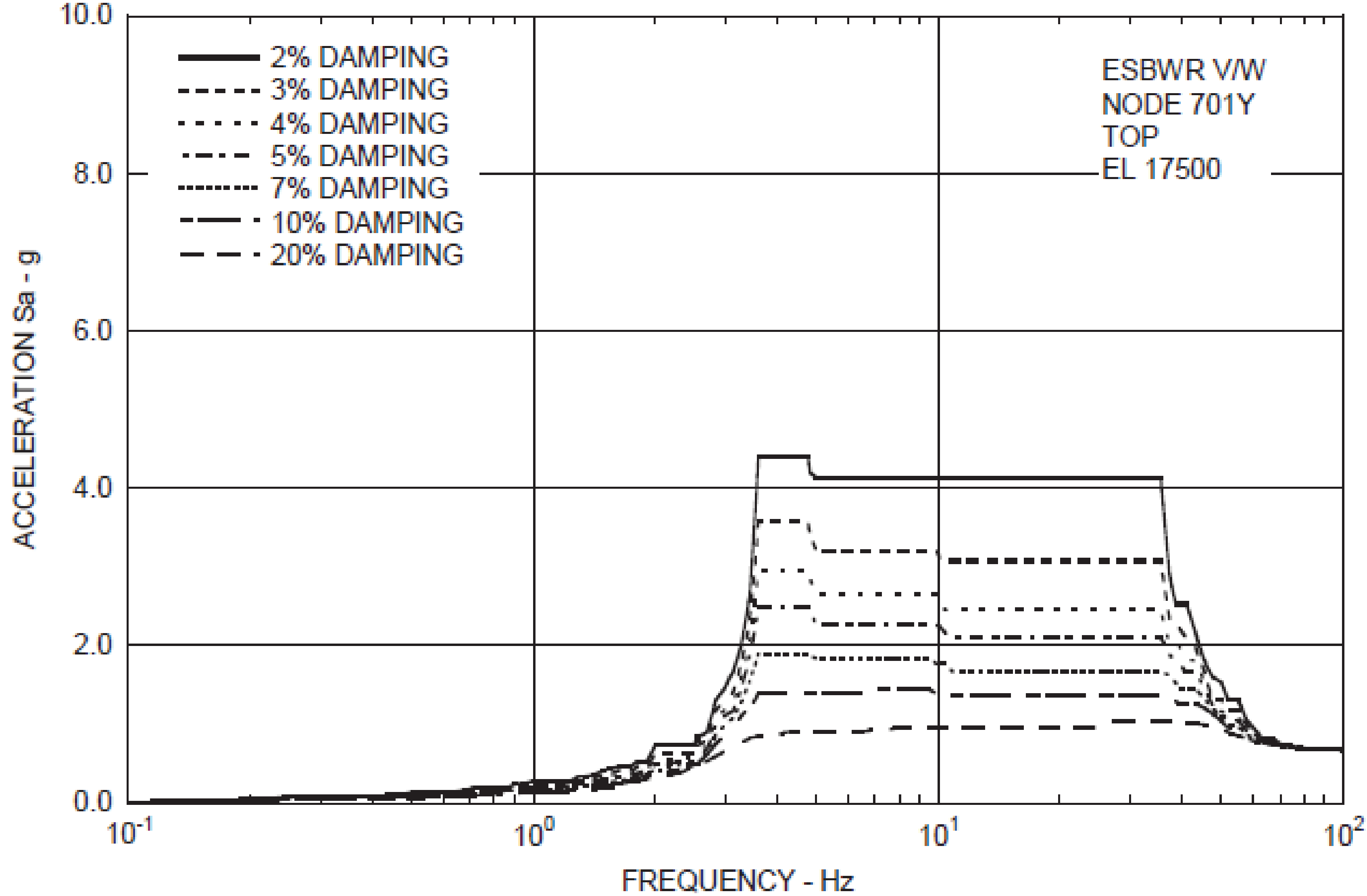
NAPS DEP 3.7-1

Figure 3A.18.2-202b Site-Specific In-Structure Response Spectra - RCCV Node 208 Y



NAPS DEP 3.7-1

Figure 3A.18.2-202c **Site-Specific In-Structure Response Spectra - Vent Wall Node 701 Y**



NAPS DEP 3.7-1

Figure 3A.18.2-202d **Site-Specific In-Structure Response Spectra - RSW Node 707 Y**

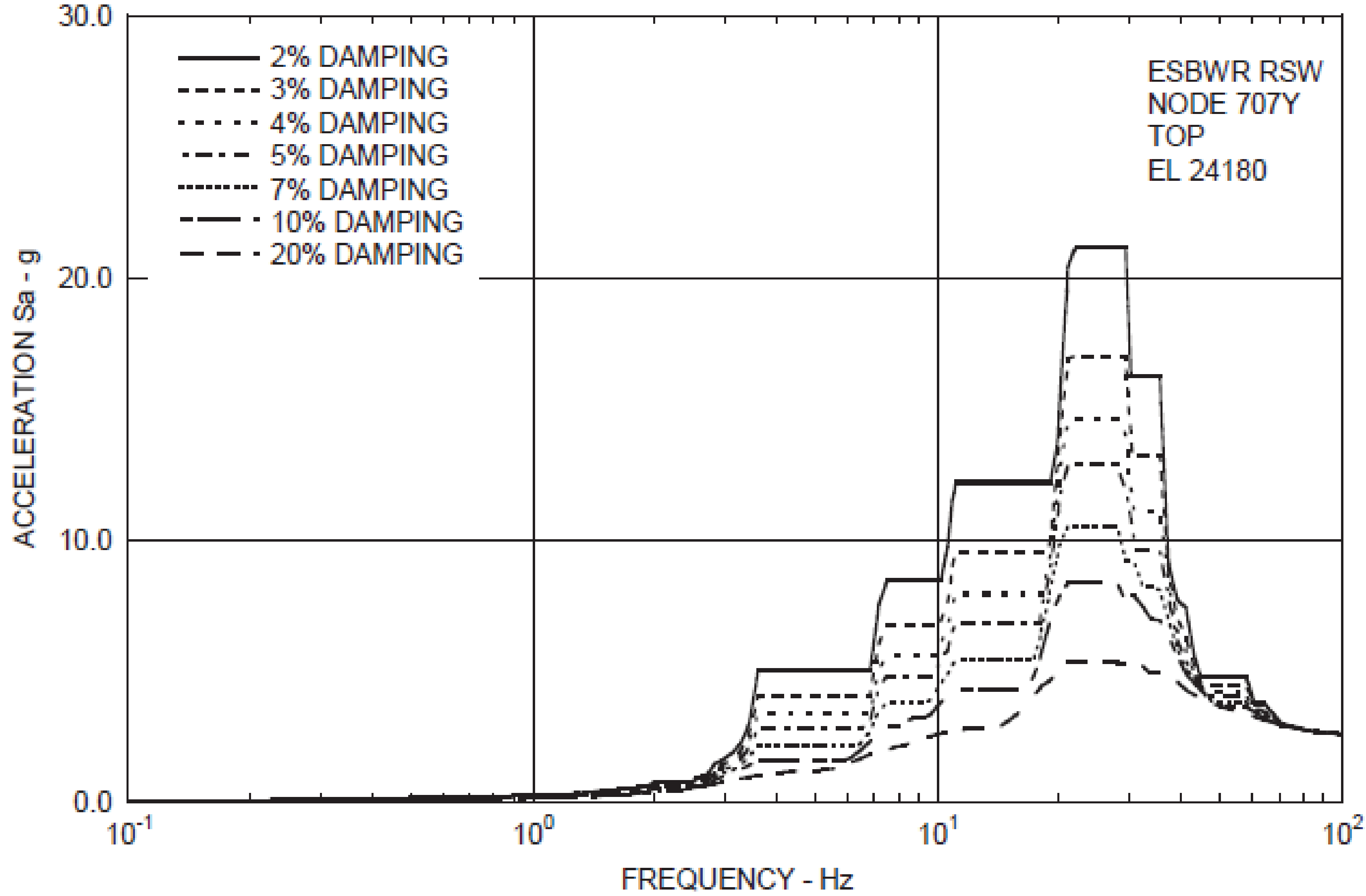


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Figure 3A.18.2-202e Site-Specific In-Structure Response Spectra - RPV Node 801 Y

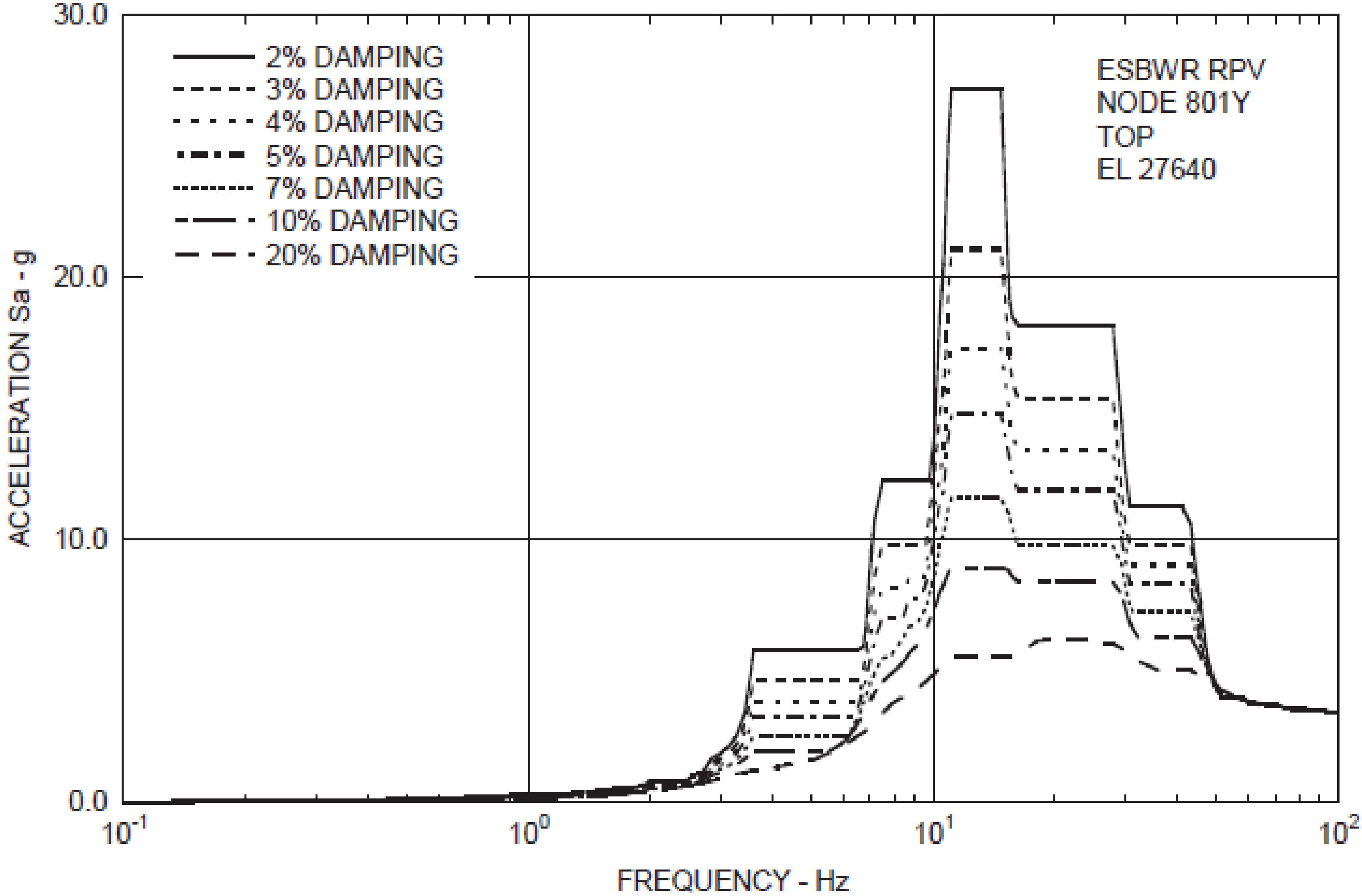

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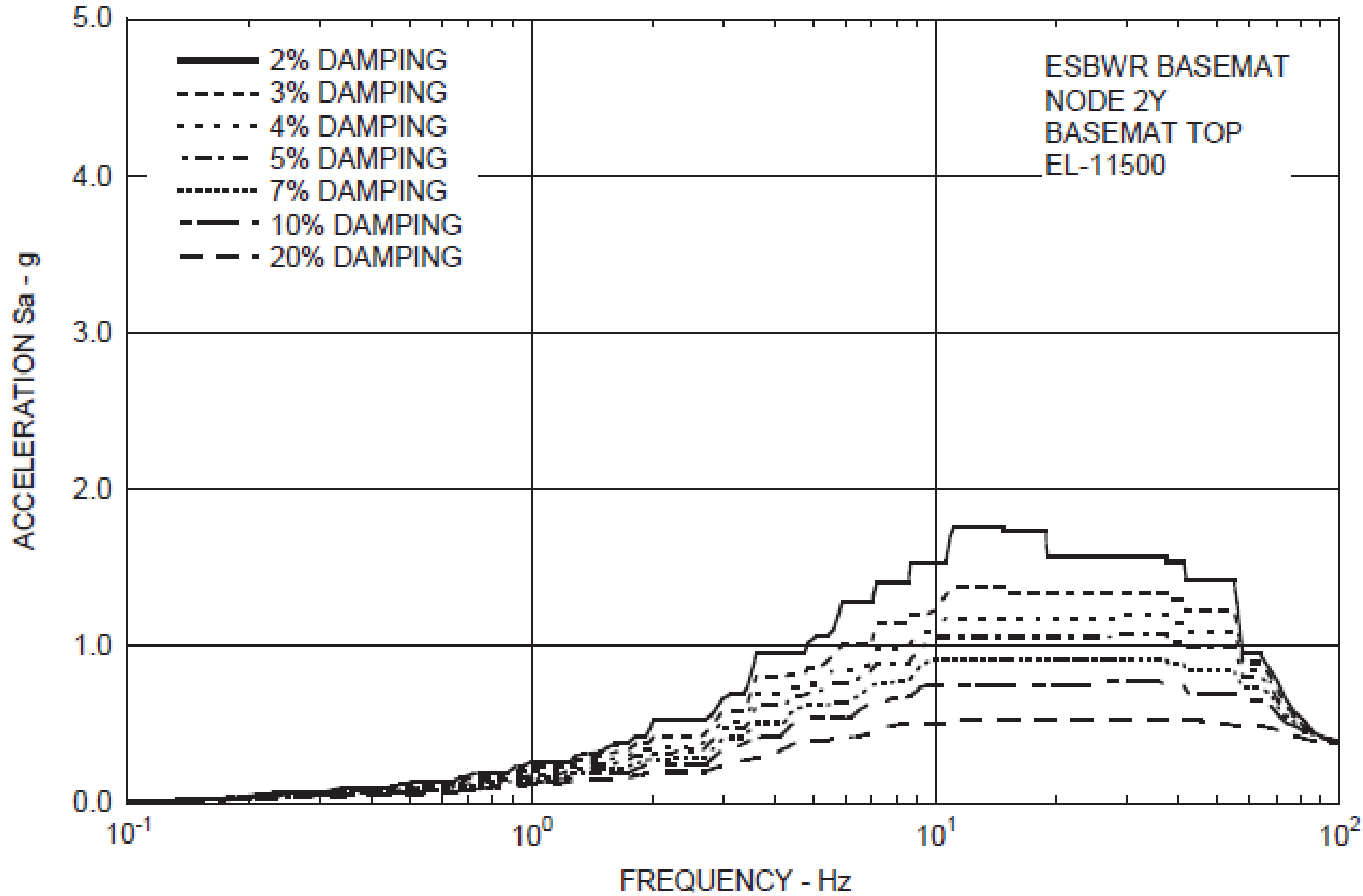
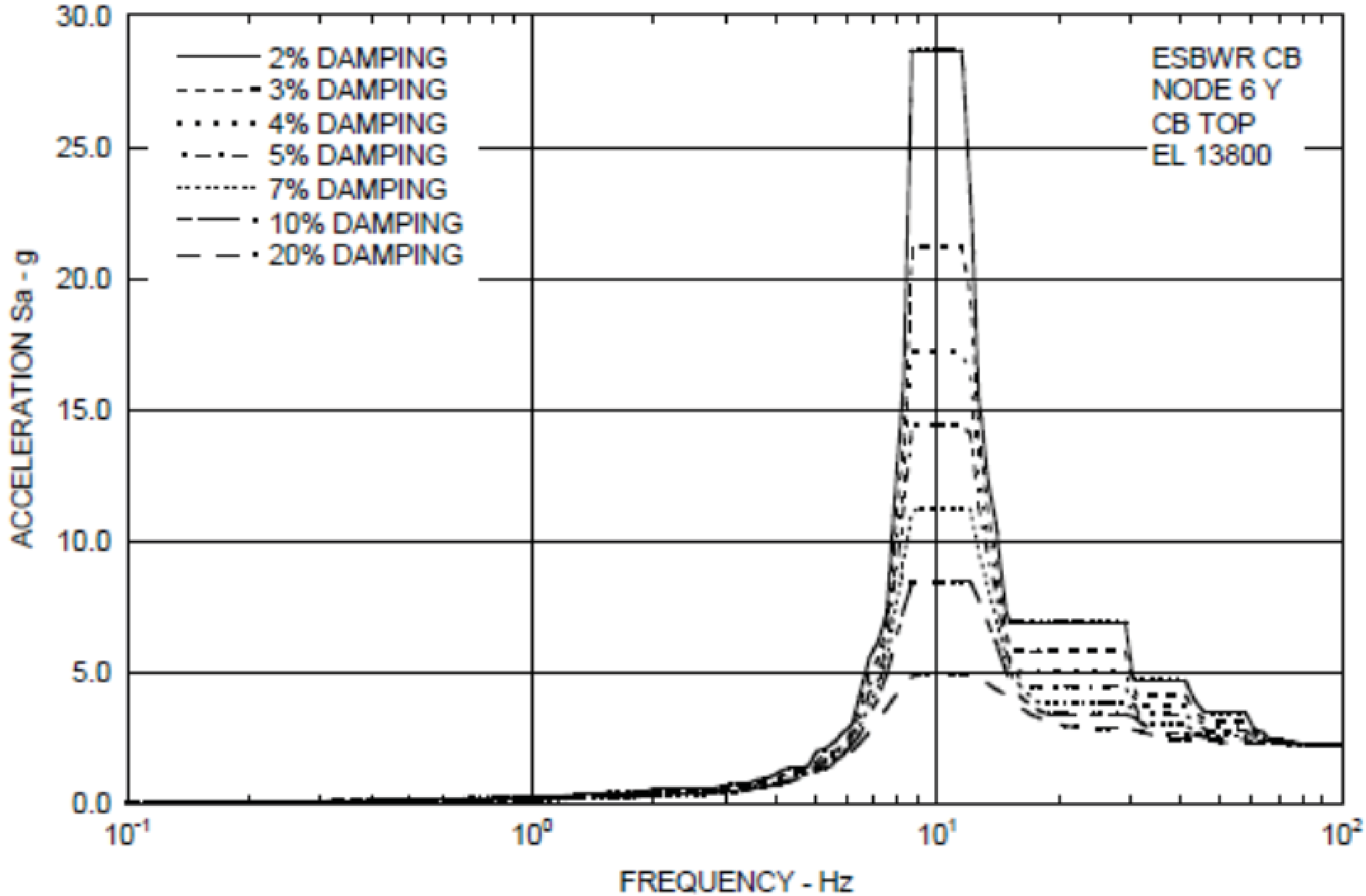
Figure 3A.18.2-202f Site-Specific In-Structure Response Spectra - RB/FB Node 2Y



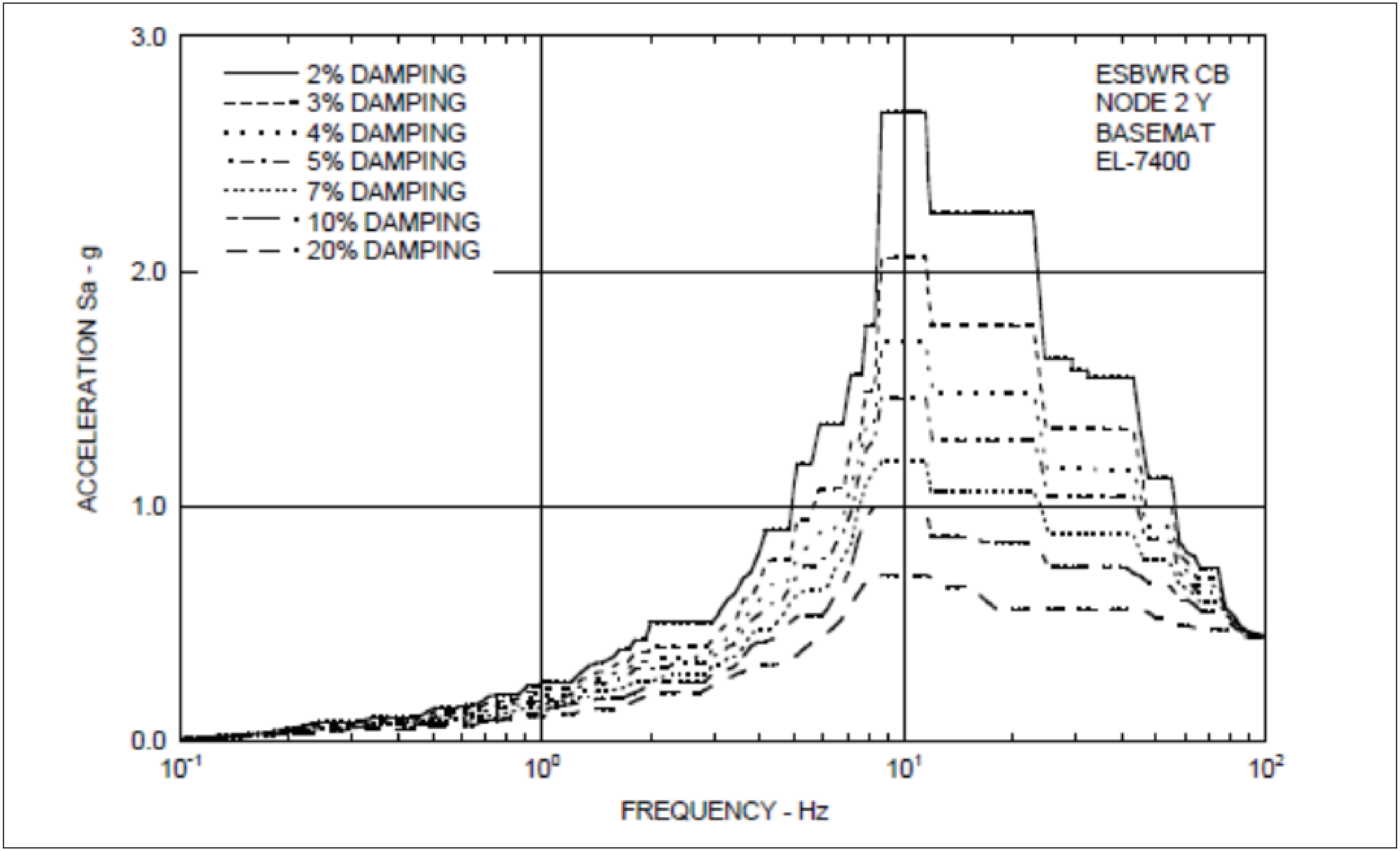
NAPS DEP 3.7-1

Figure 3A.18.2-202g Site-Specific In-Structure Response Spectra - CB Node 6 Y



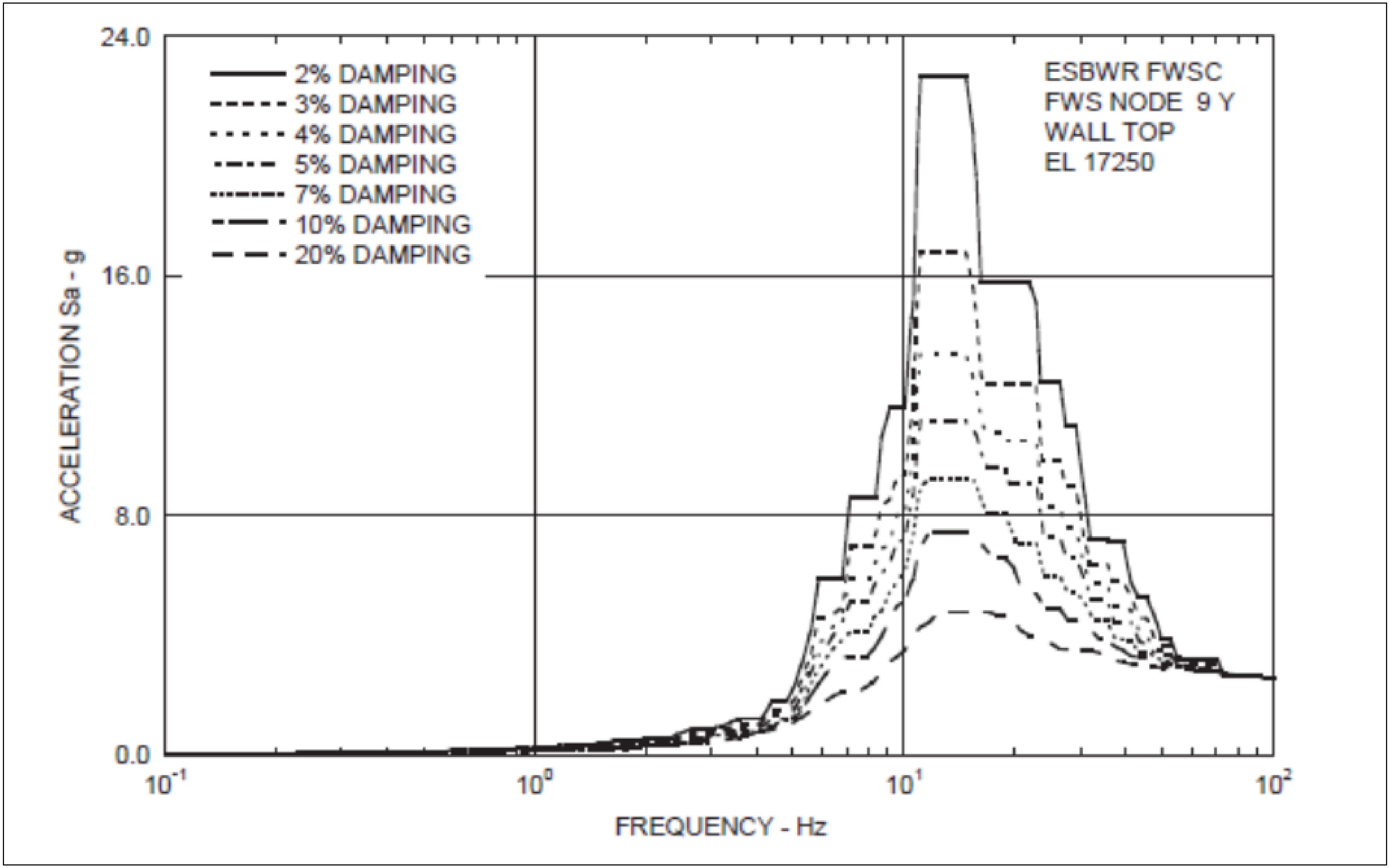
NAPS DEP 3.7-1

Figure 3A.18.2-202h Site-Specific In-Structure Response Spectra - CB Node 2 Y

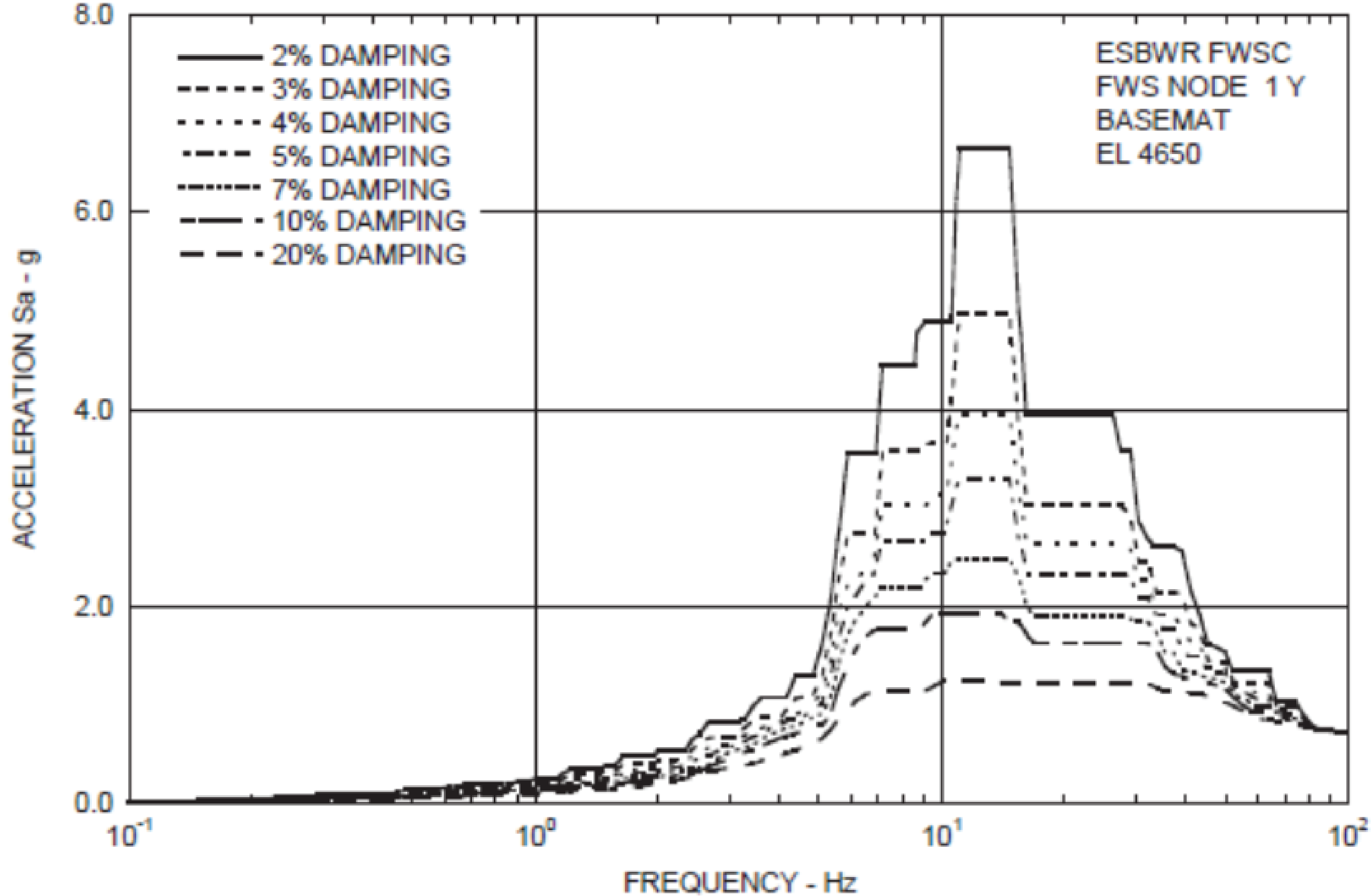


NAPS DEP 3.7-1

Figure 3A.18.2-202i Site-Specific In-Structure Response Spectra - FWS Node 9 Y

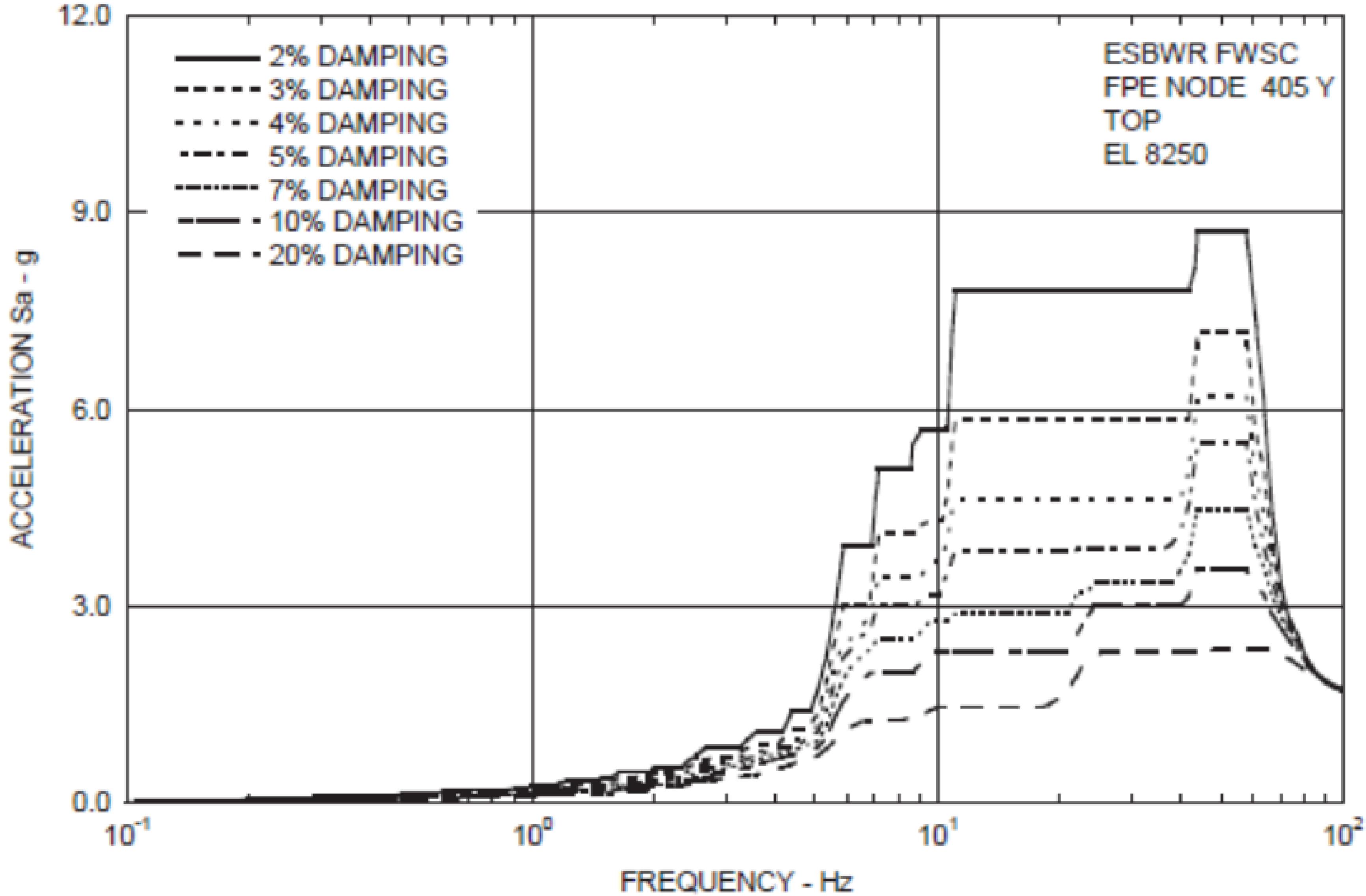


NAPS DEP 3.7-1 Figure 3A.18.2-202j **Site-Specific In-Structure Response Spectra - FWS Node 1 Y**



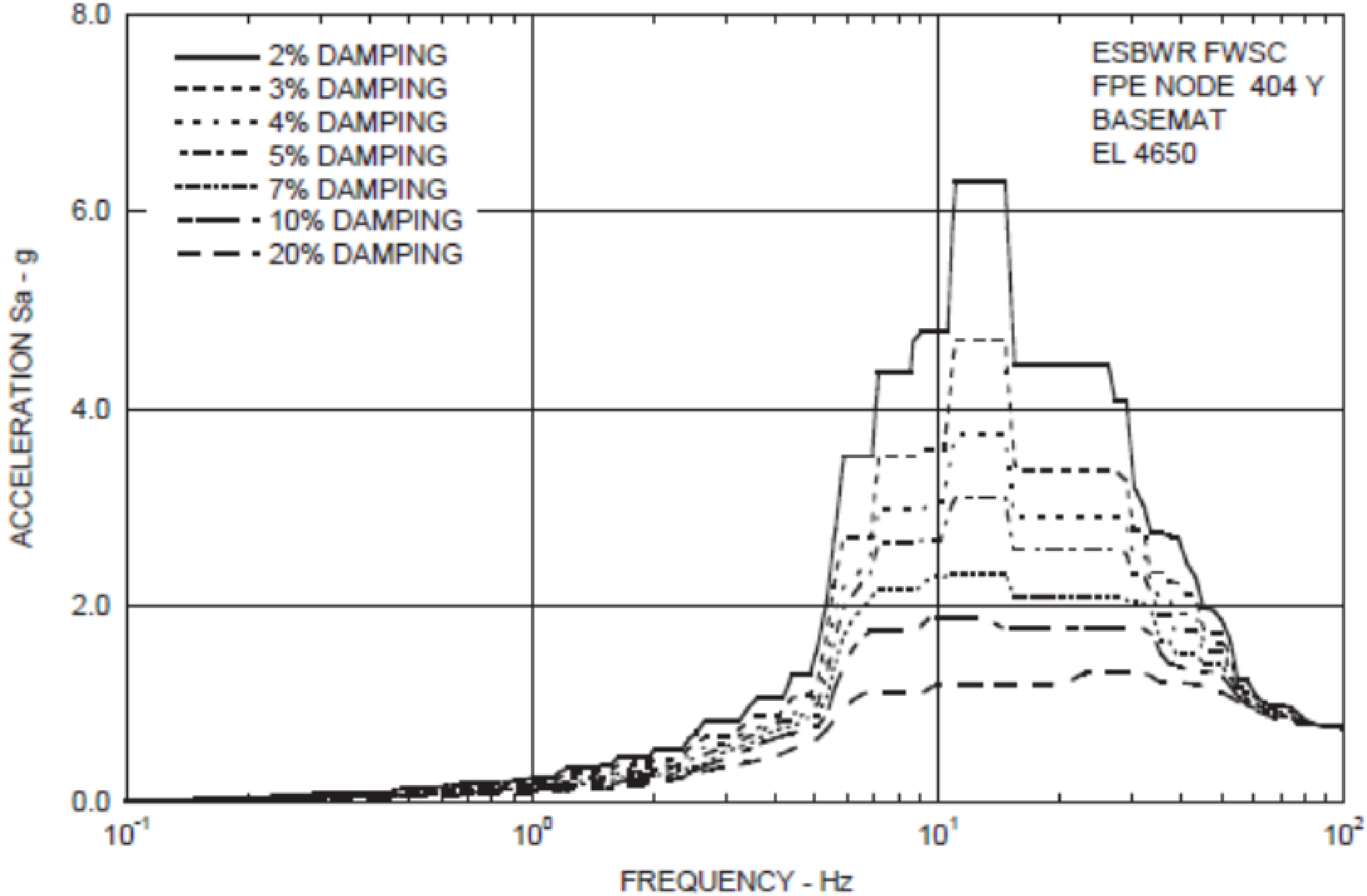
NAPS DEP 3.7-1

Figure 3A.18.2-202k **Site-Specific In-Structure Response Spectra - FPE Node 405 Y**



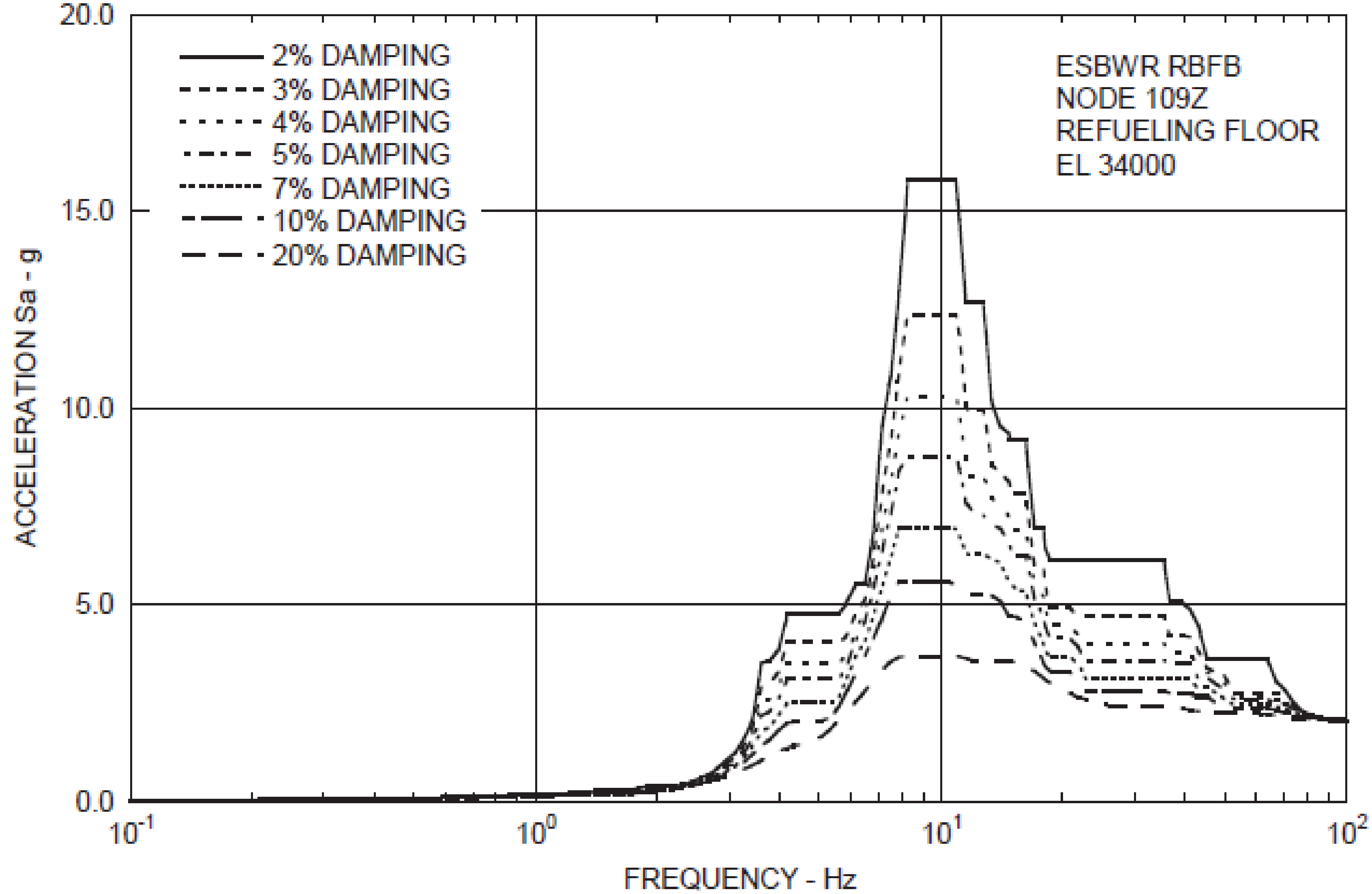
NAPS DEP 3.7-1

Figure 3A.18.2-202I **Site-Specific In-Structure Response Spectra - FPE Node 404 Y**



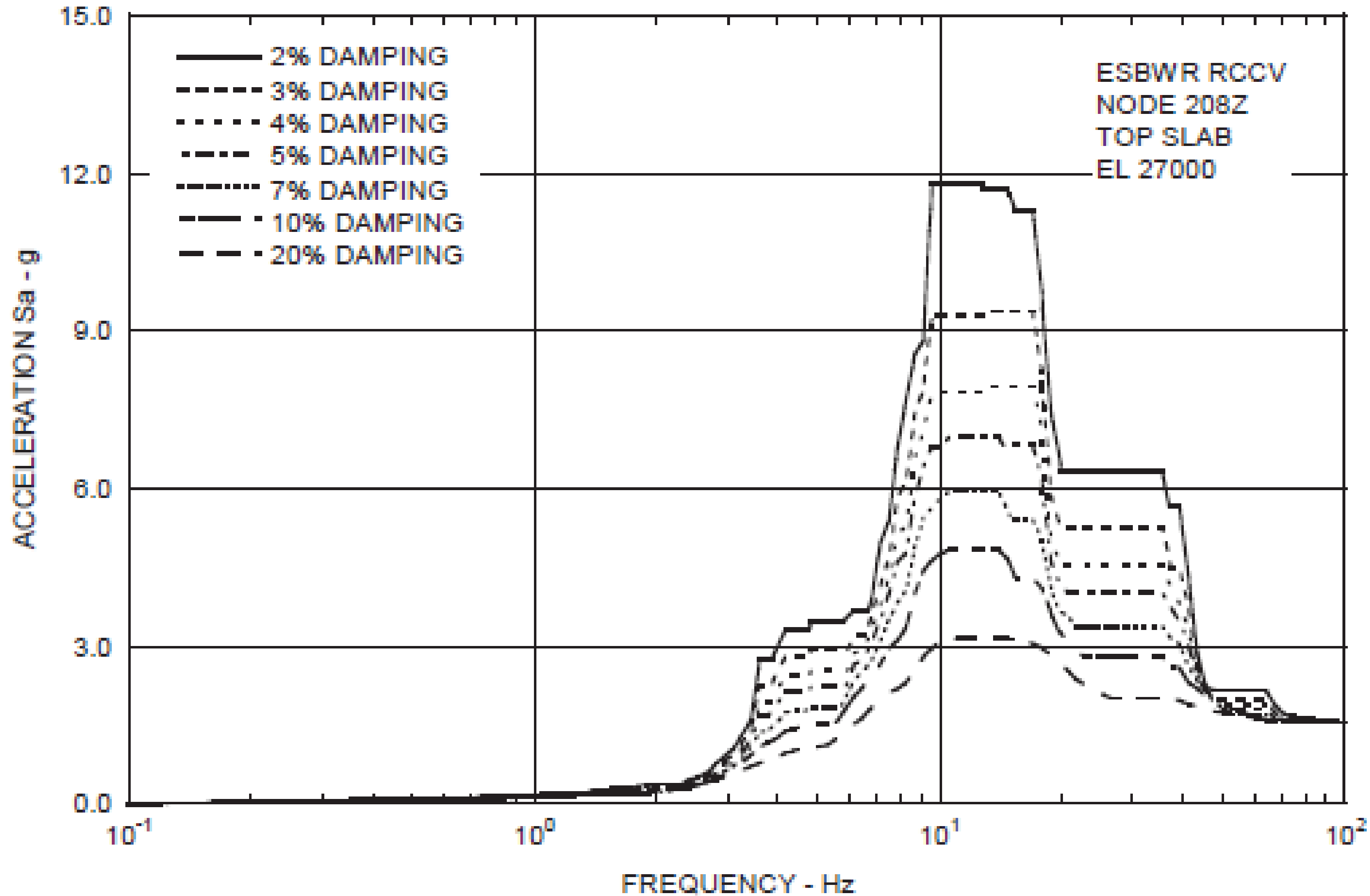
NAPS DEP 3.7-1

Figure 3A.18.2-203a **Site-Specific In-Structure Response Spectra - RB/FB Node 109 Z**



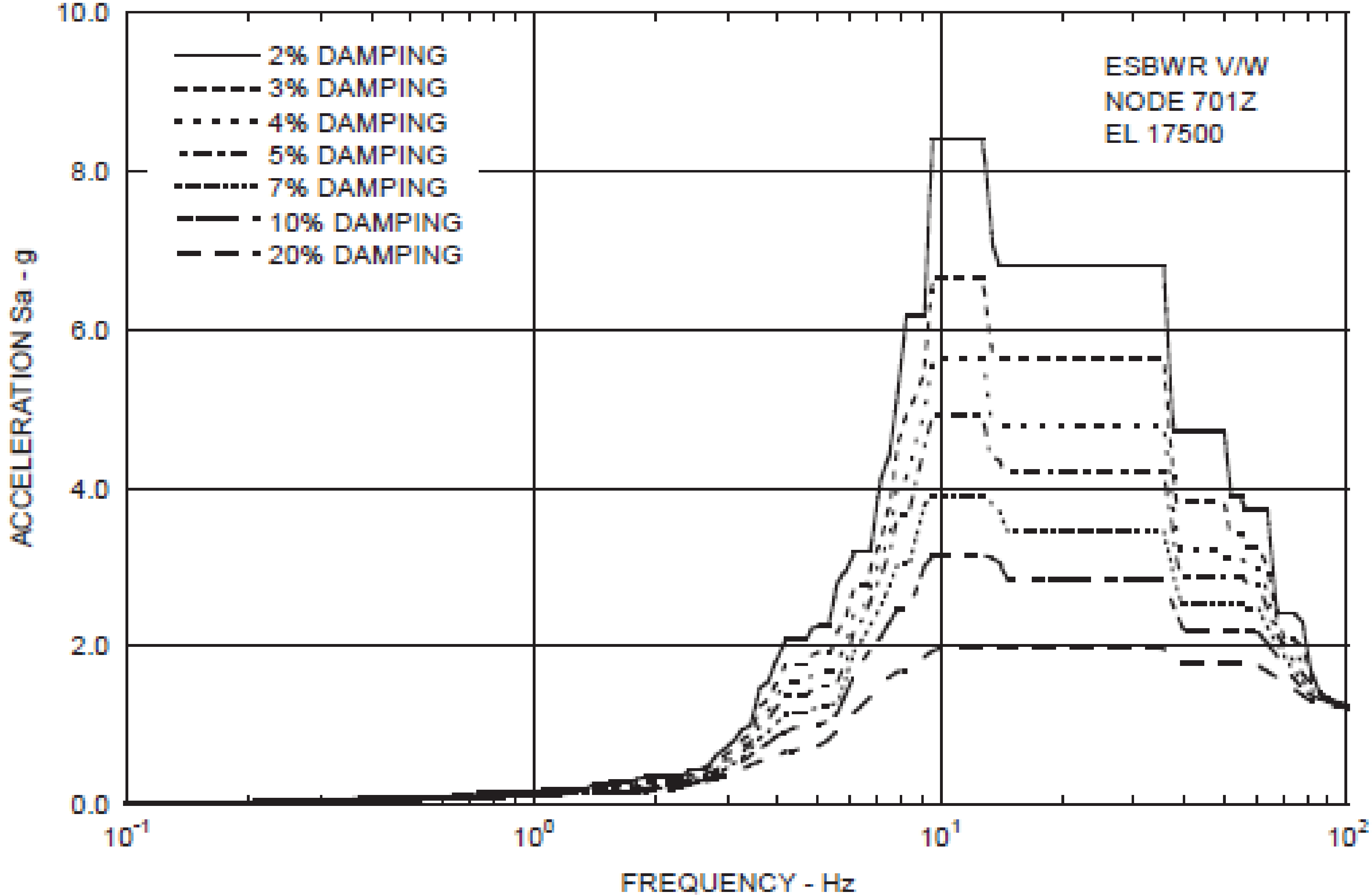
NAPS DEP 3.7-1

Figure 3A.18.2-203b Site-Specific In-Structure Response Spectra - RCCV Node 208 Z



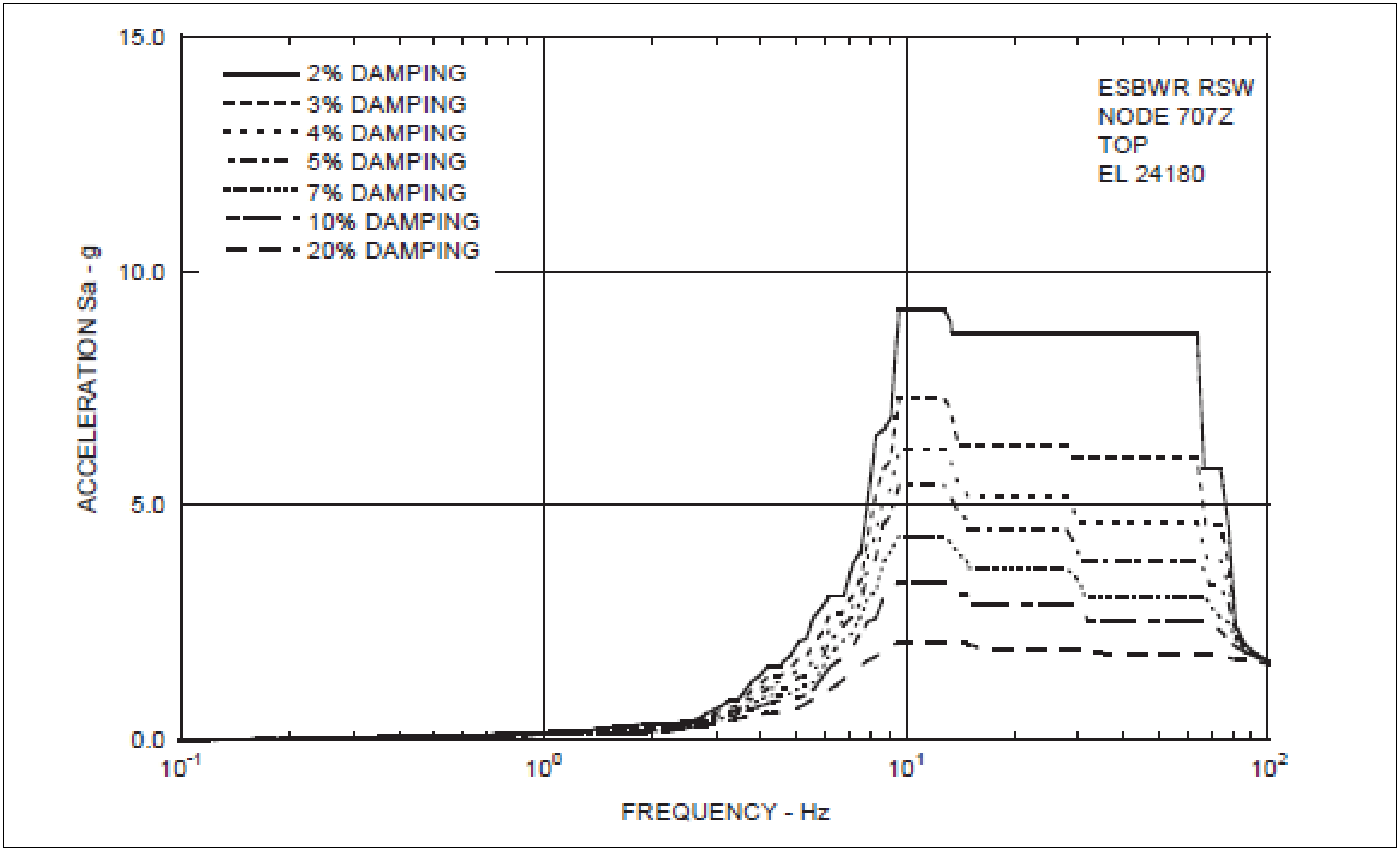
NAPS DEP 3.7-1

Figure 3A.18.2-203c **Site-Specific In-Structure Response Spectra - Vent Wall Node 701 Z**



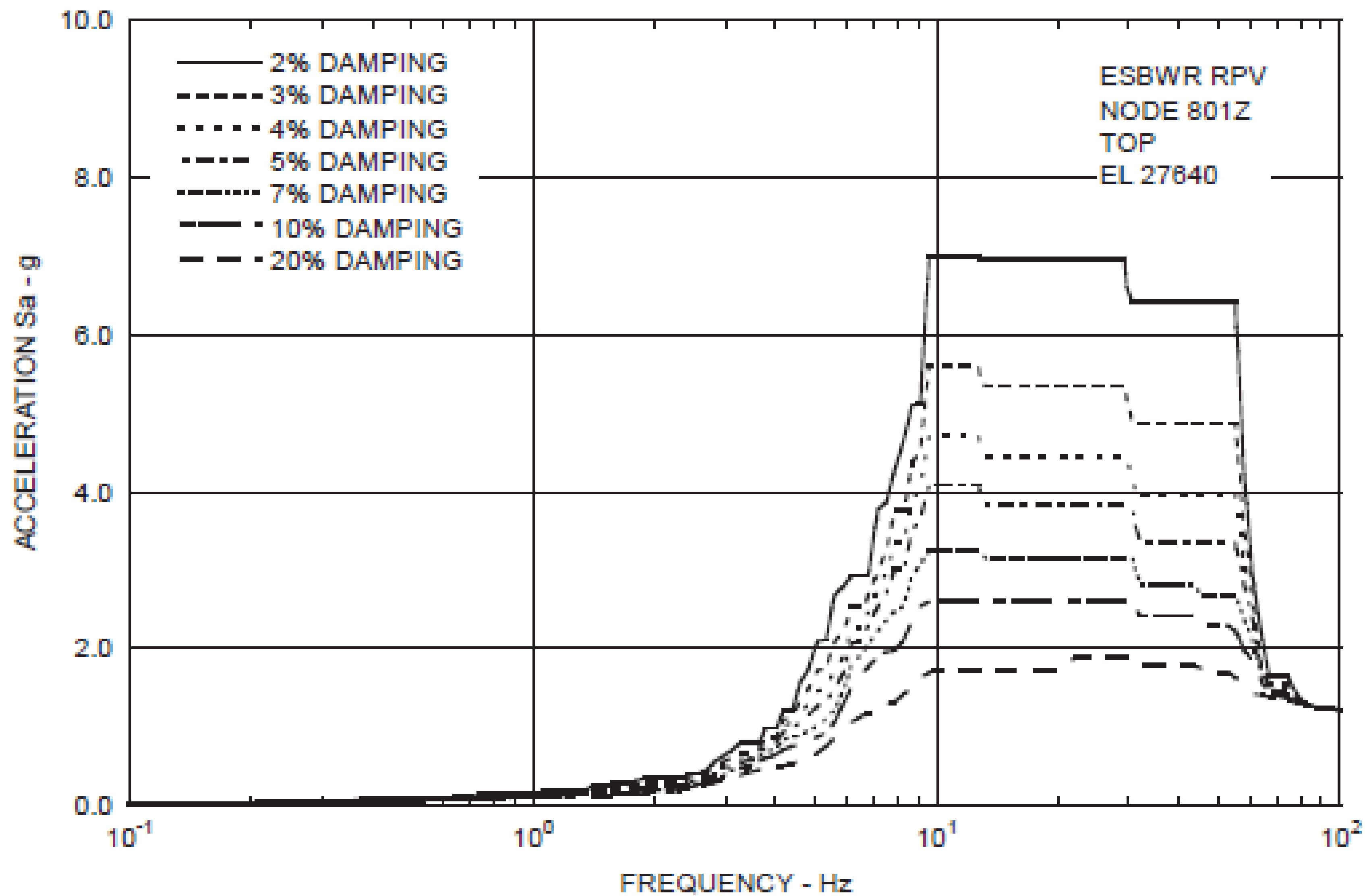
NAPS DEP 3.7-1

Figure 3A.18.2-203d **Site-Specific In-Structure Response Spectra - RSW Node 707 Z**

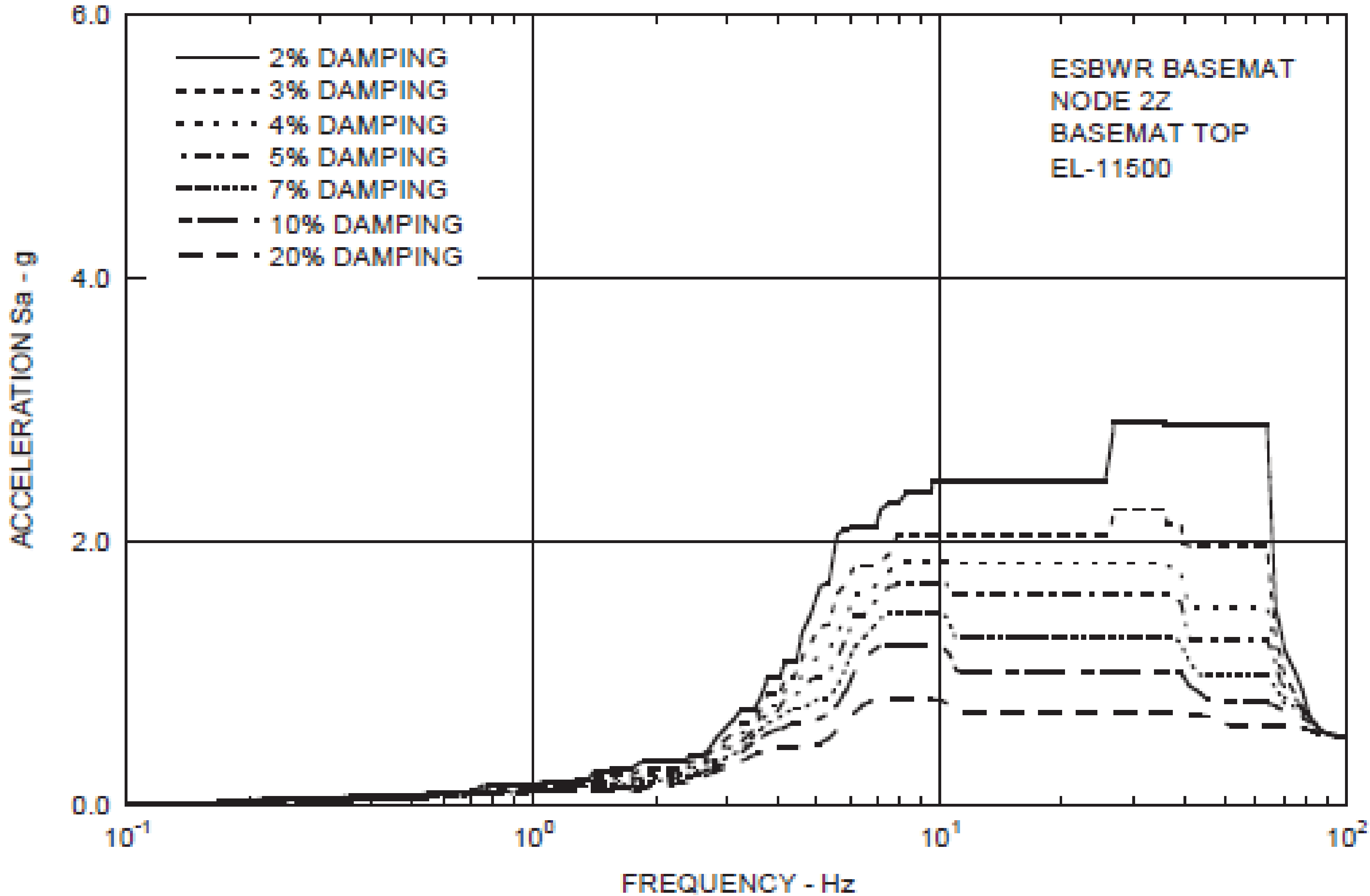


NAPS DEP 3.7-1

Figure 3A.18.2-203e **Site-Specific In-Structure Response Spectra - RPV Node 801 Z**

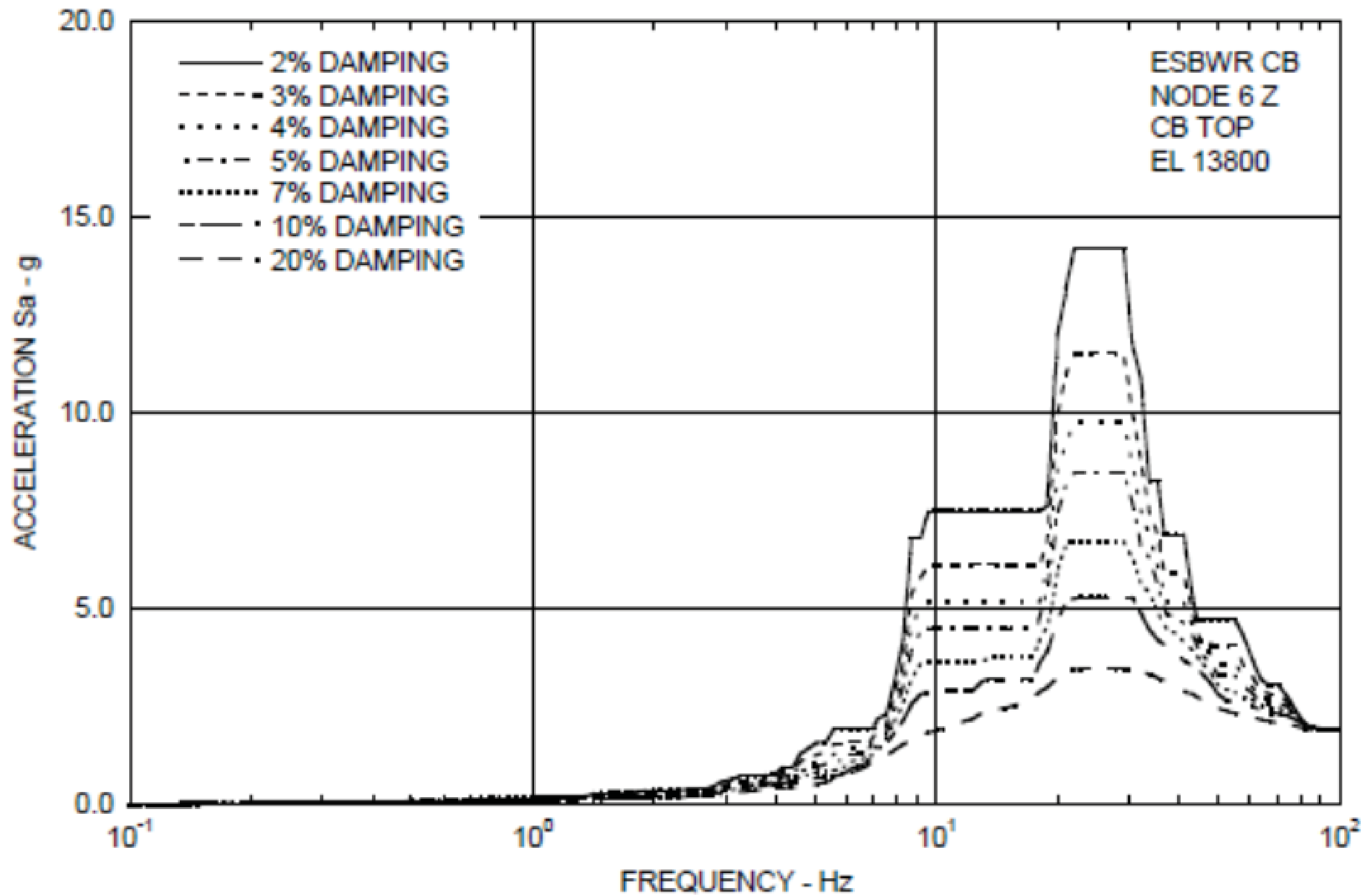


NAPS DEP 3.7-1 Figure 3A.18.2-203f **Site-Specific In-Structure Response Spectra - RB/FB Node 2Z**



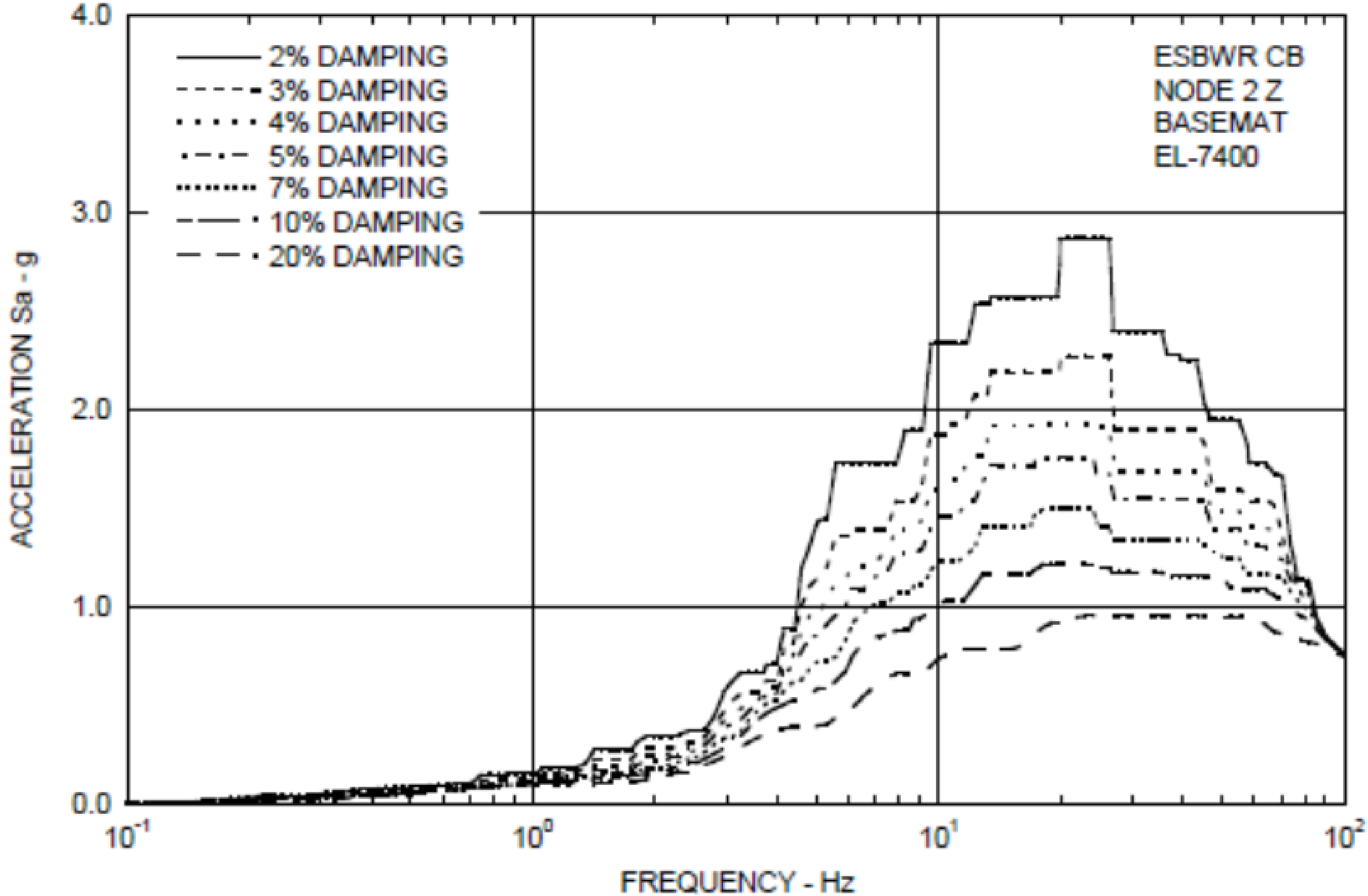
NAPS DEP 3.7-1

Figure 3A.18.2-203g **Site-Specific In-Structure Response Spectra - CB Node 6 Z**



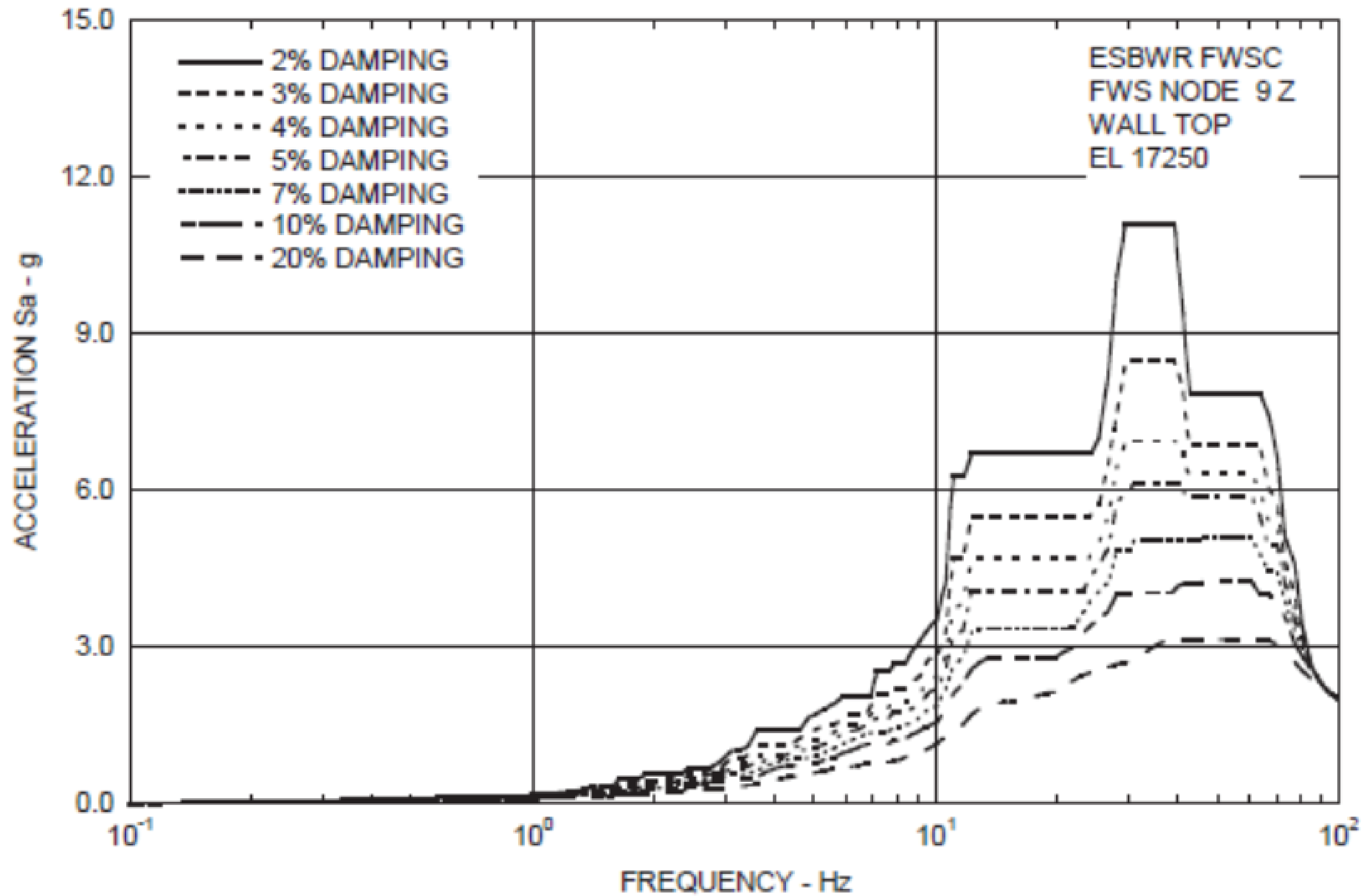
NAPS DEP 3.7-1

Figure 3A.18.2-203h **Site-Specific In-Structure Response Spectra - CB Node 2 Z**



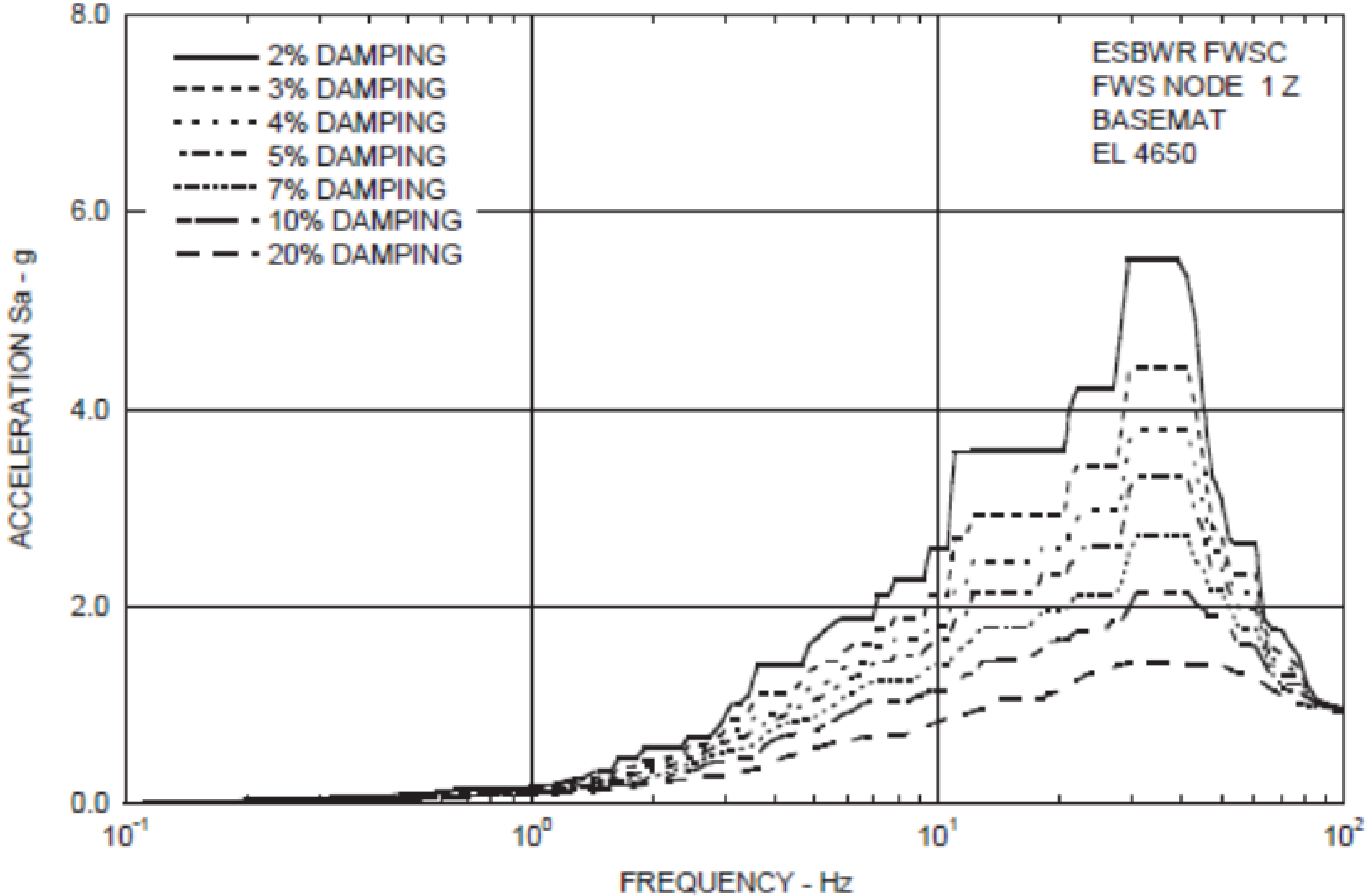
NAPS DEP 3.7-1

Figure 3A.18.2-203i **Site-Specific In-Structure Response Spectra - FWS Node 9 Z**

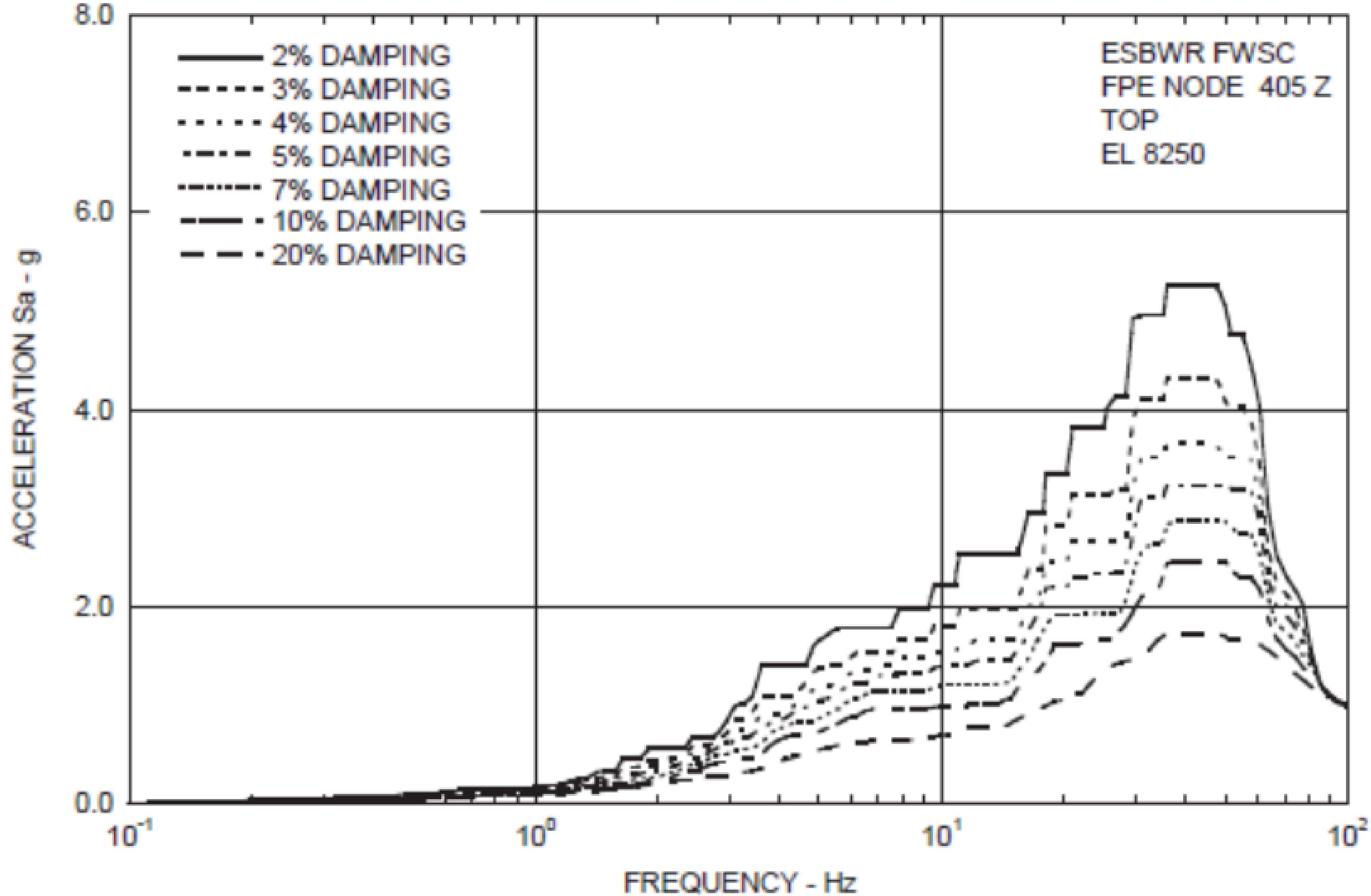


NAPS DEP 3.7-1

Figure 3A.18.2-203j **Site-Specific In-Structure Response Spectra - FWS Node 1 Z**

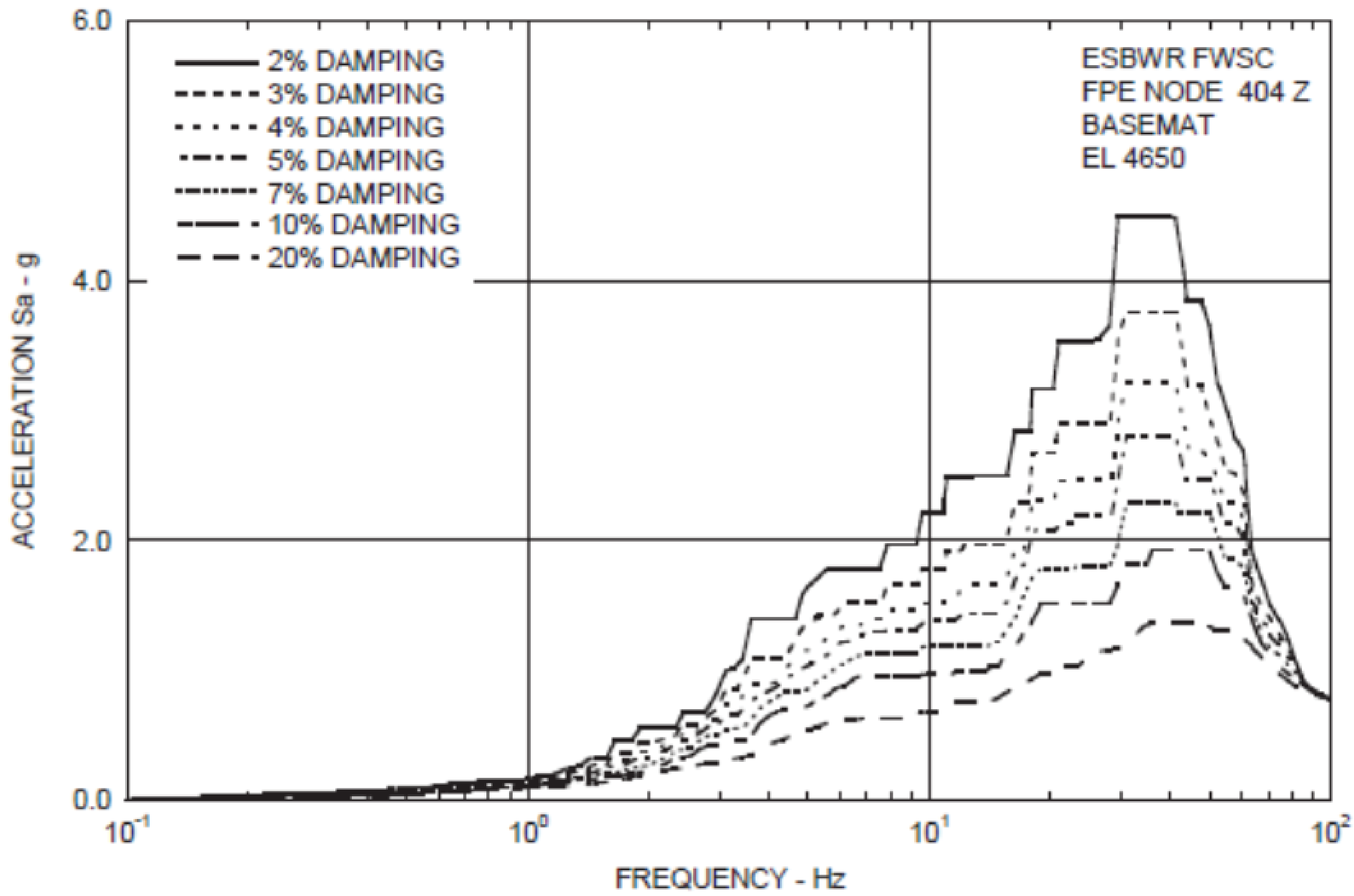


NAPS DEP 3.7-1 Figure 3A.18.2-203k **Site-Specific In-Structure Response Spectra - FPE Node 405 Z**



NAPS DEP 3.7-1

Figure 3A.18.2-203l **Site-Specific In-Structure Response Spectra - FPE Node 404 Z**



3A.18.3 Basemat Interface Loads with Foundation Medium for Foundation Stability Evaluation

Site-specific foundation stability evaluations presented in Section 3.8.5 use the results of the SSI analyses for contact spring forces to calculate time histories of the seismic driving forces. These seismic driving forces are used as input for the sliding stability evaluations and dynamic bearing pressure calculations. The time histories of the horizontal and vertical driving seismic forces in the three orthogonal directions are calculated as the algebraic sum of the spring forces in the three directions at each time step from all contact spring elements at the interfaces between the structure and the surrounding soil.

The contact spring elements at the bottom of the RB/FB basemat provide the input needed for the calculation of the dynamic bearing pressures. Contact spring elements at the bottom of the CB basemat and at the bottom of the concrete fill supporting the CB provide the input needed for calculating the dynamic bearing pressures from the CB on the concrete fill and the Zone III/IV rock. Contact spring elements at the bottom of the FWSC basemat and at the bottom of the concrete fill supporting the FWSC basemat provide the input needed for calculating the dynamic bearing pressures from the FWSC on the concrete fill and the Zone III/IV rock. The calculations of the dynamic bearing pressures are based on the time histories of:

- The overturning base moments calculated for both the horizontal directions by summing algebraically the moments generated by each contact spring reaction at the bottom of the basemat
- The vertical driving seismic forces calculated as the algebraic sum of the vertical spring forces at each time step from all contact spring elements at the interface between the bottom of the basemat and the rock

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3A.19 Unit 3 Site-Specific Seismic Analyses Conclusions

The site-specific seismic analyses of Seismic Category I buildings provide enveloping site-specific seismic demands that are compared to the corresponding enveloping loads used for the standard design of SSCs. The comparisons serve to determine exceedances resulting from the site-specific GMRS that exceed the standard design CSDRS. The enveloping results of the site-specific analyses also form the basis for the site-specific evaluations of Seismic Category I SSCs presented in Appendix 3G.

3A.19.1 **Reactor Building/Fuel Building**

Unit 3 site-specific RB/FB SSI analyses indicate the following:

- Site-specific seismic demands exceed the seismic loads used for the standard design of the RB/FB structures. Site-specific evaluations of RB/FB structures use input seismic loads that are based on the site-specific enveloping loads described in Section 3A.18.1.1 and bound the effects of structural stiffness variations described in Section 3A.17.9.1.
- Site-specific ISRS are developed for all locations of the RB/FB to serve as input for the site-specific seismic analyses and qualification of RB/FB equipment and components as described in Section 3A.18.2. Site-specific evaluations of the equipment and components address effects of structural stiffness variations described in Section 3A.17.9.1.

3A.19.2 **Control Building**

Unit 3 site-specific CB SSI analyses indicate the following:

- Site-specific seismic demands exceed the seismic loads used for the standard design of the CB structures. Site-specific evaluations of CB structures use input seismic loads that are based on the site-specific enveloping loads described in Section 3A.18.1.2 and bound the effects of structural stiffness variation as described in Section 3A.17.9.2.
- Site-specific ISRS are developed for all locations of the CB to serve as input for the site-specific seismic analysis and qualification of CB equipment and components as described in Section 3A.18.2. Site-specific evaluations of the equipment and components address effects of structural stiffness variations described in Section 3A.17.9.2.

3A.19.3 **Fire Water Service Complex**

Unit 3 site-specific FWSC SSI analyses indicate the following:

- Site-specific seismic demands slightly exceed the seismic loads used for the standard design of the FWSC structures. Site-specific evaluations of FWSC structures use input seismic loads that are based on the site-specific enveloping loads described in Section 3A.18.1.3 and include the site-specific SSSI effects of the CB

on the FWSC seismic response. The enveloping loads are amplified to address effects of concrete cracking described in Section 3A.17.9.3.

- Site-specific ISRS are developed to serve as input for the site-specific seismic analysis and qualification of the FWSC equipment and components as described in Section 3A.18.2. These ISRS envelope the site-specific SSSI effects of the CB on the FWSC seismic response. Site-specific evaluations of equipment and components supported on the FPE roof consider ISRS that include the effects of structural concrete cracking as described in Section 3A.17.9.3.

3A.19.4 **SSSI Effects**

Results of the site-specific evaluations of the effects of SSSI, which are described in Section 3A.17.11, show that the SSSI between the FWSC, CB, and RB/FB have small effects on the site-specific seismic responses of these Seismic Category I structures. The enveloping results of the site-specific SSI analyses of CB stand-alone models, which are used for the site-specific evaluations of the CB SSCs in Appendix 3G, bound the effects of SSSI on the CB. However, to bound the amplifications of the FWSC response that are due to the SSSI with the CB, the seismic design basis that is used for the FWSC site-specific evaluations in Appendix 3G is developed as the envelope of the results of the SSI analysis of the FWSC stand-alone model and the SSSI analyses of the FWSC-CB combined model.

Appendix 3B Containment Hydrodynamic Load Definitions

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3C Computer Programs Used in the Design and Analysis of Seismic Category I Structures

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add the following at the end of Appendix 3C.

~~NAPS CDI~~
NAPS DEP 3.7-1

3C.7.4 Site-Specific Dynamic Soil-Structure Interaction Analysis Program – SASSI2010

3C.7.4.1 Description

SASSI2010 is used to solve a wide range of dynamic SSI problems in two or three dimensions.

3C.7.4.2 Validation

SASSI version 2010 was obtained from Isatis LLC under the contract with the Regents of the University of California and implemented by Shimizu Corporation of Tokyo, Japan on HP Z420 Workstation computer using Windows 7 OS. Program validation documentation is available at Shimizu Corporation.

The validation test report documents details of the SASSI2010 verification and validation of all of the SASSI2010 program modules used in the Unit 3 SSI analyses with passing frequencies up to ~~50 Hz~~ 70 Hz. The verification and validation are performed in accordance with the Shimizu Quality Assurance Program. The report describes the methodology used to perform the verification and validation; provides the acceptance criteria; lists the capabilities, options, and limitations that are verified and validated; and provides the details and results of the validation problems. Verification methods include comparison with classical solutions, analytical results or experimental test data; comparisons with results from other software; and comparisons of results of various analysis problems. Acceptance criteria include numerical accuracy, good numerical agreement, and expected behavior. The process confirms that the SASSI2010 computer program is adequate for the 3D seismic response analyses of the Unit 3 SSI systems.

3C.7.5 Free-Field Site Response Analysis – P-SHAKE

A model comparable to the free-field site response analysis SHAKE method described in the DCD Section 3C.7.3 is the PSHAKE method described in Section 2.5.2 (Reference 2.5-222) and Section 3.7.1. P-SHAKE is a Bechtel proprietary modified version of SHAKE.

3C.7.5.1 Description

P-SHAKE is a Bechtel proprietary modified version of SHAKE. P-SHAKE generates the same design earthquake-induced strain-compatible soil properties and site response motions as generated by SHAKE and the input files of the two programs for the most part are compatible. P-SHAKE is, however, built on a different program logic that allows the site response analysis to be performed with acceleration response spectrum as input instead of acceleration time histories used by SHAKE.

3C.7.5.2 Validation

The P-SHAKE program validation documents are located in Bechtel's Computation Service Library.

3C.7.5.3 Extent of Application

P-SHAKE is used to provide the site-specific earthquake-induced design ground motions and the associated strain-compatible soil properties for the Seismic Category I structures (RB/FB, CB, and FWSC).

3C.7.6 Site-Specific Dynamic Soil-Structure Interaction Analysis Program – ACS SASSI

3C.7.6.1 Description

ACS SASSI is a finite element computer code for performing 3D dynamic SSI analyses for shallow, embedded, deeply embedded, and buried structures under external vibratory or impulsive forces or earthquake ground motions. ACS SASSI is based on the SASSI code developed by the University of California at Berkeley.

3C.7.6.2 Validation

ACS SASSI was obtained from Ghiocel Predictive (GP) Technologies, Inc. under a 10 CFR 50, Appendix B, quality purchase order. Program validation documentation is available at GP Technologies, Inc., 6 South Main Street, 2nd Floor, Pittsford, NY 14534. Specific to the Unit 3 site conditions, GEH and Shimizu performed analyses to validate the results for ACS SASSI for use in performing sensitivity analyses for Unit 3.

3C.7.6.3 Extent of Application

This program is used to perform sensitivity analyses for Unit 3 site-specific SSI analysis for Seismic Category I structures.

Appendix 3D Computer Programs Used in the Design of Components, Equipment, and Structures

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3E [Deleted]

Appendix 3F Response of Structures to Containment Loads

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3G Design Details and Evaluation Results of Seismic Category I Structures

~~This section of the referenced DCD is incorporated by reference with no departures or supplements.~~

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add the following at the beginning of this section, just after the section title.

NAPS DEP 3.7-1

Sections 3G.1 through 3G.6 provide the standard design details and evaluation results for the Seismic Category I structures based on the CSDRS and the standard design analyses. Sections 3G.7 through 3G.10 provide the Unit 3 site-specific design details and evaluation results for the Seismic Category I structures based on the Unit 3 ground motion response spectra.

Add the following at the end of this section.

NAPS DEP 3.7-1

3G.7 Site-Specific Structural Evaluation of Reactor Building/Fuel Building Complex

The site-specific structural evaluations of the RB and FB, which are founded on a common basemat, are performed as the RB/FB complex and are described in this section.

3G.7.1 Objective and Scope

The objective of this section is to document the site-specific structural evaluations based on the Unit 3 analyses performed to address exceedances of the CSDRS. Results of SSI analyses indicate that seismic load demands, in some cases, exceed the seismic load demands used in the standard design. This section describes the site-specific design and analysis of the combined RB/FB structure.

3G.7.2 Conclusions

The major conclusions are summarized.

3G.7.3 Structural Description

The RB/FB structures are described in DCD Sections 3G.1.3 and 3G.3.3.

3G.7.4 Analytical Models

3G.7.4.1 Structural Models

Unit 3 site-specific structural models are based on the standard design structural models described in DCD Sections 3G.1.4 and 3G.3.4. The RB/FB is analyzed as an integrated structure.

3G.7.4.2 Foundation Models

Unit 3 site-specific foundation models are based on the standard design foundation models described in DCD Section 3G.1.4.2.

3G.7.5 Structural Analysis and Design

3G.7.5.1 Site Design Parameters

Key site-specific parameters used as inputs for the Unit 3 stability and bearing pressure calculations are identified in Table 3G.7-201.

Stability and bearing pressure calculations are performed using the results obtained from the site-specific SSI analyses of the RB/FB model for BE, LB, and UB partial and full column profiles

3G.7.5.2 Site Design Loads

The Unit 3 site-specific seismic loads for the RB/FB are obtained by site-specific SSI analyses, with results described in Section 3A.18.1.1.

3G.7.5.3 Stability Requirements

The stability requirements for the RB/FB are given in DCD Section 3G.1.5.3. A factor of safety of 1.1 is required for both overturning

and sliding. Analyses demonstrate that the RB/FB meets the required factors of safety for overturning stability and for stability against sliding.

3G.7.5.4 Structural Design Evaluation

[Information for this section will be provided later]

3G.7.5.5 Foundation Stability

The methodology used for the site-specific stability calculations is consistent with the methodology used for the standard design stability evaluations presented in the DCD Section 3.8.5.5. The seismic stability of the RB/FB is evaluated against overturning and sliding.

The factors of safety against overturning due to earthquake loading for the RB/FB, using the SSI seismic analysis results presented in Appendix 3A, are determined by using the energy approach described in DCD Section 3.7.2.14. The sliding evaluation is performed for two orthogonal horizontal directions separately using a linear time history analysis approach. In each direction, the phasing between the horizontal and vertical seismic forces is considered at each time step to compute the sliding factor of safety in the time domain.

Table 3G.7-225 presents the calculated overturning and sliding stability factors of safety for the SSI analyses Cases 1 through 6, which are described in Table 3A.15-201. The calculated overturning and sliding stability factors of safety are equal to or greater than the acceptance criteria in DCD Section 3G.1.5.3 and DCD Table 3.8-14 for a factor of safety acceptance criteria of 1.1. Therefore, the calculations demonstrate the adequate stability of the RB/FB.

The results of the sliding stability evaluations in Table 3G.7-225(b) show that the base friction and the lateral resistance provided by the concrete fill and Zone III rock embedment alone are sufficient to ensure that the RB/FB foundation maintains a factor of safety against sliding that is greater than the required acceptance criteria of 1.1. The maximum lateral pressures required to maintain a sliding factor of safety greater than 1.1 are less than the allowable lateral bearing capacity of the subgrade surrounding the RB/FB in Table 3G.7-201. Lateral passive pressures required to ensure the sliding stability of the RB/FB at the Unit 3 site are calculated assuming triangular distribution. In Section 3G.7.5.6, these pressures are compared with the corresponding lateral pressures used for the standard design of the RB/FB structures.

The calculations of the RB/FB foundation maximum soil dynamic bearing pressure demands follow the same Energy Balance (EB)/Modified Energy Balance (MEB) method that was used for the standard design calculations, as described in DCD Section 3G.1.5.5 (DCD Reference 3G.1-2). These dynamic bearing pressure demands are compared with the allowable dynamic bearing pressure of the Zone III-IV rock to ensure the capacity of the subgrade is sufficient to resist the dynamic bearing pressures from the RB/FB foundation.

The calculated maximum dynamic soil bearing pressures for the RB/FB foundation for the SSI analyses Cases 1 through 6, which are listed in Table 3A.15-201, are shown in Table 3G.7-231. The site-specific dynamic foundation bearing pressure calculations show that the Unit 3 site-specific demands are lower than the dynamic bearing pressures considered in the standard design of the RB/FB foundation (DCD Table 3G.1-58). The calculated maximum dynamic bearing pressure is also lower than the allowable dynamic bearing pressure of the Zone III-IV rock underlying the RB/FB foundation provided in Table 3G.7-201.

3G.7.5.6 Lateral Pressures on Exterior Embedded Walls

Figures 3G.7-205 through 3G.7-212 provide plots of the Unit 3 site-specific total lateral soil pressure acting on the below grade exterior walls of the RB/FB. Two plots are presented in each figure comparing the site-specific lateral pressure demands on each wall segment with the corresponding lateral pressure loads used for the standard design of RB/FB structures in DCD Section 3G.1. The first plot presents the comparison of the at-rest static and the dynamic components of the lateral pressures acting on the walls. The seismic lateral pressure demands represent the envelope of the results obtained from the site-specific SSI analyses described in Section 3A.17.12.4. The second plot compares the site-specific total pressures (static plus dynamic) and the maximum passive pressures required for sliding stability of the RB/FB at the Unit 3 site with the corresponding standard design loads. Figures 3G.7-205 through 3G.7-212 show that, near the slab at Elevation 270.3 ft NAVD88 (standard design Elevation -1.50 m), the site-specific total lateral pressures exceed the lateral pressures used for the standard design of RB/FB structures. The maximum total site-specific lateral pressures presented in this section are used as input for the site-specific evaluations of the RB/FB structures to demonstrate that the

capacities of the RB/FB below-grade exterior walls are sufficient to resist the Unit 3 site-specific lateral pressure demands.

NAPS DEP 3.7-1

Table 3G.7-201 **RB/FB Site-Specific Parameters**

Parameters		**Values**
Building Width:		
	X-direction (NS-direction)	229.7 ft (70 m)
	Y-direction (EW-direction)	160.8 ft (49 m)
Zone III Rock Embedment Depth:		
	Nominal Zone III Rock Elevation	El. 273 ft, NAVD88 (standard design El. -0.68 m)
	Depth to bottom of RB/FB basemat	48.6 ft (14.82 m)
At-Rest Lateral Pressure Coefficient K_0		
	Structural fill	0.36
	Concrete fill (calculated per Eq. 7-2)	0.176 (Poisson ratio ν_c = 0.15)
Unit weight (dry and submerged)		
	Structural fill	2.08 t/m^3 (dry) 1.295 t/m^3 (submerged)
	Concrete fill	2.32 t/m^3(dry) 1.445 t/m^3 (submerged)
Water Level:		
	Nominal Groundwater Elevation	El. 282.6 ft, NAVD88 (standard design El. 2.24 m)
	Depth below finished ground level grade	7.4 ft (2.26 m)
Friction Coefficient, μ:		
	Foundation/Zone III/IV Rock Interface	0.6 *)
Allowable Lateral Dynamic Bearing Pressure:		
	Zone III rock at El. 224.4 ft., NAVD88 (standard design El. -15.5 m)	32.8 ksf (1.57 MPa)
Allowable Dynamic Bearing Pressure:		
	Zone III-IV rock	259 ksf (12.4 MPa)

*) A value of 0.6 is used for the friction coefficient, which is the lowest value for concrete fill and Zone III-IV rock and the foundation structural concrete.

NAPS DEP 3.7-1

Table 3G.7-225 Factors of Safety for RB/FB Foundation Stability

(a) RB/FB Overturning Stability Evaluation

Subgrade Condition	Partial Column Profiles						Full Column Profiles					
	LB (Case 1)		BE (Case 2)		UB (Case 3)		LB (Case 4)		BE (Case 5)		UB (Case 6)	
Direction	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW
m_0gh (MN·m)	43,403	24,215	43,403	24,215	43,403	24,215	43,403	24,215	43,403	24,215	43,403	24,215
W_b (MN·m)	1,122	1,202	1,122	1,202	1,122	1,202	1,122	1,202	1,122	1,202	1,122	1,202
E_s (MN·m)	26.86	18.28	33.21	21.33	39.58	24.90	30.09	21.01	37.93	23.90	45.78	29.16
$FS=(m_0gh-W_b)/E_s$	1,574	1,259	1,273	1,079	1,068	**924**	1,405	1,095	1,115	963	**924**	789

Notes:

m_0 = total mass of structure and basemat
g = acceleration due to gravity
h = height of the center of structure mass at the overturning position

W_b = potential energy caused by the effect of buoyancy
E_s = maximum kinetic energy
FS = Factor of Safety

The effect of shear key is neglected conservatively.
The bold red number is the minimum Factor of Safety against overturning.

NAPS DEP 3.7-1

Table 3G.7-225 **Factors of Safety for RB/FB Foundation Stability** *(continued)*

(b) Evaluation of RB/FB Stability for Sliding at Bottom of Basemat (El. 224.4 ft NAVD88)

Basemat width in NS Dir.	70.0	m
Basemat width in EW Dir.	49.0	m
Depth of Zone III rock embedment	14.81	m
Total Weight	2360	MN
Buoyancy	597	MN

Subgrade Condition	Partial Column Profiles						Full Column Profiles					
	LB (Case 1)		BE (Case 2)		UB (Case 3)		LB (Case 4)		BE (Case 5)		UB (Case 6)	
Sliding Direction	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW
Time (sec)	6.330	3.695	4.410	3.200	4.410	3.195	3.150	3.695	1.820	3.200	3.320	3.195
Vertical Seismic Load (MN)	738	489	841	609	977	648	888	604	1125	830	371	861
Minimum Vertical Load (MN)	1025	1274	922	1154	786	1115	875	1159	638	933	1392	902
F_v: Horizontal Seismic Force (MN)	477	651	535	687	443	773	494	631	443	634	1009	682
F_o: Soil Force due to Turbine Building (MN)	127	0	127	0	127	0	127	0	127	0	127	0
F_{ub}: Bottom Friction Force (MN)	615	765	553	693	472	669	525	695	383	560	835	541
F_r: Lateral Resistance Force (MN)	50	0	176	64	156	181	159	0	244	137	415	209
FS $(=(F_{ub}+F_r)/(F_v+F_o))$	1.10	1.17	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
σ_{max}: Maximum Stress (MPa) Associated with Lateral Resistance F_r	0.14	0.00	0.48	0.12	0.43	0.35	0.44	0.00	0.67	0.26	**1.14**	0.40

The bold red number (1.14) is the maximum lateral pressure demand on the Zone III rock.

NAPS DEP 3.7-1

Table 3G.7-231 Maximum Soil Dynamic Bearing Pressure Demand for RB/FB

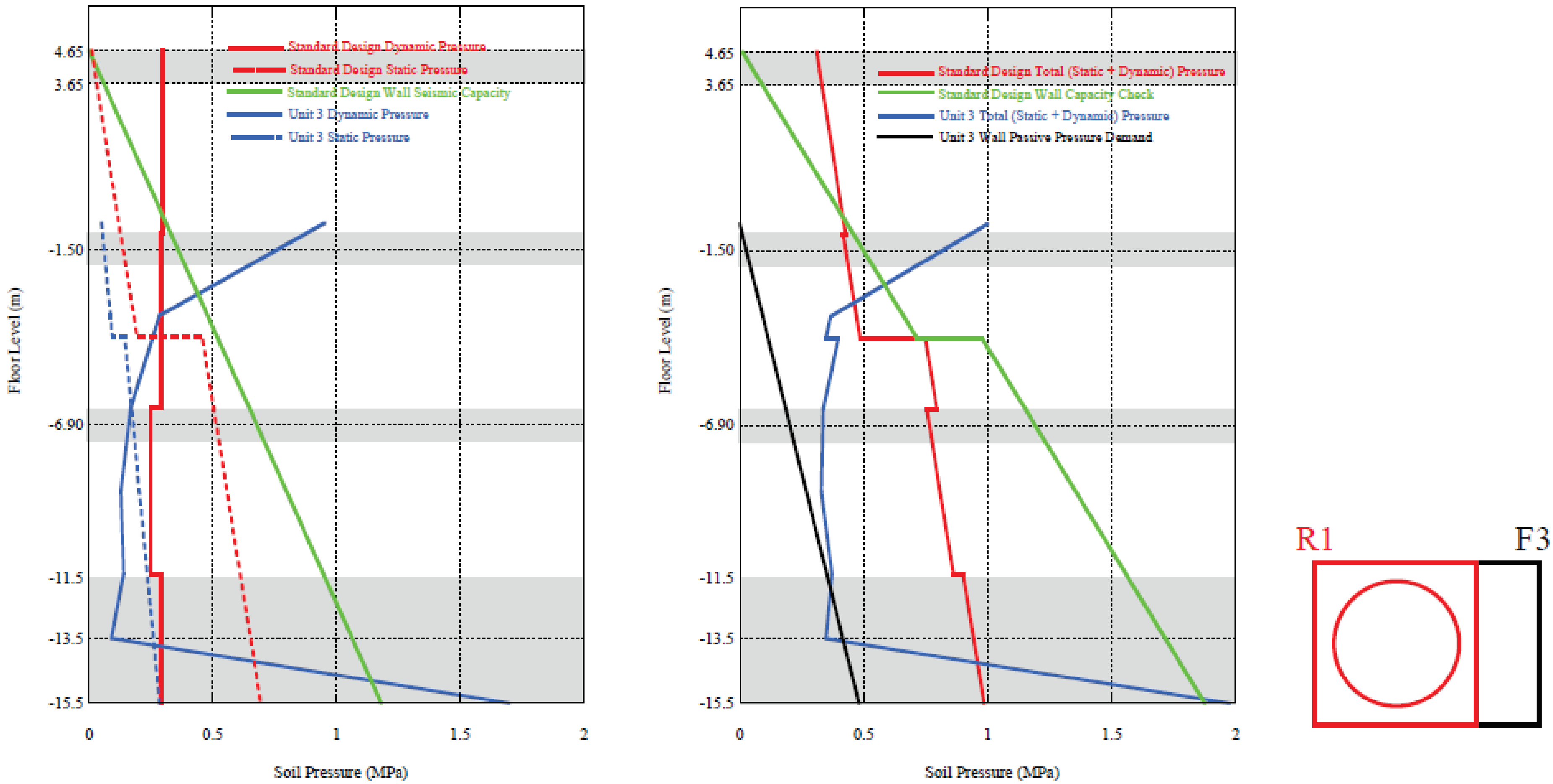
Basemat width in NS (X) Dir.	70.0	m
Basemat width in EW (Y) Dir.	49.0	m
Gravity Load (D)	2360	MN
Buoyancy (B)	597	MN

		Subgrade Condition	Partial Column Analyses						Full Column Analyses					
			LB (Case 1)		BE (Case 2)		UB (Case 3)		LB (Case 4)		BE (Case 5)		UB (Case 6)	
Direction Vertical Seismic Load			downward		downward		downward		downward		downward		downward	
SASSI	Time	(sec)	3.905		7.080		7.075		3.565		3.640		3.625	
	Vertical seismic load (V_z)	(MN)	244.3		510.0		611.7		440.7		447.5		414.7	
	Total vertical load	(MN)	2,604		2,870		2,972		2,801		2,808		2,775	
	Moment in NS-dir (Mx)	(MN-m)	10,855		8,433		9,948		10,422		9,381		8,299	
	Moment in EW-dir (My)	(MN-m)	4,046		7,704		7,178		4,574		7,618		8,058	
Simplified Method**			EB	MEB	EB	MEB	EB	MEB	EB	MEB	EB	MEB	EB	MEB
NS dir. ↓ EW dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.015	-0.015	-0.001	-0.001	-0.001	-0.001	0.004	0.004	0.010	0.010	0.002	0.002
	Max. basemat moment (Mx)	(MN-m)	10,855	10,855	8,433	8,433	9,948	9,948	10,422	10,422	9,381	9,381	8,299	8,299
	Max. bearing pressure 1 (Px)	(MPa)	1.03	1.03	1.05	1.05	1.12	1.12	1.08	1.08	1.05	1.05	1.02	1.02
	Max. bearing pressure 2 (Py)	(MPa)	---	0.14	---	0.28	---	0.26	---	0.16	---	0.27	---	0.29
	Max. Bearing pressure (Pxy=Px+Py)	(MPa)	---	1.18	---	1.32	---	1.37	---	1.24	---	1.32	---	1.30
EW dir. ↓ NS dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.020	-0.020	-0.002	-0.002	-0.002	-0.002	0.006	0.006	0.014	0.014	0.003	0.003
	Max. basemat moment (My)	(MN-m)	4,046	4,046	7,704	7,704	7,178	7,178	4,574	4,574	7,618	7,618	8,058	8,058
	Max. bearing pressure 1(Py)	(MPa)	0.90	0.90	1.11	1.11	1.12	1.12	0.98	0.98	1.09	1.09	1.10	1.10
	Max. bearing pressure 2 (Px)	(MPa)	---	0.27	---	0.21	---	0.25	---	0.26	---	0.23	---	0.21
	Max. Bearing Pressure (Pyx=Py+Px)	(MPa)	---	1.18	---	1.32	---	1.37	---	1.24	---	1.32	---	1.30
Envelope of Pxy and Pyx		(MPa)	---	1.18	---	1.32	---	**1.37**	---	1.24	---	1.32	---	1.30

Notes: *: SASSI2010 analysis is a linear time history analysis with the 3D excitation.
**: EB and MEB stand for energy balance (EB) and modified energy balance (MEB) methods.
The bold red number (1.37) is the maximum dynamic bearing pressure demand on the Zone III-IV rock.

NAPS DEP 3.7-1

Figure 3G.7-205 **Lateral Pressures on R1 Wall – Envelope of LB, BE and UB Partial Column Profiles Analyses**

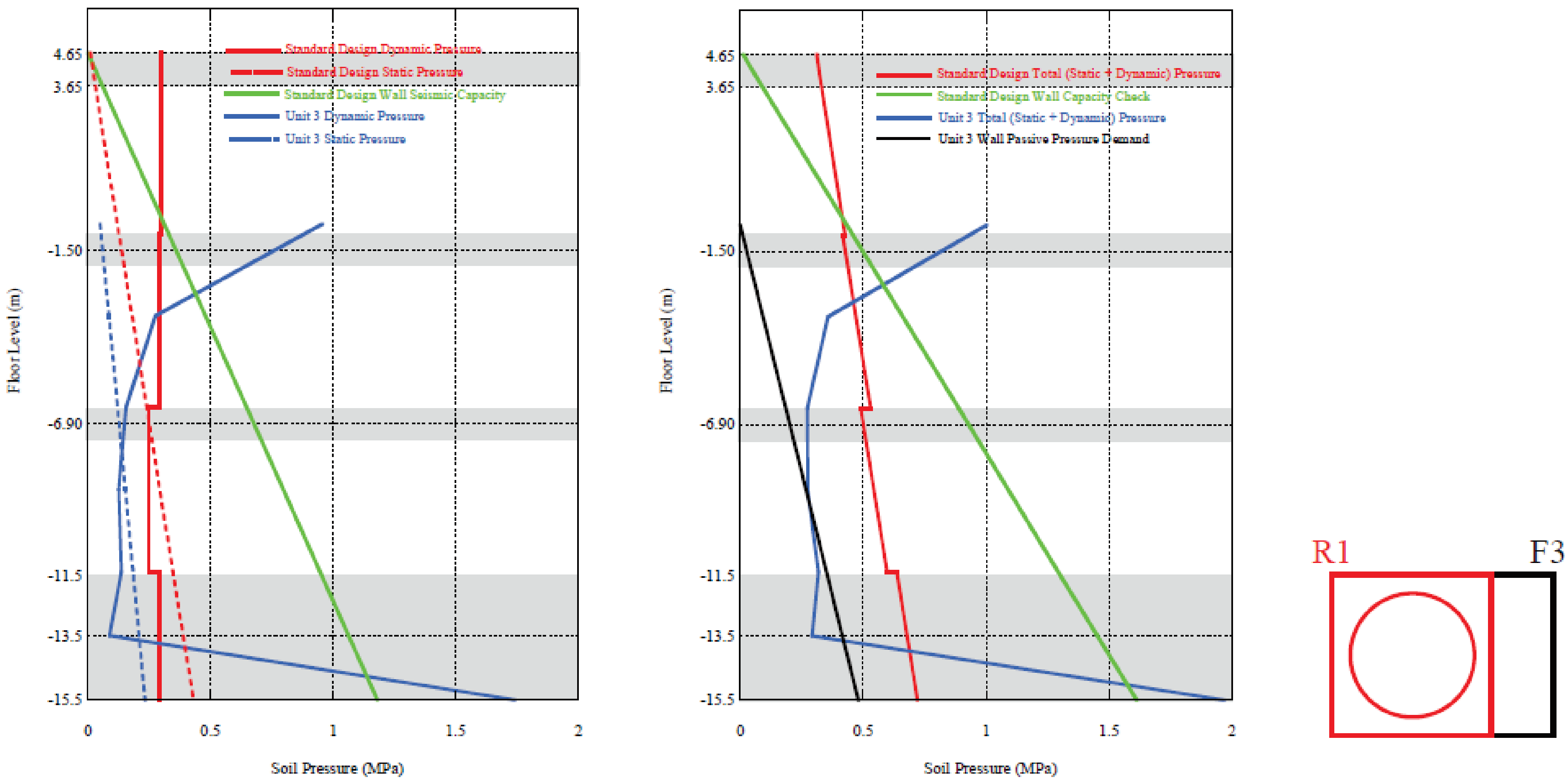


(a) Static and Dynamic Pressures **(b) Total Active and Required Passive Pressures**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.7-206 **Lateral Pressures on F3 Wall – Envelope of LB, BE and UB Partial Column Profiles Analyses**

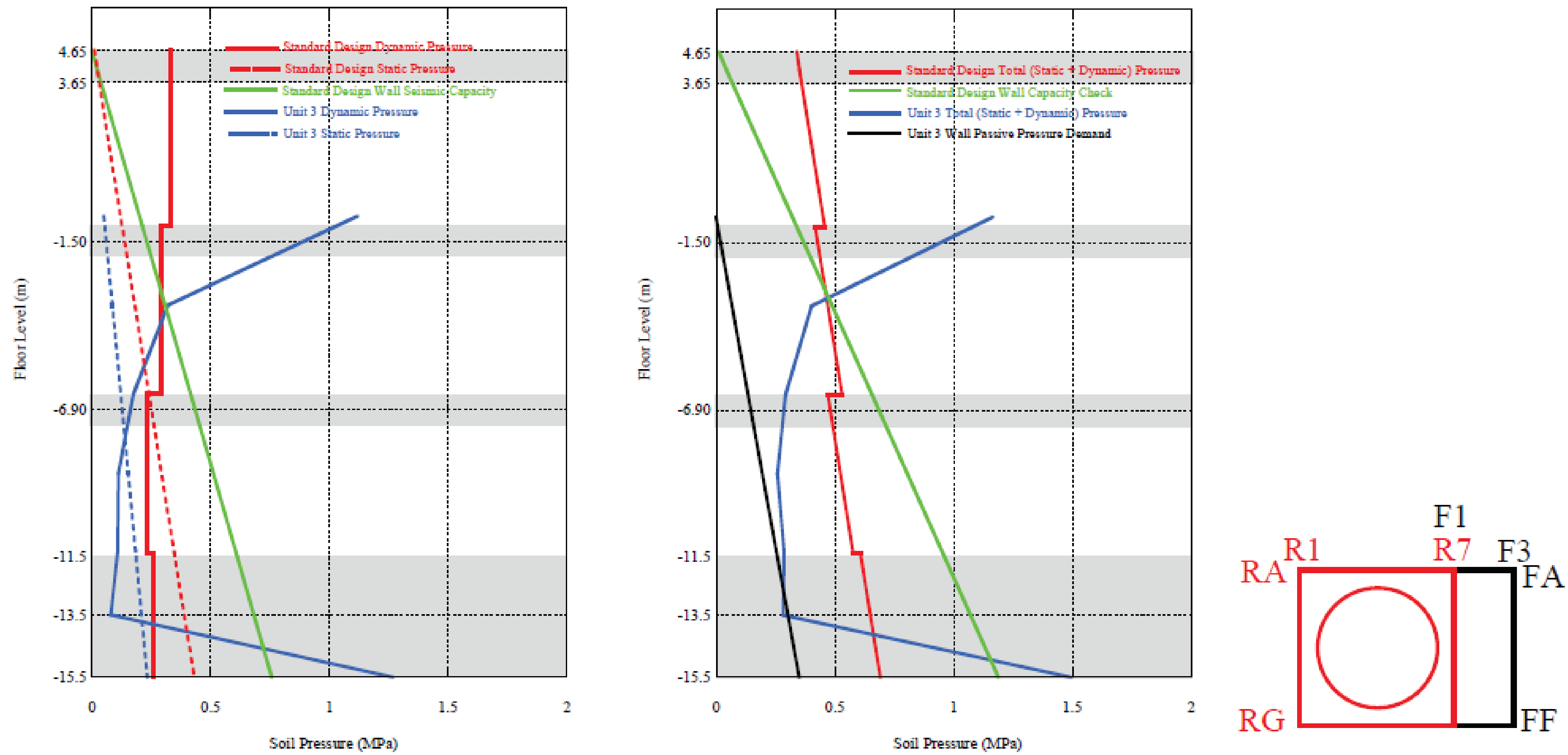


(a) Static and Dynamic Pressures **(b) Total Active and Required Passive Pressures**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.7-207 **Lateral Pressures on RA, RG and FF Walls – Envelope of LB, BE and UB Partial Column Profiles Analyses**

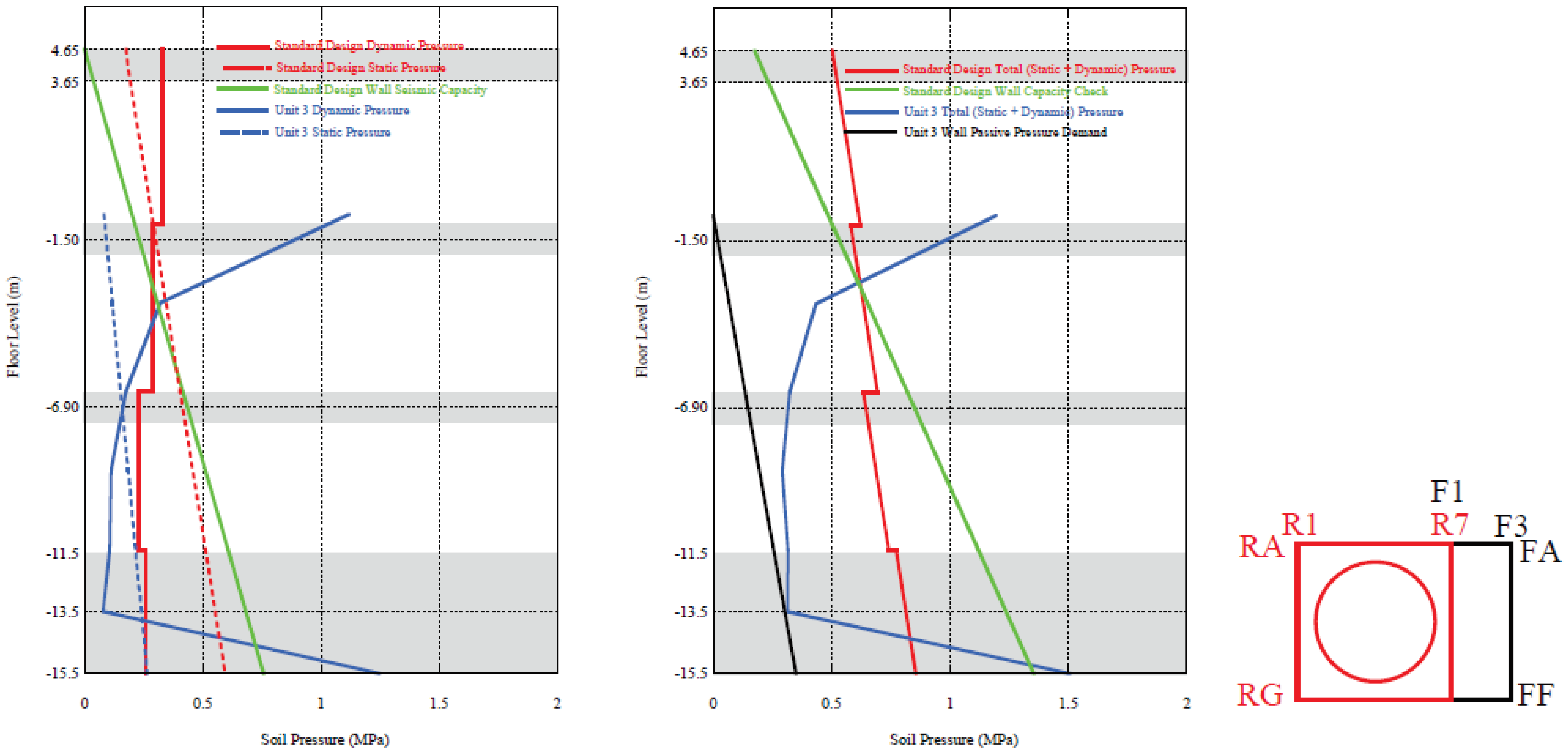


(a) Static and Dynamic Pressures **(b) Total Active and Required Passive Pressures**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.7-208 Lateral Pressures on FA Wall – Envelope of LB, BE and UB Partial Column Profiles Analyses



(a) Static and Dynamic Pressures

(b) Total Active and Required Passive Pressures

Note: The shaded area shows thickness of the floor slabs and basemat.

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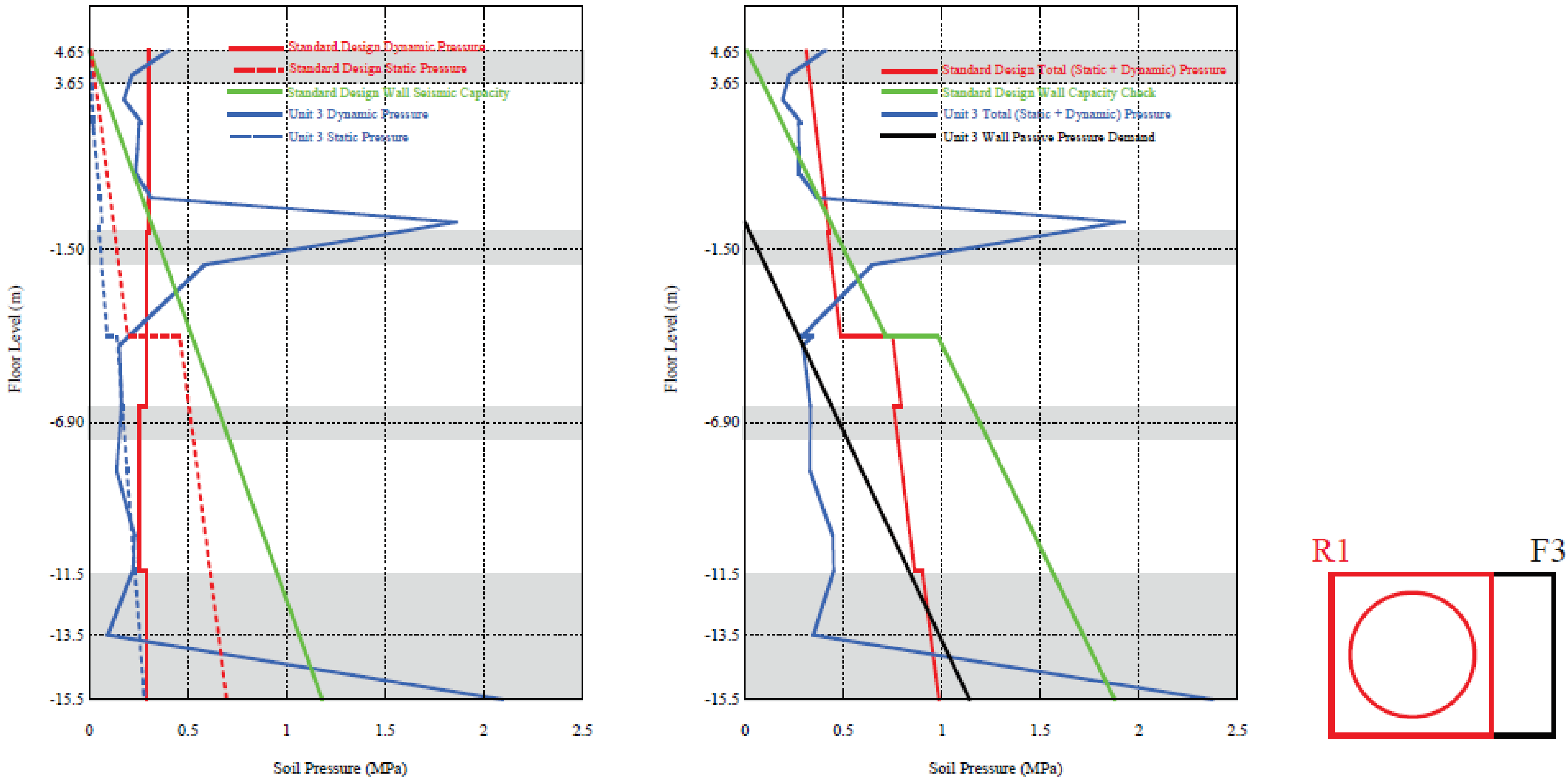
Figure 3G.7-209 **Lateral Pressures on R1 Wall – Envelope of LB, BE and UB Full Column Profiles Analyses**



(a) Static and Dynamic Pressures

(b) Total Active and Required Passive Pressures

Note: The shaded area shows thickness of the floor slabs and basemat.

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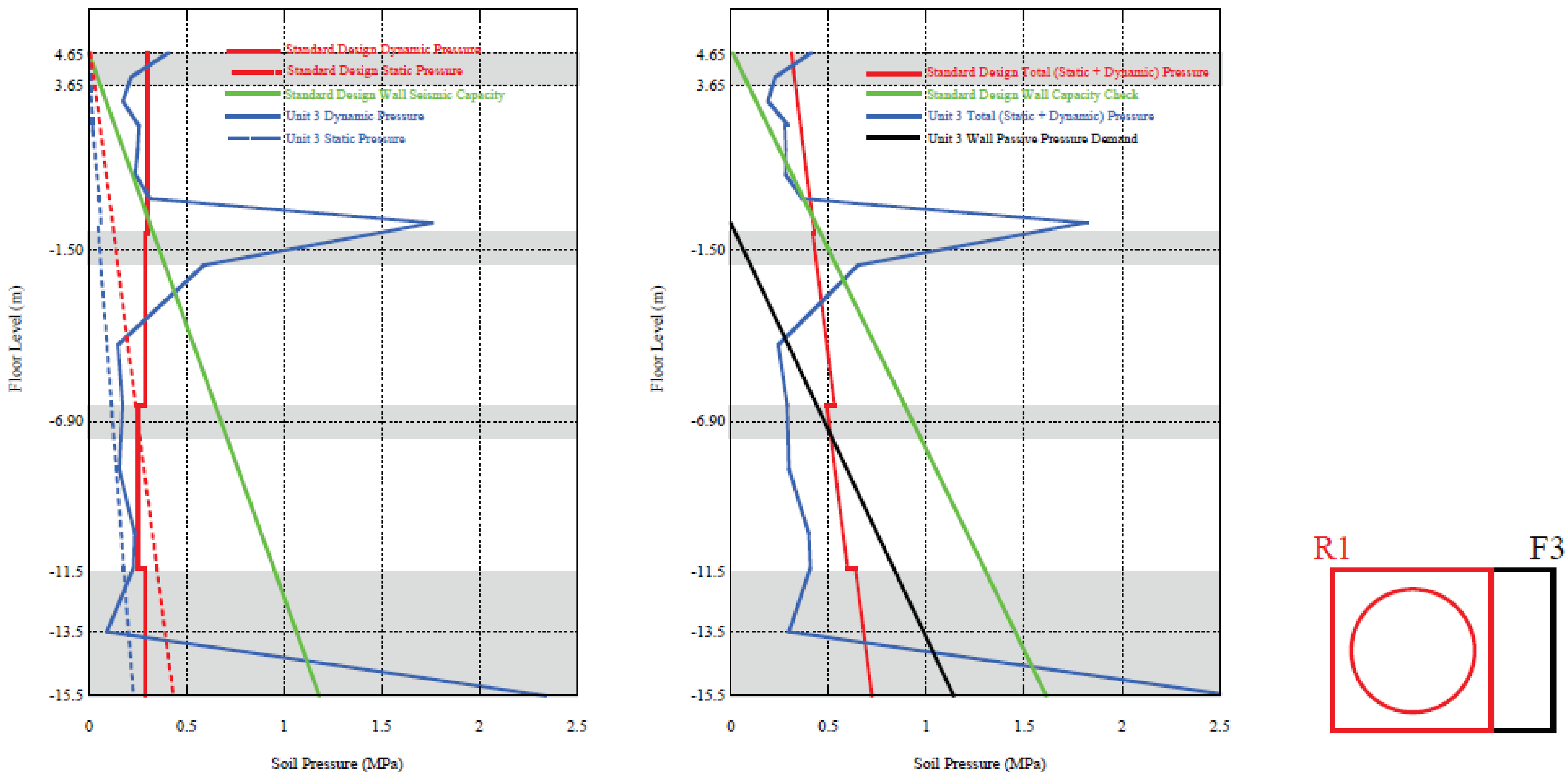
Figure 3G.7-210 **Lateral Pressures on F3 Wall - Envelope of LB, BE and UB Full Column Profiles Analyses**



(a) Static and Dynamic Pressures

(b) Total Active and Required Passive Pressures

Note: The shaded area shows thickness of the floor slabs and basemat.

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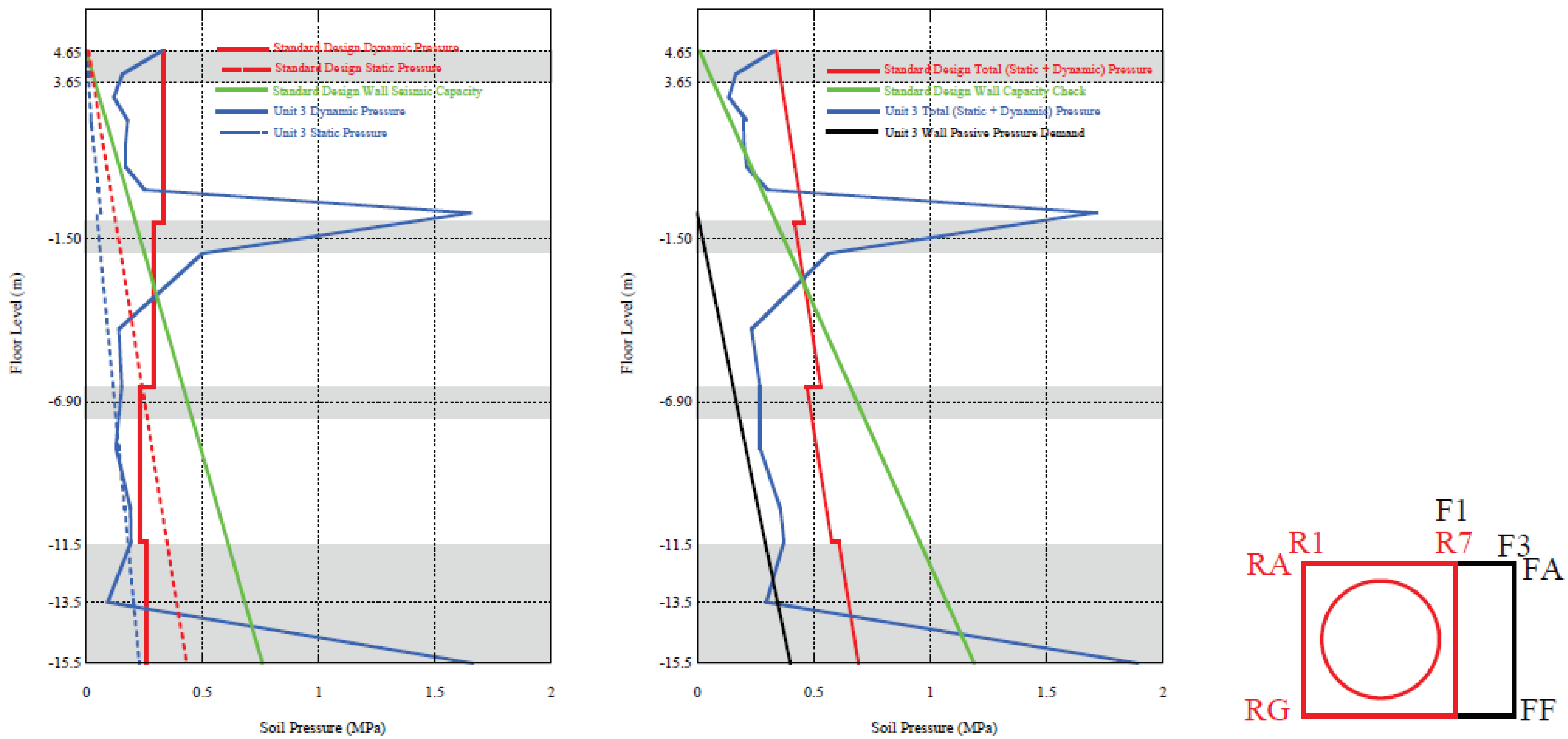
Figure 3G.7-211 **Lateral Pressures on RA, RG and FF Walls - Envelope of LB, BE and UB Full Column Profiles Analyses**



(a) Static and Dynamic Pressures **(b) Total Active and Required Passive Pressures**

Note: The shaded area shows thickness of the floor slabs and basemat.

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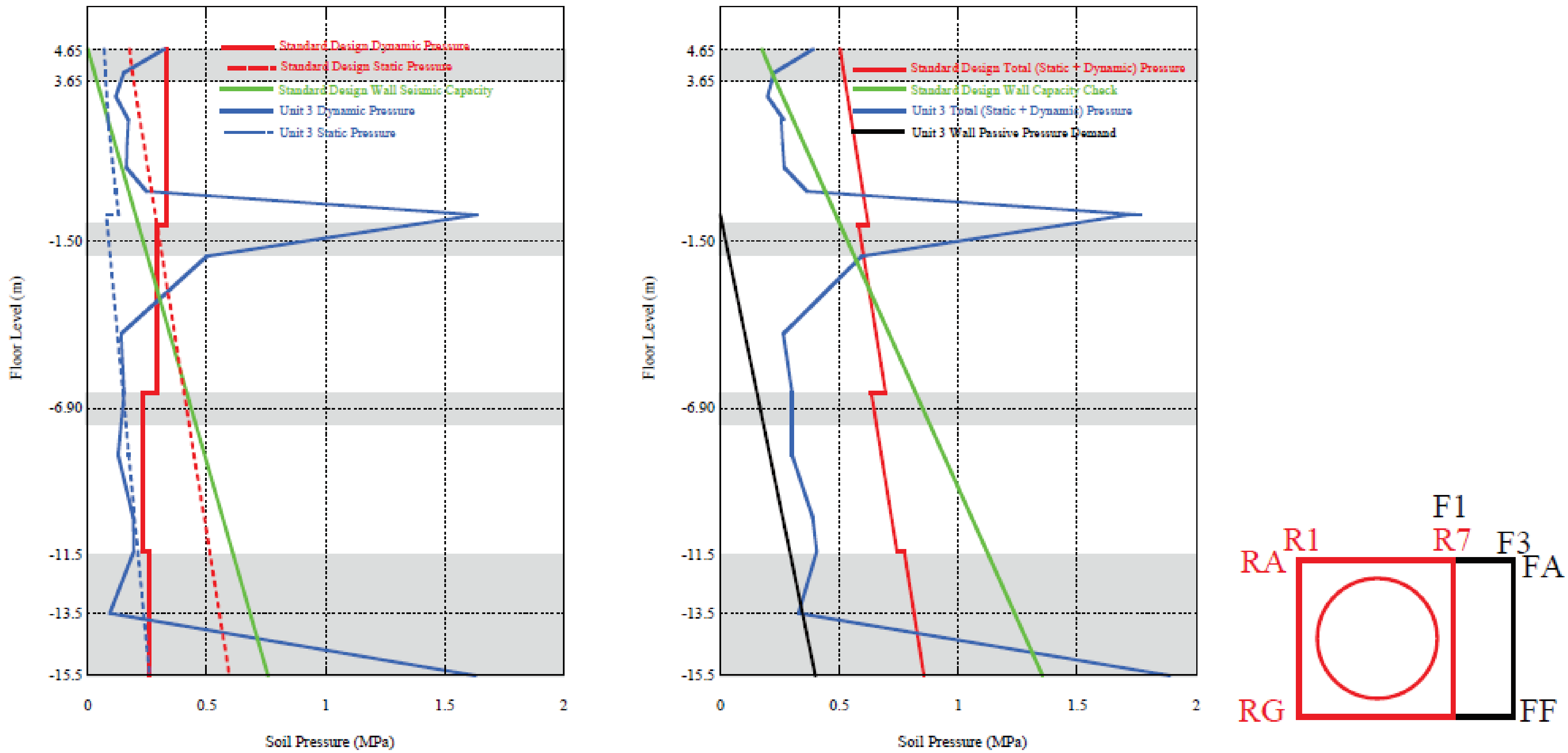
Figure 3G.7-212 **Lateral Pressures on FA Wall - Envelope of LB, BE and UB Full Column Profiles Analyses**



(a) Static and Dynamic Pressures

(b) Total Active and Required Passive Pressures

Note: The shaded area shows thickness of the floor slabs and basemat.

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3G.8 Site-Specific Structural Evaluation of Control Building

The site-specific structural evaluations of the CB are described in this section.

3G.8.1 Objective and Scope

The objective of this section is to document the site-specific structural evaluations based on the Unit 3 analyses performed to address exceedances of the CSDRS. Results of SSI and SSSI analyses indicate that seismic load demands, in some cases, exceed the seismic load demands used in the standard design. This section describes the site-specific design and analysis of the CB structure.

3G.8.2 Conclusions

The major conclusions are summarized.

3G.8.3 Structural Description

The CB structure is described in DCD Section 3G.2.3.

3G.8.4 Analytical Models

3G.8.4.1 Structural Models

Unit 3 site-specific structural models are based on the standard design structural models described in DCD Section 3G.2.4.

3G.8.4.2 Foundation Models

Unit 3 site-specific foundation models are based on the standard design foundation models described in DCD Section 3G.2.4.2.

3G.8.5 Structural Analysis and Design

3G.8.5.1 Site Design Parameters

Key site-specific parameters used as inputs for the Unit 3 stability and bearing pressure calculations are identified in Table 3G.8-201.

Stability and bearing pressure calculations are performed using the results obtained from the site-specific SSI analyses of the CB model for BE, LB, and UB partial and full column profiles

3G.8.5.2 Site Design Loads

The Unit 3 site-specific seismic loads for the CB are obtained by site-specific SSI analyses, with results described in Section 3A.18.1.2.

3G.8.5.3 Stability Requirements

The stability requirements for the CB are given in DCD Section 3G.2.5.3.

A factor of safety of 1.1 is required for both overturning and sliding. Analyses demonstrate that the CB meets the required factors of safety for overturning stability and for stability against sliding.

3G.8.5.4 Structural Design Evaluation

[Information for this section will be provided later.]

3G.8.5.5 Foundation Stability

The methodology used for the calculations of overturning and sliding seismic stability of the CB at the Unit 3 site is consistent with the methodology used for the standard design stability evaluations presented in DCD Sections 3.7.2.14 and 3.8.5.5.

Table 3G.8-208 presents overturning stability calculations of the CB at the Unit 3 site based on the results of the SSI analyses of the CB stand-alone model with full stiffness and SSE damping (Cases 7 through 12 in Table 3A.15-202). The table shows that the calculated overturning stability factor of safety is greater than the required value of 1.1.

The CB sliding evaluation is performed for two orthogonal horizontal directions separately, using a linear time history analysis approach. In each direction, the phasing between the horizontal shear and vertical seismic forces is considered at each time step to compute sliding factors of safety for different instances of time as the ratio between the base friction resistance and the horizontal driving force. The minimum value obtained during the duration of the site-specific ground motion is adopted as the factor of safety for sliding stability of the CB.

The CB sliding stability evaluations are performed considering two critical sliding planes located as follows:

- Bottom of the CB basemat at Elevation 241 ft NAVD88 (Table 3G.8-209a)
- Bottom of the underlying concrete fill at nominal Elevation 225 ft NAVD88 (Table 3G.8-209b)

The results of the sliding stability evaluations in Tables 3G.8-209a and 3G.8-209b show that the base friction and the lateral resistance provided by the concrete fill and Zone III rock embedment alone are sufficient to resist the seismic driving forces and ensure that the CB

maintains a factor of safety against sliding that is greater than the required acceptance criteria of 1.1. The calculations demonstrate that the CB will maintain the stability against sliding at the two critical sliding failure planes located at the bottom of the CB basemat and the bottom of the concrete fill supporting the CB foundation. The maximum lateral pressures required to maintain a sliding factor of safety greater than 1.1 are less than the allowable lateral bearing capacity of the subgrade surrounding the CB presented in Table 3G.8-201. Lateral passive pressures required to ensure the sliding stability of the CB at the Unit 3 site are calculated assuming triangular distribution. In Section 3G.8.5.6, these pressures are compared with the corresponding lateral pressures used for the standard design of the CB structure.

Tables 3G.8-211a and 3G.8-211b present a summary of calculations of the site-specific dynamic bearing pressure demands on the concrete fill under the CB basemat and the underlying Zone III/IV rock, respectively. The calculations follow the same EB/MEB method that was used for the standard design calculations. The calculations presented in Tables 3G.8-211a and 3G.8-211b show that the Unit 3 site-specific foundation dynamic bearing demands are lower than the dynamic bearing pressures considered in the standard design of the CB foundation (DCD Table 3G.2-27). The calculated maximum dynamic pressures are also lower than the allowable bearing pressure of the concrete fill supporting the CB basemat and the allowable bearing pressure of the underlying Unit 3 Zone III-IV rock subgrade in Table 3G.8-201.

3G.8.5.6 Lateral Pressures on Exterior Embedded Walls

Figures 3G.8-203 through 3G.8-210 provide plots of the Unit 3 site-specific total lateral pressure demands on the below grade exterior walls of the CB. Two plots are presented in each figure comparing the site-specific lateral pressure demands with corresponding lateral pressure loads used for the standard design of the CB structure in DCD Section 3G.2. The first plot presents comparison of the at-rest static and the dynamic components of the lateral pressures acting on the walls. The seismic lateral pressure demands represent the envelope of the results obtained from the site-specific SSI analyses presented in Section 3A.17.13.4. The second plot compares the site-specific total pressures (static plus dynamic) and the maximum passive pressures

required for sliding stability of the CB at Unit 3 site with the corresponding standard design loads.

Figures 3G.8-203 through 3G.8-210 show that near the slab at Elevation 267.9 ft NAVD88 (standard design Elevation -2.25 m) and near the top of the CB basemat at Elevation 251.0 ft NAVD88 (standard design Elevation -7.40 m), the site-specific total lateral pressures can exceed the total lateral pressures used for the standard design of CB structures. Figures 3G.8-205 and 3G.8-206 also show that the lateral passive pressures required to ensure the stability of the CB against sliding in the EW direction can also exceed the pressures used for the standard design for the CB wall capacity check. The site-specific CB structural evaluation uses these site-specific lateral load demands as input to demonstrate that the standard design of the CB structure is adequate to withstand the Unit 3 site-specific lateral pressure demands on the CB below-grade exterior walls.

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Table 3G.8-201 **CB Site-Specific Parameters**

Parameters	**Values**
Building Width:	
X-direction (NS-direction)	30.3 m(99.4ft)
Y-direction (EW-direction)	23.8 m(78.1ft)
Zone III Rock Embedment Depth:	
Nominal Zone III Rock Elevation	EL 265 ft NAVD88
Depth to bottom of CB basemat	7.28 m (23.9 ft)
Depth to bottom of concrete fill	12.19 m (40 ft)
Water Level:	
Nominal Groundwater Elevation	EL 281.0 ft NAVD88 2.74 m (9 ft) below finished ground level grade
Friction Coefficient, μ	
Foundation/Concrete Fill/Rock Interfaces	0.6*)
Allowable Lateral Dynamic Bearing Pressure:	
Zone III rock at EL 241 ft NAVD88	1.39 MPa (29.1 ksf)
Zone III rock at EL 225 ft NAVD88	1.47 MPa (30.6 ksf)
Allowable Dynamic Bearing Pressure:	
Zone III-IV rock	12.4 MPa (259 ksf)

*) A value of 0.6 is used for the friction coefficient, which is the lowest value specified for site-specific concrete fill and Zone III-IV rock interfaces with the foundation structural concrete for the CB.

NAPS DEP 3.7-1

Table 3G.8-208 **Factors of Safety for CB Foundation Overturning Stability**

Subgrade Condition	Partial Column Profiles						Full Column Profiles					
	LB (Case 7)		BE (Case 8)		UB (Case 9)		LB (Case 10)		BE (Case 11)		UB (Case 12)	
Direction	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW
m_0gh (MN·m)	1,525	1,028	1,525	1,028	1,525	1,028	1,525	1,028	1,525	1,028	1,525	1,028
W_b (MN·m)	123.9	131.7	123.9	131.7	123.9	131.7	123.9	131.7	123.9	131.7	123.9	131.7
E_s (MN·m)	1.3	1.2	1.5	1.5	1.9	1.7	1.2	1.1	1.3	1.2	1.5	1.4
$FS=(m_0gh-W_b)/E_s$	1,065	763	910	605	750	**519**	1,159	783	1,065	760	909	662

Notes:

m_0 = total mass of structure and basemat
g = acceleration due to gravity
h = height of the center of structure mass at the overturning position
W_b = potential energy caused by the effect of buoyancy
E_s = maximum kinetic energy
FS = Factor of Safety
The bold red number (519) is the minimum Factor of Safety against overturning.

NAPS DEP 3.7-1

Table 3G.8-209a **Factors of Safety for CB Foundation Sliding Stability, Evaluation of CB Stability for Sliding at Bottom of Basemat (Elevation 241 ft NAVD88)**

Basemat width in NS Dir.	30.3	m
Basemat width in EW Dir.	23.8	m
Depth of Zone III rock embedment	7.28	m
Total Weight	197	MN
Buoyancy (B)	86	MN

Subgrade Condition		Partial Column Profiles						Full Column Profiles					
		LB (Case 7)		BE (Case 8)		UB (Case 9)		LB (Case 10)		BE (Case 11)		UB (Case 12)	
Sliding Direction		NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW
Time	(sec)	3.090	1.810	4.630	1.810	1.070	1.805	1.065	1.810	3.085	1.805	3.085	1.805
Vertical seismic load (V_z)	(MN)	49	103	77	114	70	105	55	85	42	75	49	77
Minimum vertical load	(MN)	62	8	34	0	41	6	56	26	69	36	62	34
F_v: Horizontal Seismic Force (MN)	(MN)	61	44	71	84	104	120	50	43	60	52	93	67
F_{ub}: Bottom Friction Force (MN)	(MN)	37	5	20	0	25	3	33	16	41	21	37	20
F_r: Lateral Resistance Force (MN)	(MN)	30	44	58	92	90	129	22	31	24	36	65	53
$FS = ((F_{ub}+F_r)/F_v)$		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
σ_{MAX}: Maximum Stress (MPa) Associated with Lateral Resistance F_r		0.35	0.40	0.67	0.84	1.04	**1.17**	0.25	0.28	0.28	0.33	0.75	0.48

Note:The number in bold red (1.17) is the maximum lateral pressure demand on Zone III rock.

NAPS DEP 3.7-1

Table 3G.8-209b **Factors of Safety for CB Foundation Sliding Stability, Evaluation of CB Stability for Sliding at Bottom of Concrete Fill (Elevation 225 ft NAVD88)**

Basemat width in NS Dir.	30.3	m
Basemat width in EW Dir.	23.8	m
Depth of Zone III rock embedment	12.38	m
Total Weight	278	MN
Buoyancy (B)	121	MN

Subgrade Condition		**Partial Column Profiles**						**Full Column Profiles**					
		LB (Case 7)		**BE (Case 8)**		**UB (Case 9)**		**LB (Case 10)**		**BE (Case 11)**		**UB (Case 12)**	
Sliding Direction		**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**
Time	(sec)	1.135	1.805	3.085	1.805	3.085	1.805	1.065	1.810	3.085	1.805	3.085	1.805
Vertical seismic load (V_z)	(MN)	103	125	62	133	64	145	55	85	42	75	49	77
Minimum vertical load	(MN)	54	32	95	24	93	12	56	26	69	36	62	34
F_v: Horizontal Seismic Force	(MN)	65	42	113	89	146	130	50	43	60	52	93	67
F_{ub}: Bottom Friction Force	(MN)	32	19	57	14	56	7	33	16	41	21	37	20
F_r: Lateral Resistance Force	(MN)	39	27	68	83	105	136	22	31	24	36	65	53
$FS = ((F_{ub}+F_r)/F_v)$		1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
σ_{MAX}: Maximum Stress (MPa) Associated with Lateral Resistance F_r		0.27	0.15	0.47	0.45	0.72	**0.74**	0.15	0.17	0.16	0.19	0.44	0.28

Note: The number in bold red (0.74) is the maximum lateral pressure demand on Zone III rock.

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Table 3G.8-211a **Maximum Soil Dynamic Bearing Pressure Demand for CB, Calculations of Dynamic Bearing Pressure Demands on Concrete Fill under CB Basemat**

Basemat width in NS (X) Dir.	30.3	m	
Basemat width in EW (Y) Dir.	23.8	m	
Gravity Load (D)	197	MN	
Buoyancy (B)	86	MN	(considered only in combination with upward vertical seismic load (V_z))

			Partial Column Analyses						Full Column Analyses					
	Subgrade Condition		**LB (Case 7)**		**BE (Case 8)**		**UB (Case 9)**		**LB (Case 10)**		**BE (Case 11)**		**UB (Case 12)**	
	Direction Vertical Seismic Load		**downward**		**upward**		**upward**		**downward**		**upward**		**upward**	
SASSI*	Time	(sec)	3.085		1.810		1.810		1.810		1.810		1.815	
	Vertical seismic load (V_z)	(MN)	35.7		82.5		86.8		79.8		83.2		85.1	
	Total vertical load	(MN)	233		28		24		277		280		282	
	Moment in NS-dir (Mx)	(MN-m)	590		194		172		76		152		105	
	Moment in EW-dir (My)	(MN-m)	302		528		692		218		140		69	
Simplified Method**			**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**
NS dir. ↓ EW dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	15.5	18.2	17.6	21.0	0.0	0.0	0.0	0.0	0.0	0.0
	Max. basemat rotation (ϕ)	(10^{-4} rad)	0.004	0.004	0.002	0.002	0.001	0.001	0.010	0.010	0.009	0.009	0.005	0.005
	Max. basemat moment (Mx)	(MN-m)	590	590	194	196	172	173	76	76	152	152	105	105
	Max. bearing pressure 1 (Px)	(MPa)	0.48	0.48	0.09	0.10	0.08	0.08	0.40	0.40	0.43	0.43	0.42	0.42
	Max. bearing pressure 2 (Py)	(MPa)	---	0.11	---	0.23	---	0.31	---	0.08	---	0.05	---	0.02
	Max. Bearing pressure (Pxy=Px+Py)	(MPa)	---	0.59	---	0.32	---	0.39	---	0.48	---	0.48	---	0.44
EW dir. ↓ NS dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	67.3	83.6	77.8	92.2	0.0	0.0	0.0	0.0	0.0	0.0
	Max. basemat rotation (ϕ)	(10^{-4} rad)	0.008	0.008	0.014	0.057	0.020	0.156	0.011	0.011	0.015	0.015	0.009	0.009
	Max. basemat moment (My)	(MN-m)	302	302	528	301	692	273	218	218	140	140	69	69
	Max. bearing pressure 1(Py)	(MPa)	0.43	0.43	0.22	0.48	0.28	0.85	0.46	0.46	0.44	0.44	0.42	0.42
	Max. bearing pressure 2 (Px)	(MPa)	---	0.16	---	0.33	---	0.60	---	0.02	---	0.04	---	0.03
	Max. Bearing Pressure (Pyx=Py+Px)	(MPa)	---	0.59	---	0.81	---	1.46	---	0.48	---	0.48	---	0.44
	Envelope of Pxy and Pyx	(MPa)	---	0.59	---	0.81	---	**1.46**	---	0.48	---	0.48	---	0.44

Notes: *: SASSI2010 analysis is a linear time history analysis with the 3D excitation.
**: EB and MEB stand for energy balance (EB) and modified energy balance (MEB) methods.
The number in bold red (1.46) is the maximum dynamic bearing pressure demand on the Zone III-IV rock.

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Table 3G.8-211b **Maximum Soil Dynamic Bearing Pressure Demand for CB, Calculations of Dynamic Bearing Pressure Demands on Zone III-IV Rock**

Basemat width in NS (X) Dir.	30.3	m
Basemat width in EW (Y) Dir.	23.8	m
Gravity Load (D)	197	MN
Buoyancy (B)	86	MN (considered only in combination with upward vertical seismic load (V_z))

		Subgrade Condition	Partial Column Analyses						Full Column Analyses					
			LB (Case 7)		BE (Case 8)		UB (Case 9)		LB (Case 10)		BE (Case 11)		UB (Case 12)	
		Direction Vertical Seismic Load	downward		downward		downward		downward		downward		downward	
SASSI*	Time	(sec)	1.810		1.815		1.810		3.020		1.805		1.810	
SASSI*	Vertical seismic load (V_z)	(MN)	114.1		112.3		122.3		16.9		106.1		121.0	
SASSI*	Total vertical load	(MN)	392		390		400		294		384		399	
SASSI*	Moment in NS-dir (Mx)	(MN-m)	95		144		154		314		83		13	
SASSI*	Moment in EW-dir (My)	(MN-m)	177		353		380		378		251		169	
Simplified Method**			EB	MEB	EB	MEB	EB	MEB	EB	MEB	EB	MEB	EB	MEB
NS dir. ↓ EW dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
NS dir. ↓ EW dir.	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.004	-0.004	-0.010	-0.010	-0.006	-0.006	0.006	0.006	0.005	0.005	0.002	0.002
NS dir. ↓ EW dir.	Max. basemat moment (Mx)	(MN-m)	95	95	144	144	154	154	314	314	83	83	13	13
NS dir. ↓ EW dir.	Max. bearing pressure 1 (Px)	(MPa)	0.57	0.57	0.58	0.58	0.60	0.60	0.49	0.49	0.55	0.55	0.56	0.56
NS dir. ↓ EW dir.	Max. bearing pressure 2 (Py)	(MPa)	---	0.06	---	0.12	---	0.13	---	0.13	---	0.09	---	0.06
NS dir. ↓ EW dir.	Max. Bearing pressure (Pxy=Px+Py)	(MPa)	---	0.63	---	0.70	---	0.73	---	0.63	---	0.64	---	0.62
EW dir. ↓ NS dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
EW dir. ↓ NS dir.	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.016	-0.016	-0.008	-0.008	-0.008	-0.008	0.008	0.008	0.003	0.003	0.002	0.002
EW dir. ↓ NS dir.	Max. basemat moment (My)	(MN-m)	177	177	353	353	380	380	378	378	251	251	169	169
EW dir. ↓ NS dir.	Max. bearing pressure 1(Py)	(MPa)	0.61	0.61	0.66	0.66	0.69	0.69	0.54	0.54	0.62	0.62	0.61	0.61
EW dir. ↓ NS dir.	Max. bearing pressure 2 (Px)	(MPa)	---	0.03	---	0.04	---	0.04	---	0.09	---	0.02	---	0.00
EW dir. ↓ NS dir.	Max. Bearing Pressure (Pyx=Py+Px)	(MPa)	---	0.63	---	0.70	---	0.73	---	0.63	---	0.64	---	0.62
	Envelope of Pxy and Pyx	(MPa)	---	0.63	---	0.70	---	**0.73**	---	0.63	---	0.64	---	0.62

Notes: *: SASSI2010 analysis is a linear time history analysis with the 3D excitation.
**: EB and MEB stand for energy balance (EB) and modified energy balance (MEB) methods.
The number in bold red (0.73) is the maximum dynamic bearing pressure demand on the Zone III-IV rock.

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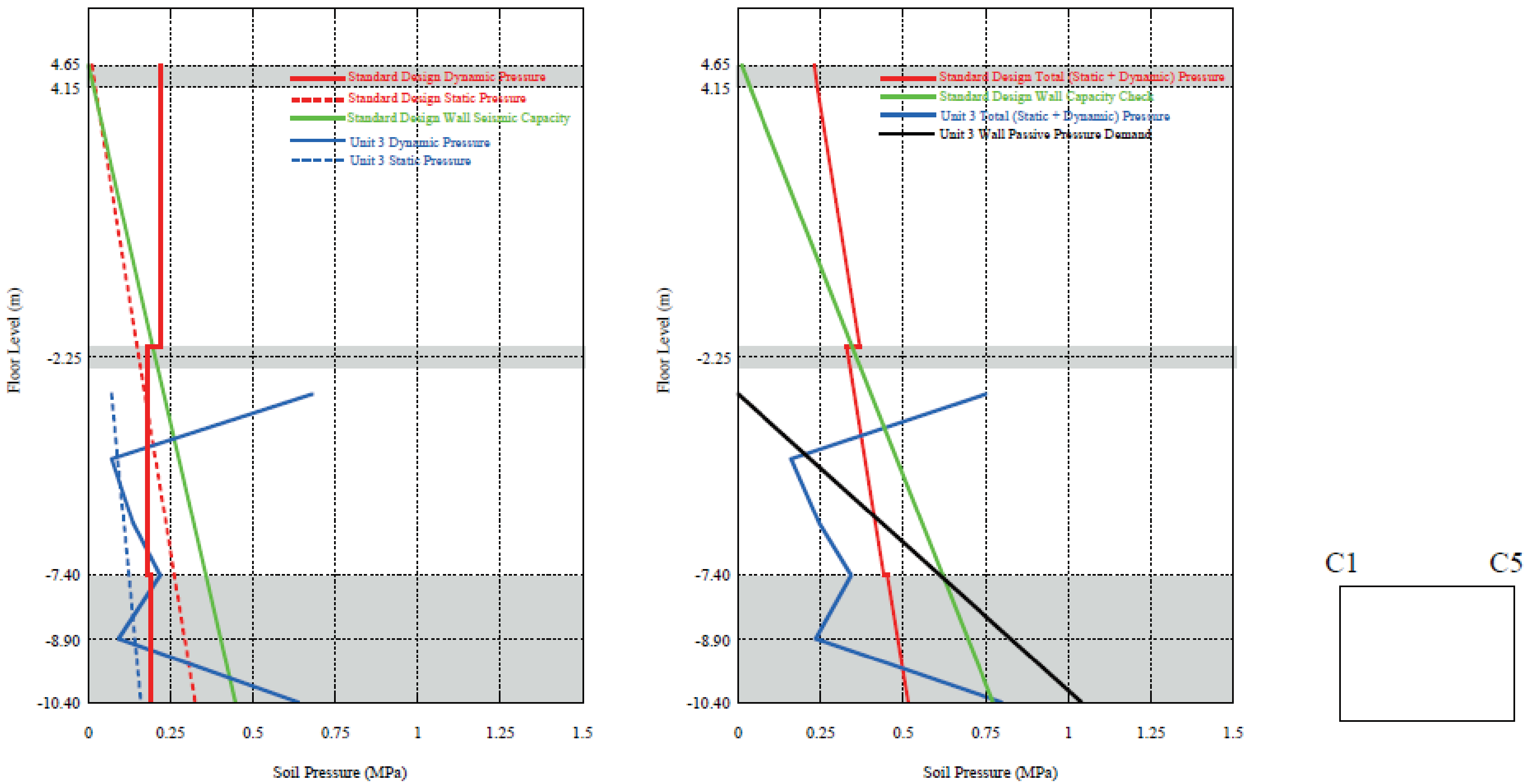
Figure 3G.8-203 **Lateral Pressure C1 Wall - Envelope of LB, BE and UB Partial Column Profiles Analyses**

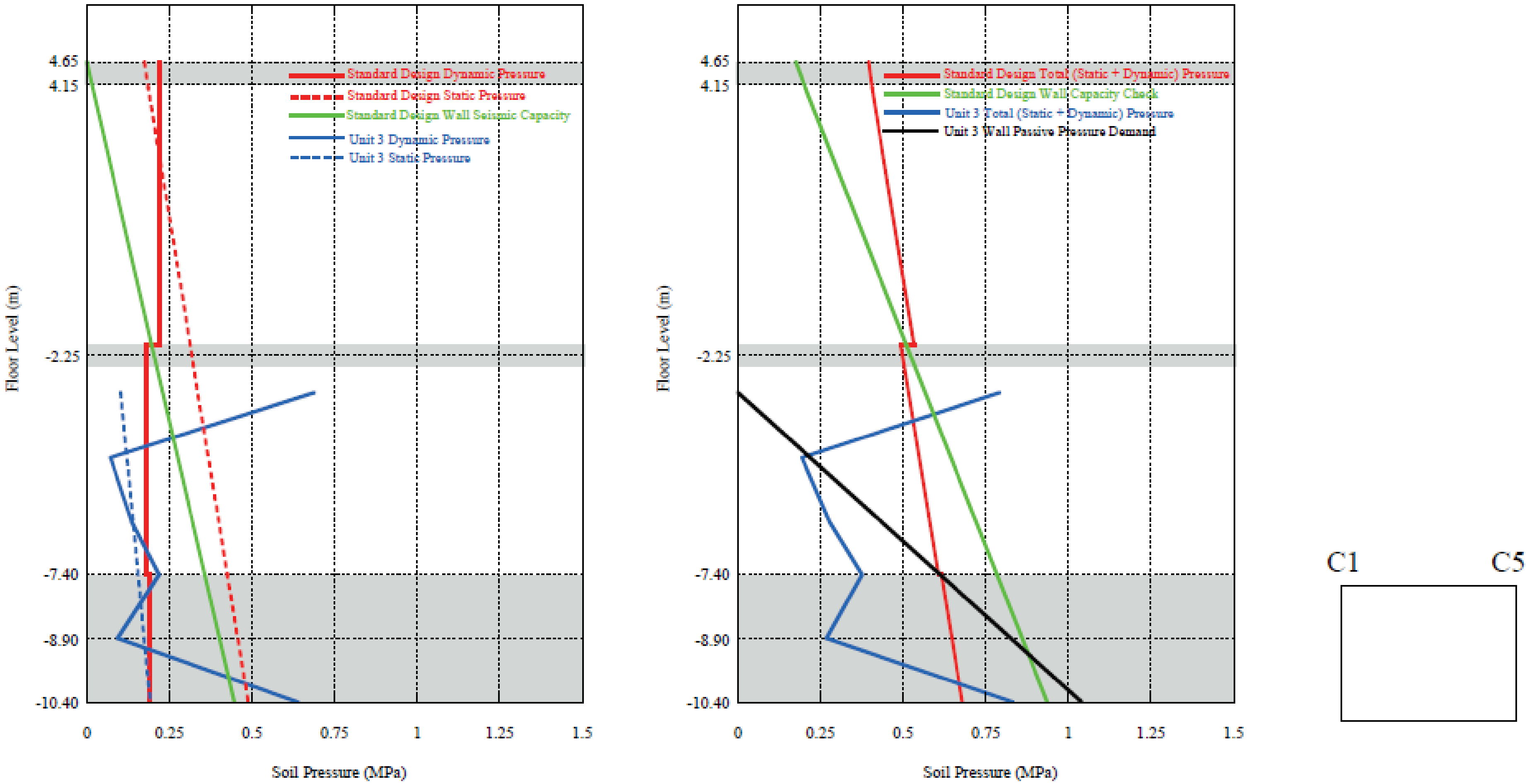


(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-204 **Lateral Pressure C5 Wall - Envelope of LB, BE and UB Partial Column Profiles Analyses**

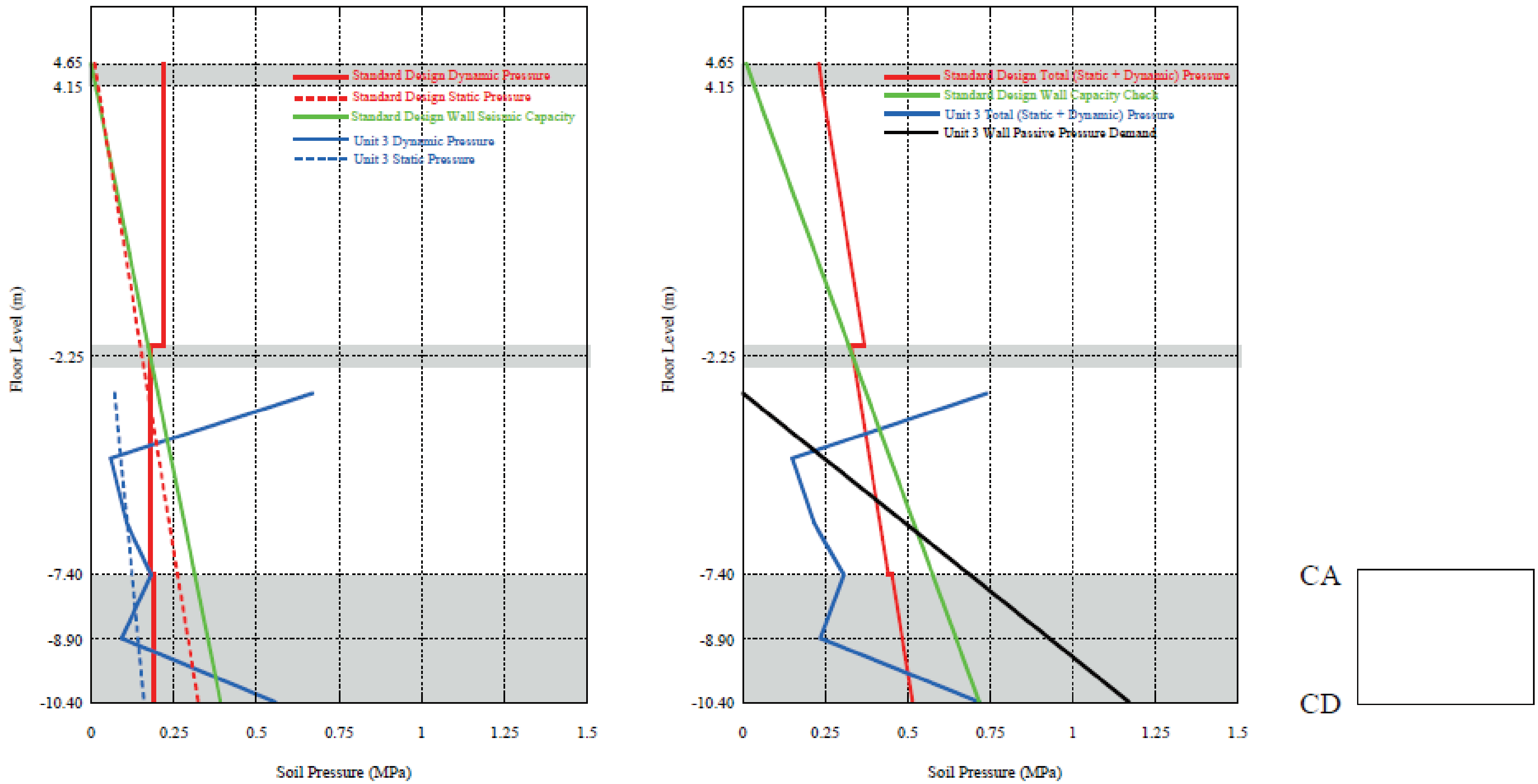


(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-205 Lateral Pressure CA Wall -- Envelope of LB, BE and UB Partial Column Profiles Analyses

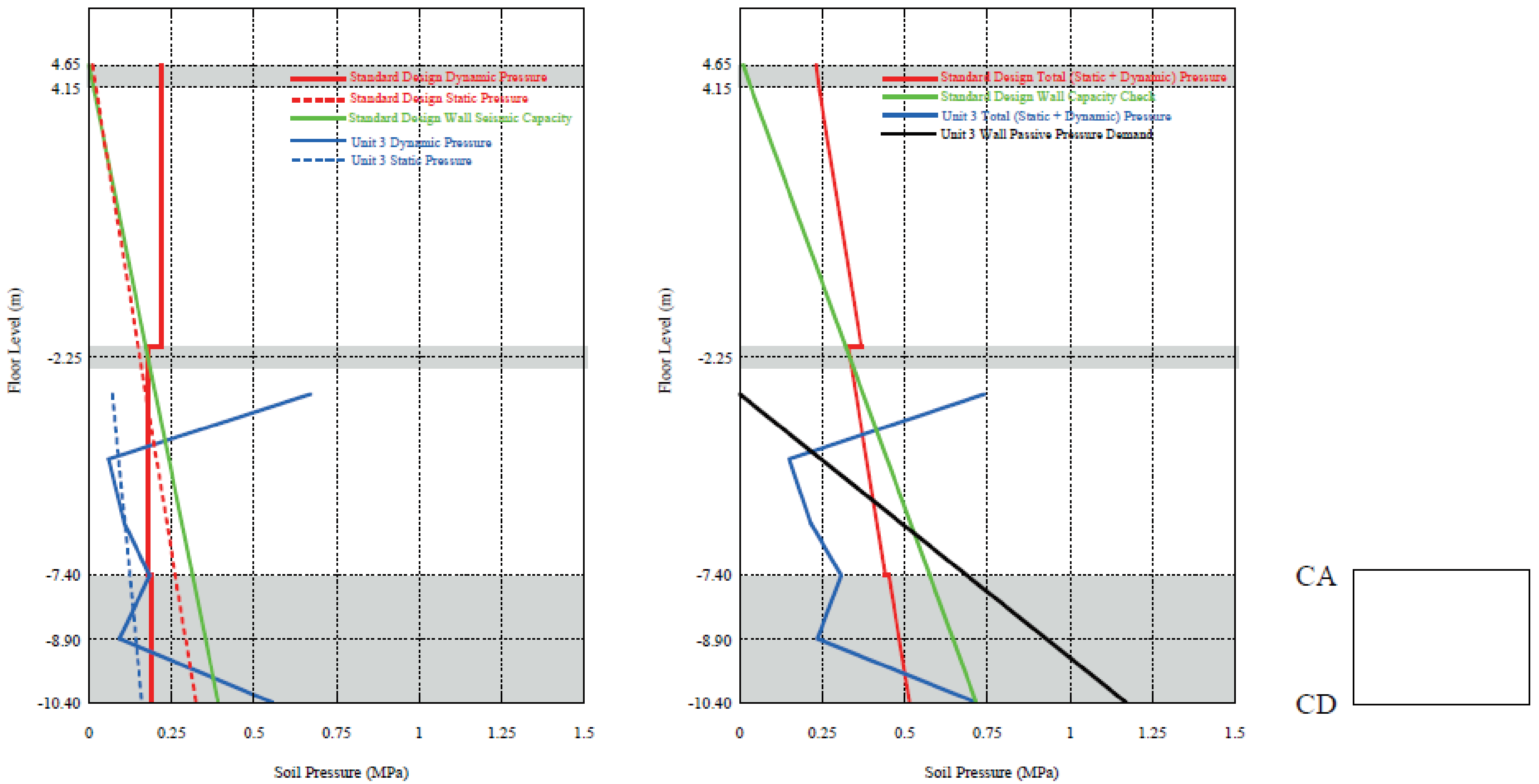


(a) Static Pressure and Dynamic Pressure (b) Total Pressure and Wall Passive Pressure

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-206 Lateral Pressure CD Wall - - Envelope of LB, BE and UB Partial Column Profiles Analyses

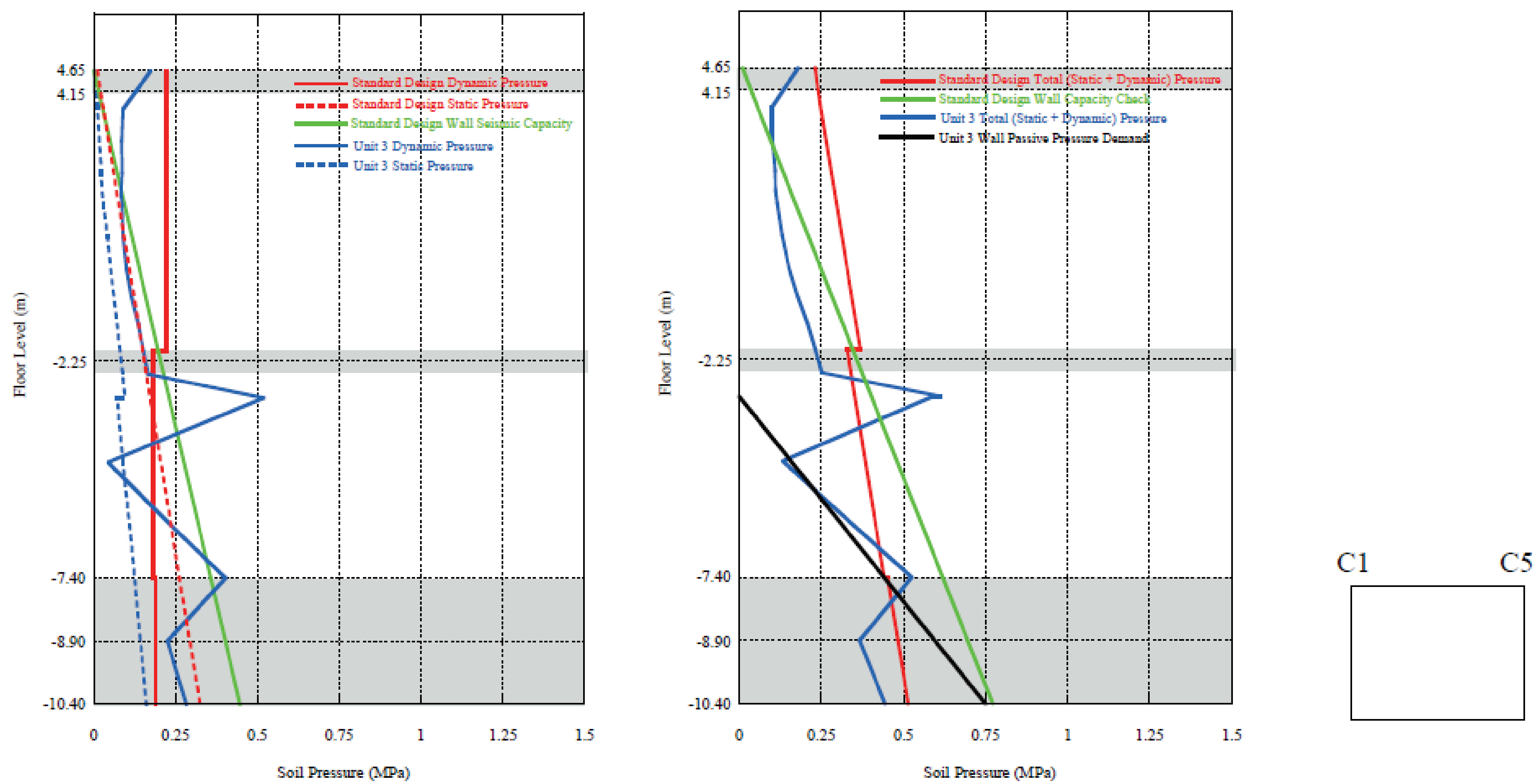


(a) Static Pressure and Dynamic Pressure (b) Total Pressure and Wall Passive Pressure

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-207 **Lateral Pressure C1 Wall - Envelope of LB, BE and UB Full Column Profiles Analyses**

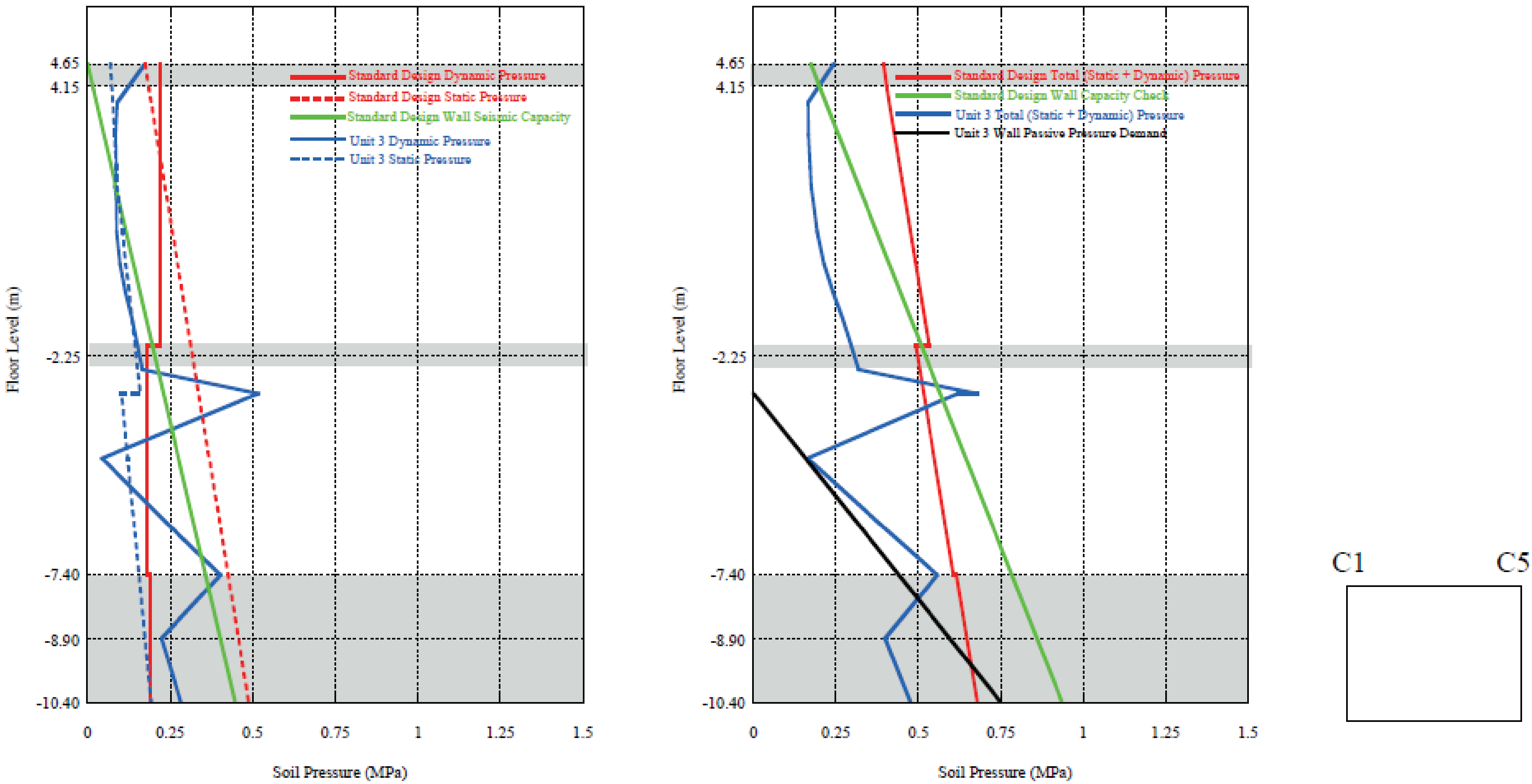


(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-208 **Lateral Pressure C5 Wall - Envelope of LB, BE and UB Full Column Profiles Analyses**

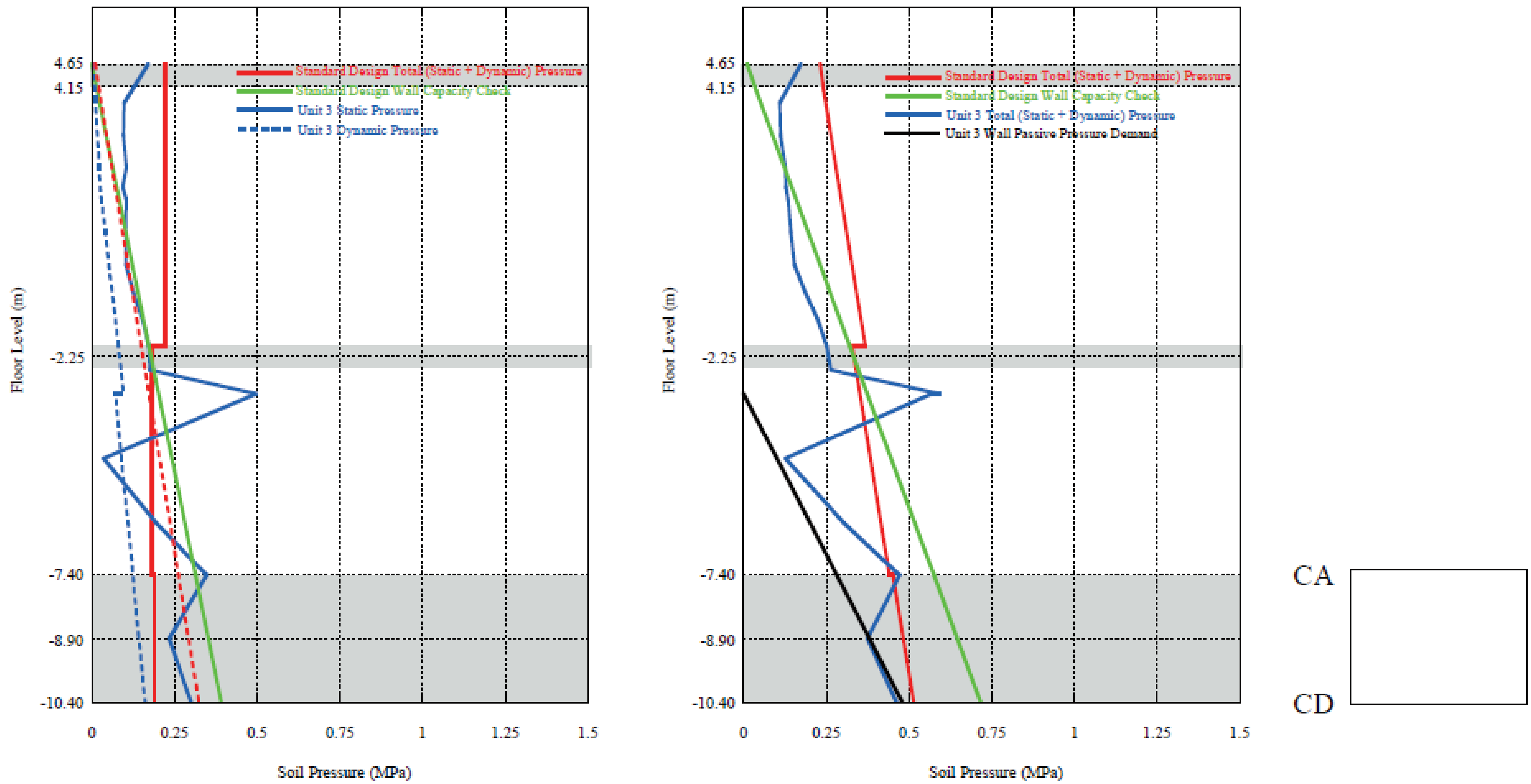


(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-209 **Lateral Pressure CA Wall- - Envelope of LB, BE and UB Full Column Profiles Analyses**

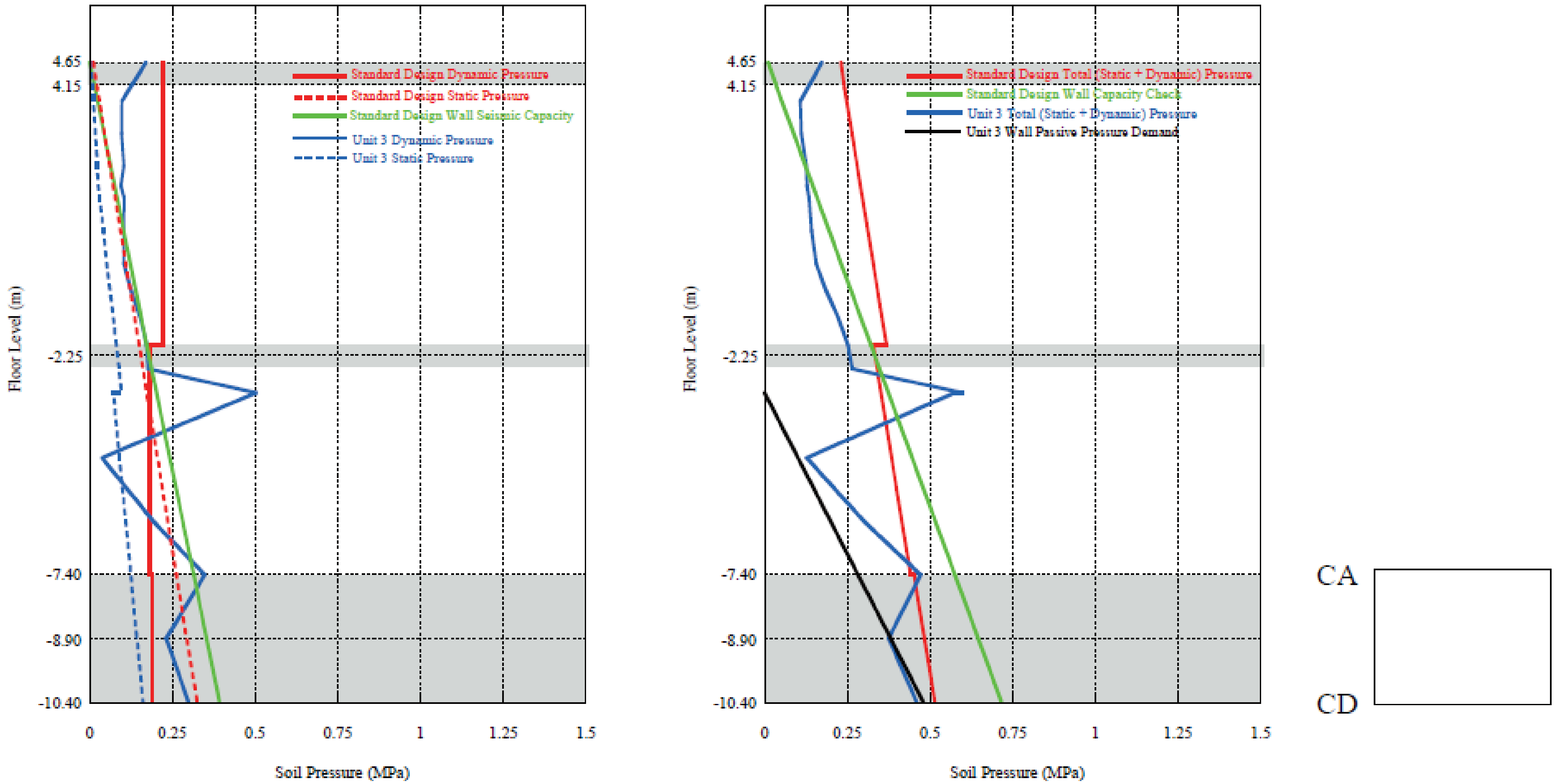


(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

Figure 3G.8-210 **Lateral Pressure CD Wall -- Envelope of LB, BE and UB Full Column Profiles Analyses**



(a) Static Pressure and Dynamic Pressure **(b) Total Pressure and Wall Passive Pressure**

Note: The shaded area shows thickness of the floor slabs and basemat.

NAPS DEP 3.7-1

3G.9 Site-Specific Structural Evaluation of Fuel Building

The FB is evaluated with the RB in Section 3G.7 for the Unit 3 site-specific structural evaluation.

NAPS DEP 3.7-1

3G.10 Site-Specific Structural Evaluation of Fire Water Service Complex

The Unit 3 site-specific structural evaluations of the FWSC are described in this section.

3G.10.1 Objective and Scope

The objective of this section is to document the FWSC site-specific structural evaluations based on the Unit 3 analyses performed to address exceedances of the CSDRS. Results of SSI and SSSI analyses indicate that seismic load demands, in some cases, exceed the seismic load demands used in the standard design. This section describes the site-specific design and analysis of the FWSC structures.

3G.10.2 Conclusions

The major conclusions are summarized.

3G.10.3 Structural Description

The FWSC structures are described in DCD Section 3G.4.3.

3G.10.4 Analytical Models

3G.10.4.1 Structural Models

Unit 3 site-specific structural models are based on the standard design structural models described in DCD Section 3G.2.4.

3G.10.4.2 Foundation Models

Unit 3 site-specific foundation models are based on the standard design foundation models described in DCD Section 3G.2.4.2.

3G.10.5 Structural Analysis and Design

3G.10.5.1 Site Design Parameters

Key site-specific parameters used as inputs for the Unit 3 stability and bearing pressure calculations are identified in Table 3G.10-201.

The stability and bearing pressure calculations are performed using the results obtained from:

- Site-specific SSI analyses of the FWSC stand-alone model with full (uncracked concrete) stiffness properties and SSE damping values for BE, LB, and UB subgrade profiles using the deep input control motion applied at the bottom of the underlying concrete fill (analysis Cases 7 through 9 in Table 3A.15-203)
- Site-specific SSSI analyses of the FWSC-CB combined model with full (uncracked concrete) stiffness properties and SSE damping values for BE, LB, and UB subgrade profiles using the deep input control motion applied at the bottom of the underlying concrete fill (analysis Cases FC7 through FC9 in Table 3A.15-206).

3G.10.5.2 Site Design Loads

The Unit 3 site-specific seismic loads for the FWSC are obtained by site-specific SSI analyses, with results described in Section 3A.18.1.3.

3G.10.5.3 Stability Requirements

The stability requirements for the FWSC are given in DCD Section 3G.4.5.3. A factor of safety of 1.1 is required for both overturning and sliding. Analyses demonstrate that the FWSC meets the required factors of safety for overturning stability and for stability against sliding.

3G.10.5.4 Structural Design Evaluation

[Information for this section will be provided later.]

3G.10.5.5 Foundation Stability

The methodology used for the stability calculations is consistent with the methodology used for the standard design stability evaluations described in DCD Sections 3.7.2.14 and 3.8.5.5. The seismic stability of the FWSC is evaluated against overturning of the FWSC basemat and sliding for two critical sliding planes located as follows:

- Bottom of the FWSC basemat at Elevation 282 ft NAVD88
- Bottom of the underlying concrete fill at nominal Elevation 220 ft NAVD88

Tables 3G.10-214(a), (b), and (c) provide the calculated overturning stability factors of safety for the SSI analyses (Cases 7 through 9 in

Table 3A.15-203) and the SSSI analyses (Cases FC7 through FC9 in Table 3A.15-206). The overturning stability is greater than required.

Sliding stability is evaluated for two orthogonal horizontal directions separately using a linear time history analysis approach. In each direction, the phasing between the horizontal shear and vertical seismic forces is considered at each time step to compute the sliding factor of safety in the time domain. The lateral resistance force demands on the shear keys and subgrade surrounding the concrete fill block under the FWSC are computed if, at the particular instance of time, the friction resistance at the bottom of the FWSC basemat alone is not sufficient to achieve a minimum factor of safety of 1.1 against sliding. Table 3G.10-214(b) shows the site-specific lateral force demands on the FWSC shear keys. These lateral loads demands are used as input for the site-specific evaluation of the FWSC structure to demonstrate that the design of shear keys is adequate for the Unit 3 site.

Table 3G.10-214(b) shows the maximum lateral resistance pressure transfer from the shear keys to the concrete fill supporting the FWSC basemat, which is below the allowable lateral bearing pressure of the concrete fill of 8.0 MPa.

Table 3G.10-214(b) presents a summary of the calculations of the FWSC global stability against sliding at the critical sliding plane at the bottom of the concrete fill, Elevation 220 ft NAVD88. For the instances of time the friction resistance at the bottom of the concrete fill alone is not sufficient, the lateral resistance force required to achieve a minimum factor of safety of 1.1 against sliding is calculated. The value of the maximum site-specific lateral passive pressure demand is below the allowable dynamic lateral bearing pressure of 1.44 MPa of Zone III rock at FWSC location specified in Table 3G.10-201.

Tables 3G.10-215(a) and (b) provide results of calculations of the maximum dynamic bearing pressure demands from the FWSC foundation to the supporting concrete fill. The calculations follow the same EB/MEB method that was used for the standard design calculations. These dynamic bearing pressure demands are compared with the allowable dynamic bearing pressure of the underlying concrete fill and Zone III-IV rock to ensure that the capacity of the subgrade is sufficient to resist the dynamic bearing pressures from the FWSC and the concrete fill supporting the FWSC basemat.

The calculations of the dynamic bearing pressure under the FWSC basemat presented in Table 3G.10-215(a) yield a site-specific demand that is lower than the maximum toe bearing pressure of 1.2 MPa considered by the standard design in DCD Table 3G.4-23 and 8.0 MPa allowable bearing pressure of concrete fill material. Table 3G.10-215(b) presents the calculations of the maximum dynamic bearing pressures on the surface of the underlying Zone III-IV rock. The table shows that the maximum dynamic bearing pressure demand from the FWSC and supporting concrete fill block on the Zone III/IV rock is lower than the rock allowable dynamic bearing pressure specified in Table 3G.10-201.

NAPS DEP 3.7-1

Table 3G.10-201 **FWSC Site-Specific Parameters**

Parameters	Values
Building Width:	
X-direction (NS-direction)	52 m
Y-direction (EW-direction)	20 m
Zone III Rock Embedment Depth:	
Nominal Zone III Rock Elevation	El. 244 ft. NAVD88
Depth of Shear Key	3.6 m (11.8 ft)
Depth to bottom of concrete fill block	7.32 m (24 ft)
Water Level:	
Nominal Groundwater Elevation	El. 281.5 ft NAVD88 2.59 m (8.5 ft) below finished ground level grade
Friction Coefficient, μ:	
Foundation/Concrete Fill/Rock Interfaces	0.6*)
Allowable Lateral Dynamic Bearing Pressure:	
Zone III rock at Elevation 220 ft, NAVD88	1.44 MPa (30.1 ksf)
Allowable Dynamic Bearing Pressure:	
Zone III-IV rock	12.4 MPa (259 ksf)

*) A value of 0.6 is used for the friction coefficient, which is the lowest value specified for concrete fill and Zone III-IV rock and the foundation structural concrete.

NAPS DEP 3.7-1

Table 3G.10-214 Factors of Safety for FWSC Foundation Stability

(a) FWSC Overturning Stability Evaluation

Subgrade Condition	SSI Analyses Cases						SSSI Analyses Cases					
	LB (Case 7)		BE (Case 8)		UB (Case 9)		LB (Case FC7)		BE (Case FC8)		UB (Case FC9)	
Direction	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW	NS	EW
m_0gh (MN·m)	3402	946	3402	946	3402	946	3402	946	3402	946	3402	946
W_b (MN·m)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
E_s (MN·m)	0.8	0.7	0.9	0.8	1.0	1.0	0.8	0.7	0.9	0.8	1.1	1.0
$FS=(m_0gh-W_b)/E_s$	4,099	1,284	3,636	1,127	3,249	933	4,163	1,305	3,702	1,127	3,183	**902**

Notes:

m_0 = total mass of structure and basemat
g = acceleration due to gravity
h = height of the center of structure mass at the overturning position

W_b = potential energy caused by the effect of buoyancy
E_s = maximum kinetic energy
FS = Factor of Safety

The effect of shear key is neglected conservatively.
The bold red number is the minimum Factor of Safety against overturning.

NAPS DEP 3.7-1

Table 3G.10-214 **Factors of Safety for RB/FB Foundation Stability** *(continued)*

(b) Evaluation of FWSC Stability for Sliding at Bottom of Basemat (Elevation 282 ft NAVD88)

Basemat width in NS Dir.	52.0	m
Basemat width in EW Dir.	20.0	m
Depth of Zone III rock embedment	3.6	m
Total Weight	69	MN
Buoyancy	6	MN

	SSI Analyses Cases						**SSSI Analyses Cases**					
Subgrade Condition	**LB (Case 7)**		**BE (Case 8)**		**UB (Case 9)**		**LB (Case FC7)**		**BE (Case FC8)**		**UB (Case FC9)**	
Sliding Direction	**NS**	**EW**	**NS**	**NS dir**	**EW dir**	**NS dir**	**NS dir**	**EW dir**	**NS dir**	**EW**	**NS**	**EW**
Time (sec)	3.095	4.955	1.155	1.810	1.060	1.645	1.070	4.955	1.110	1.140	1.140	1.645
Vertical Seismic Load (MN)	11	55	99	103	71	74	58	57	48	67	92	68
Minimum Vertical Load (MN)	151	107	64	60	92	89	104	106	114	95	71	95
F_v: Horizontal Seismic Force (MN)	99	70	86	66	92	96	74	71	97	75	87	109
F_{ub}: Bottom Friction Force (MN)	91	64	38	36	55	53	63	64	69	57	42	57
F_r': Lateral Resistance Force at Shear Key (MN)	18	12	**56**	37	46	52	19	15	38	25	53	**63**
FS (=(F_{ub}+F_r')/F_v)	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
σ_{max}: Maximum Stress (MPa) Associated with Lateral Resistance F_r'	0.28	0.14	**0.87**	0.43	0.71	0.60	0.30	0.17	0.59	0.29	0.82	0.73

The bold red numbers are the maximum lateral pressure demands.

NAPS DEP 3.7-1

Table 3G.10-214 **Factors of Safety for RB/FB Foundation Stability** *(continued)*

(c) Evaluation of FWSC Stability for Sliding at Bottom of Concrete Fill (Elevation 220 ft NAVD88)

Basemat width in NS Dir.	52.0	m
Basemat width in EW Dir.	20.0	m
Depth of Zone III rock to bottom of concrete fill block	7.32	m
Total Weight	618	MN
Buoyancy	191	MN

	SSI Analyses Cases						**SSSI Analyses Cases**					
Subgrade Condition	**LB (Case 7)**		**BE (Case 8)**		**UB (Case 9)**		**LB (Case FC7)**		**BE (Case FC8)**		**UB (Case FC9)**	
Sliding Direction	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**	**NS**	**EW**
Time (sec)	3.085	1.815	1.065	1.810	1.060	1.645	3.080	1.810	1.060	1.810	1.055	1.645
Vertical Seismic Load (MN)	81	140	214	183	212	201	78	241	233	284	167	184
Minimum Vertical Load (MN)	345	187	214	144	215	226	349	186	194	183	260	243
F_v: Horizontal Seismic Force (MN)	253	118	201	213	236	183	254	135	215	184	257	202
F_{ub}: Bottom Friction Force (MN)	208	112	128	86	129	136	209	112	116	86	156	146
F_r: Lateral Resistance Force (MN)	71	17	93	147	130	66	70	37	120	117	127	77
FS $(=(F_{ub}+F_r)/F_v)$	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
σ_{max}: Maximum Stress (MPa) Associated with Lateral Resistance F_r'	0.48	0.04	0.63	0.39	**0.89**	0.17	0.48	0.10	0.82	0.31	0.86	0.20

The bold red numbers are the maximum lateral pressure demands.

NAPS DEP 3.7-1

Table 3G.10-215 **Maximum Soil Dynamic Bearing Pressure Demand for FWSC**

(a) Calculations of Dynamic Bearing Pressure Demands on Concrete Fill under FWSC Basemat

Basemat width in NS (x) Dir.	52.0	m
Basemat width in EW (y) Dir.	20.0	m
Gravity Load (D)	169	MN
Buoyancy (B)	6	MN (considered only in combination with upward vertical seismic load (V_z))

			SSI Analyses Cases						SSSI Analyses Cases					
Subgrade Condition			**LB (Case 7)**		**BE (Case 8)**		**UB (Case 9)**		**LB (Case FC7)**		**BE (Case FC8)**		**UB (Case FC9)**	
Direction Vertical Seismic Load			**upward**		**upward**		**upward**		**downward**		**upward**		**upward**	
SASSI*	Time	(sec)	1.115		1.145		1.140		1.115		1.140		1.140	
	Vertical seismic load (V_z)	(MN)	15		61		87		28		67		92	
	Total vertical load	(MN)	148		102		76		196		95		71	
	Moment in NS-dir (Mx)	(MN-m)	742		784		732		697		454		674	
	Moment in EW-dir (My)	(MN-m)	703		818		898		555		799		756	
Simplified Method**			**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**
NS dir. → EW dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	5.3	5.7	0.0	0.0	0.0	0.0	4.8	5.1
	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.04	-0.04	-0.05	-0.05	0.00	0.00	0.011	0.011	0.004	0.004	0.007	0.008
	Max. basemat moment (Mx)	(MN-m)	742	742	784	784	732	734	697	697	454	454	674	675
	Max. bearing pressure 1 (Px)	(MPa)	0.22	0.22	0.18	0.18	0.15	0.16	0.27	0.27	0.14	0.14	0.14	0.14
	Max. bearing pressure 2 (Py)	(MPa)	---	0.20	---	0.24	---	0.27	---	0.16	---	0.23	---	0.23
	Max. Bearing pressure (Pxy=Px+Py)	(MPa)	---	0.43	---	0.42	---	0.43	---	0.43	---	0.37	---	0.37
EW dir. → NS dir.	Max. basemat uplift ratio α	(%)	18.0	21.9	56.5	56.1	58.4	74.5	0.0	0.0	55.0	58.6	54.9	70.5
	Max. basemat rotation (ϕ)	(10^{-4} rad)	0.28	0.31	0.00	-0.01	0.23	0.62	0.01	0.01	0.06	0.11	0.05	0.13
	Max. basemat moment (My)	(MN-m)	703	708	818	720	898	631	555	555	799	689	756	568
	Max. bearing pressure 1(Py)	(MPa)	0.34	0.36	0.33	0.45	0.33	0.57	0.35	0.35	0.32	0.44	0.29	0.46
	Max. bearing pressure 2 (Px)	(MPa)	---	0.11	---	0.20	---	0.32	---	0.08	---	0.12	---	0.25
	Max. Bearing Pressure (Pyx=Py+Px)	(MPa)	---	0.47	---	0.64	---	0.89	---	0.43	---	0.56	---	0.71
	Envelope of Pxy and Pyx	(MPa)	---	0.47	---	0.64	---	**0.89**	---	0.43	---	0.56	---	0.71

Notes: *: SASSI2010 analysis is a linear time history analysis with the 3D excitation.
**: EB and MEB stand for energy balance (EB) and modified energy balance (MEB) methods.
The bold red number is the maximum dynamic toe bearing pressure demand..

NAPS DEP 3.7-1

Table 3G.10-215 **Maximum Soil Dynamic Bearing Pressure Demand for FWSC** *(continued)*

(b) Calculations of Dynamic Bearing Pressure Demands on Zone III-IV Rock

Basemat width in NS (x) Dir.	52.0	m
Basemat width in EW (y) Dir.	20.0	m
Gravity Load (D)	618	MN
Buoyancy (B)	191	MN (considered only in combination with upward vertical seismic load (V_z))

			SSI Analyses Cases						SSSI Analyses Cases					
Subgrade Condition			**LB (Case 7)**		**BE (Case 8)**		**UB (Case 9)**		**LB (Case FC7)**		**BE (Case FC8)**		**UB (Case FC9)**	
Direction Vertical Seismic Load			**upward**		**upward**		**upward**		**downward**		**downward**		**upward**	
SASSI*	Time	(sec)	3.035		1.810		5.275		3.035		3.085		3.025	
SASSI*	Vertical seismic load (V_z)	(MN)	39		284		45		37		65		83	
SASSI*	Total vertical load	(MN)	389		143		382		655		684		344	
SASSI*	Moment in NS-dir (Mx)	(MN-m)	1991		1135		1178		2886		5074		5961	
SASSI*	Moment in EW-dir (My)	(MN-m)	2409		1870		2637		1637		686		1072	
Simplified Method**			**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**	**EB**	**MEB**
NS dir. ↓ EW dir.	Max. basemat uplift ratio α	(%)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	34.9	44.7
NS dir. ↓ EW dir.	Max. basemat rotation (ϕ)	(10^{-4} rad)	0.002	0.002	0.003	0.003	-0.003	-0.003	-0.004	-0.004	-0.004	-0.004	-0.005	-0.007
NS dir. ↓ EW dir.	Max. basemat moment (Mx)	(MN-m)	1991	1991	1135	1135	1178	3310	2886	2886	5074	5074	5961	5646
NS dir. ↓ EW dir.	Max. bearing pressure 1 (Px)	(MPa)	0.85	0.85	0.99	0.99	0.77	0.77	0.95	0.95	1.22	1.22	0.99	1.20
NS dir. ↓ EW dir.	Max. bearing pressure 2 (Py)	(MPa)	---	0.69	---	0.54	---	0.76	---	0.47	---	0.20	---	0.56
NS dir. ↓ EW dir.	Max. Bearing pressure (Pxy=Px+Py)	(MPa)	---	1.55	---	1.53	---	1.53	---	1.42	---	1.42	---	1.76
EW dir. ↓ NS dir.	Max. basemat uplift ratio α	(%)	31.5	39.9	38.0	78.3	36.5	47.0	0.0	0.0	0.0	0.0	-3.3	0.0
EW dir. ↓ NS dir.	Max. basemat rotation (ϕ)	(10^{-4} rad)	-0.10	-0.13	-0.21	-0.66	0.41	0.59	0.005	0.005	0.004	0.004	0.003	0.003
EW dir. ↓ NS dir.	Max. basemat moment (My)	(MN-m)	2409	2330	1870	1222	2637	2469	1637	1637	686	2279	1072	1074
EW dir. ↓ NS dir.	Max. bearing pressure 1(Py)	(MPa)	1.07	1.24	0.68	1.27	1.13	1.39	1.10	1.10	0.86	0.86	0.64	0.64
EW dir. ↓ NS dir.	Max. bearing pressure 2 (Px)	(MPa)	---	0.37	---	0.58	---	0.25	---	0.32	---	0.56	---	0.66
EW dir. ↓ NS dir.	Max. Bearing Pressure (Pyx=Py+Px)	(MPa)	---	1.61	---	1.85	---	1.63	---	1.42	---	1.42	---	1.30
	Envelope of Pxy and Pyx	(MPa)	---	1.61	---	**1.85**	---	1.63	---	1.42	---	1.42	---	1.76

Notes: *: SASSI2010 analysis is a linear time history analysis with the 3D excitation.
**: EB and MEB stand for energy balance (EB) and modified energy balance (MEB) methods.
The bold red number is the maximum dynamic toe bearing pressure demand..

Appendix 3H Equipment Qualification Design Environmental Conditions

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3I Designated NEDE-24326-1-P Material Which May Not Change Without Prior NRC Approval

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3J Evaluation of Postulated Ruptures in High Energy Pipes

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3K Resolution of Intersystem Loss of Coolant Accident

This section of the referenced DCD is incorporated by reference with no departures or supplements.

Appendix 3L Reactor Internals Flow Induced Vibration Program

This section of the referenced DCD is incorporated by reference with no departures or supplements.

2.4.2 ~~Not used.~~ ITAAC for Structural Fill Surrounding Seismic Category I Structures

Design Description

Structural fill surrounding the embedded walls for Seismic Category I structures meets properties for (1) the angle of internal friction; (2) the local effect on wall pressure as determined by the product of: peak ground acceleration α, (in g), Poisson's ratio ν, and density γ; and (3) soil density.

Inspections, Test, Analyses and Acceptance Criteria

Table 2.4.2-1 provides a definition of the inspections, tests, and/or analyses, together with associated acceptance criteria for the Structural Fill.

Table 2.4.2-1 ITAAC for Structural Fill Surrounding Seismic Category I Structures

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
1. The structural fill material surrounding Seismic Category I structures meets the following properties: • the angle of internal friction ≥35 degrees • the local effect on wall lateral pressures ≤1220 kg/m^3 (76 lbf/ft^3), as determined by the following equation: $\alpha(0.95\nu + 0.65)\gamma$ where: α = peak ground acceleration (in g) ν = Poisson's ratio γ = density • the soil density $\gamma \geq 2000$ kg/m^3 (125 lbf/ft^3).	Tests, inspections, analyses, or a combination thereof, will be performed to evaluate the properties of the structural fill.	A report exists and concludes that the tests, inspections, analyses, or a combination thereof, confirm that the structural fill material surrounding Seismic Category I structures meets the following properties: • the angle of internal friction ≥35 degrees • the local effect on wall lateral pressures ≤1220 kg/m^3 (76 lbf/ft^3), as determined by the following equation: $\alpha(0.95\nu + 0.65)\gamma$ where: α = peak ground acceleration (in g) ν = Poisson's ratio γ = density • the soil density $\gamma \geq 2000$ kg/m^3 (125 lbf/ft^3).

Table 2.4.15-1 ITAAC for the Turbine Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. The Turbine Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Turbine Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Turbine Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Turbine Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Turbine Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.
2. The design of the Turbine Building precludes any adverse interaction with Seismic Category I structures.	Perform site-specific structure-soil-structure interactions analyses to evaluate seismic interaction between the Turbine Building and adjacent Seismic Category I structures, using methodology consistent with that used for the Seismic Category I structures.	Site-specific structure-soil-structure interactions analyses conclude there are no adverse interactions between the Turbine Building and Seismic Category I structures.

Table 2.4.16-1 ITAAC for the Radwaste Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. The Radwaste Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Radwaste Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Radwaste Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Radwaste Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Radwaste Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.
2. The Radwaste Building has an exterior wall static pressure capacity of at least 3 psi.	Perform an analysis to determine the static wall pressure capacity of the exterior walls of the as-built Radwaste Building.	Results of the Radwaste Building analysis demonstrate that the exterior wall static pressure capacity is at least 3 psi.
3. The design of the Radwaste Building precludes any adverse interaction with Seismic Category I structures.	Perform site-specific structure-soil-structure interactions analyses to evaluate seismic interaction between the Radwaste Building and adjacent Seismic Category I structures, using methodology consistent with that used for the Seismic Category I structures.	Site-specific structure-soil-structure interactions analyses conclude there are no adverse interactions between the Radwaste Building and Seismic Category I structures.

Table 2.4.17-1 ITAAC for the Service Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. The Service Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Service Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Service Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Service Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Service Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.
2. The design of the Service Building precludes any adverse interaction with Seismic Category I structures.	Perform site-specific structure-soil-structure interactions analyses to evaluate seismic interaction between the Service Building and adjacent Seismic Category I structures, using methodology consistent with that used for the Seismic Category I structures.	Site-specific structure-soil-structure interactions analyses conclude there are no adverse interactions between the Service Building and Seismic Category I structures.

Table 2.4.18-1 ITAAC for the Ancillary Diesel Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. The Ancillary Diesel Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Ancillary Diesel Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Ancillary Diesel Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Ancillary Diesel Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Ancillary Diesel Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.
2. The design of the Ancillary Diesel Building precludes any adverse interaction with Seismic Category I structures.	Perform site-specific structure-soil-structure interactions analyses to evaluate seismic interaction between the Ancillary Diesel Building and adjacent Seismic Category I structures, using methodology consistent with that used for the Seismic Category I structures.	Site-specific structure-soil-structure interactions analyses conclude there are no adverse interactions between the Ancillary Diesel Building and Seismic Category I structures.